Pile Design and Construction Rules of Thumb

SECOND EDITION

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Site investigation and soil conditions

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1.1 Origin of rocks and soils

Geotechnical engineers for the most part deal with rocks and soils. Therefore, it is important to understand rock and soil properties. First let us look at the origin of soils and rocks.

What came first: rocks or soils? The answer is rocks. There were rocks on Earth before sands. Let us see how this happened. The universe was full of dust clouds. Dust particles, due to gravity, became small-sized objects. They were known as planetesimals. These planetesimals smashed on to each other and larger planets, such as Earth, were formed.

$$\text{Dust cloud} \rightarrow \text{small planets (planetesimals)} \rightarrow \text{collisions} \rightarrow \text{planets}$$

Many dust clouds can be seen in space with telescopes. The horsehead nebula is the most famous dust cloud. Dust clouds gave rise to millions of smaller planets that smashed on to each other. The early Earth was a very hot place due to these collisions. The Earth was too hot to have water on the surface. All water evaporated and existed in the atmosphere.

Due to extreme heat, the whole Earth was covered in a lava ocean.

1.1.1 Earth cools down

Millions of years later, the Earth cooled down. Water vapor that was in the atmosphere started to fall on the Earth. Rain started to fall on Earth for the first time. This rain could have lasted for millions of years. Oceans were formed and the molten Lava Ocean became a huge rock. Unfortunately, we have not found a single piece of rock that formed from the very first molten lava ocean.

1.1.2 Rock weathering

The flow of water, change in temperatures, volcanic actions, chemical actions, earthquakes, and falling meteors broke rocks into much smaller pieces. Millions of years later, rocks have broken down to pieces so small that we differentiate them from rocks by calling them sand. Hence for our first question, ‘which came first, rock or sand?’ The answer is rocks came first.

- Basalt
- Diorite

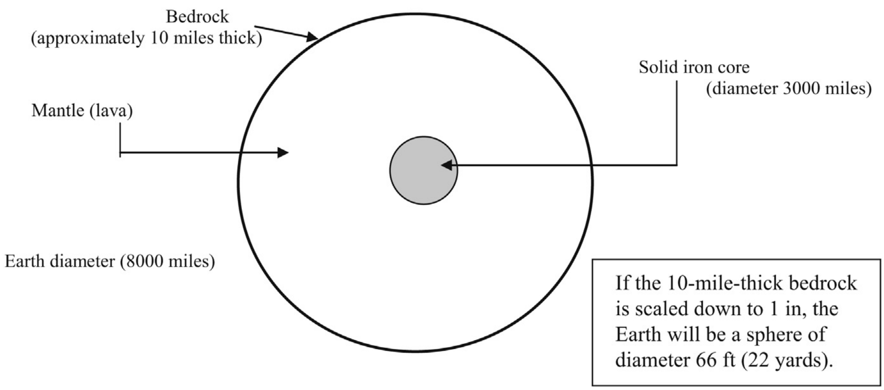


Figure 1.1 The Earth.

Dust cloud → small planets (planetesimals) → planets → lava ocean → lava ocean cools down to form rocks → first rains → oceans → weathering of rocks → formation of sand and clay particles

1.1.3 Brief overview of rocks

All rocks are basically divided into three categories. They are as follows:

- igneous rocks
- sedimentary rocks
- metamorphic rocks

1.1.3.1 Igneous rocks

Igneous rocks were formed from solidified lava. Earth's diameter is measured to be approximately 8000 miles and the bedrock is estimated to be only 10 miles. Earth has a solid core with a diameter of 3000 miles and the rest is all lava or known as the mantle. Occasionally, lava comes out of the Earth during volcanic eruptions, cools down, and becomes rock. Such rocks are known as igneous rocks (Fig. 1.1).

Some of the common igneous rocks are as follows:

- granite
- diabase
- basalt
- diorite

Igneous rocks have two main divisions.

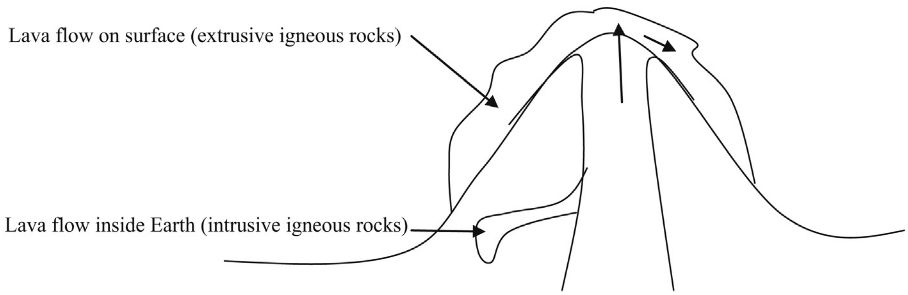


Figure 1.2 Intrusive and extrusive rocks.

1.1.3.1.1 Extrusive igneous rocks

We all have seen lava flowing on the surface of the Earth, on the television. When lava is cooled on the surface of the Earth, extrusive igneous rocks are formed. Typical examples for this type of rocks are andesite, basalt, obsidian, pumice, and rhyolite.

1.1.3.1.2 Intrusive igneous rocks

This type of rock is formed when lava flows inside the Earth. Typical examples are diorite, gabbro, and granite (Fig. 1.2).

The structure of intrusive igneous rocks and extrusive igneous rocks are different.

1.1.3.2 Sedimentary rocks

Volcanic eruption is not the only process of rock origin. Soil particles constantly deposit in lakebeds and ocean floors. Millions of years later, these depositions get solidified and are converted into rock. Such rocks are known as sedimentary rocks (Fig. 1.3).

Some of the common sedimentary rocks are as follows:

- sandstones
- shale
- mudstone
- limestone

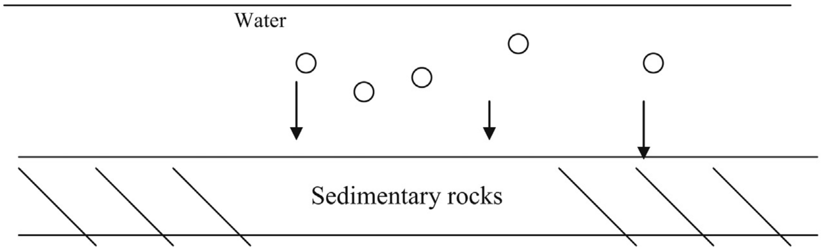


Figure 1.3 Sedimentary process.

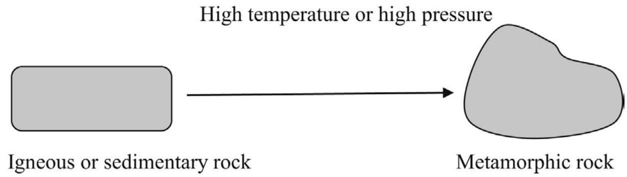


Figure 1.4 Metamorphic rocks.

Imagine a bowl of sugar left all alone for a few months. Sugar particles would bind together due to chemical forces acting between them and solidify. Similarly, sandstone is formed when sand is left under pressure for thousands of years. Limestone, siltstone, mudstone, and conglomerate form in a similar fashion.

Lime → limestone
Mud → mudstone (also known as shale)
Silt → siltstone
Mixture of sand, stone, and mud → conglomerate

1.1.3.3 Metamorphic rocks

Other than these two rock types, geologists have discovered another one. The third rock type is known as *metamorphic rock*. Imagine a butterfly metamorphosing from a caterpillar. The caterpillar will transform into a completely different creature.

Early geologists had no problem in identifying sedimentary and igneous rocks. They were not sure what to do with the third type of rock. They did not look like solidified lava. Also, they did not look like sedimentary rocks. Metamorphic rocks were formed from previously existing rocks due to heavy pressure and temperature. Volcanoes, meteors, earthquakes, and plate tectonic movements could generate huge pressures and very high temperatures (Fig. 1.4).

1.1.3.3.1 Formation of metamorphic rocks

Sedimentary and igneous rocks could transform into metamorphic rocks. For example, when shale or mudstone is subjected to very high temperatures, it becomes schist. When limestone is subjected to high temperatures, it transforms into marble. Some metamorphic rocks and their parent rocks are as follows.

- Metamorphic rock gneiss: parent rock could be shale or granite
- Metamorphic rock schist: parent rock is shale
- Metamorphic rock marble: parent rock is limestone

Shale produces the greatest diversity of metamorphic rocks. Metamorphic rocks, such as slate, phyllite, schist, and gneiss, all come from shale.

1.2 Soil strata types

Soils strata are formed due to the forces of nature. The following main forces can be identified for the formation of soil strata:

- water (soil deposits in river beds, ocean floor, lake beds, and river deltas)
- wind (deposits of soil due to wind)
- glacial (deposits of soil due to glacier movement)
- gravitational forces (deposits due to landslides)
- organic (deposits due to organic matter such as trees, animals, and plants)
- weathered in situ (rocks or soil can weather at the same location)

1.2.1 Water

1.2.1.1 Alluvial deposits (river beds)

Soils deposited in riverbeds are known as alluvial deposits. Some textbooks use the word 'fluvial deposits' for the same thing. The size of particles deposited in riverbeds depends on the speed of flow. If the flow of a river is strong, only large cobble-type material can get deposited.

1.2.1.2 Marine deposits

Soil deposits in ocean beds are known as marine deposits or marine soils. Though oceans can be very violent, seabeds are very calm for the most part. Hence, very small particles would deposit upon seabeds. Texture and composition depends on proximity to land and biological matter.

1.2.1.3 Lacustrine deposits (lake beds)

Typically, very small particles deposit on lakebeds due to the tranquility of water. Lake deposits are mostly clays and silts.

1.2.2 Wind deposits (eolian deposits)

Wind can carry particles and create deposits. These wind-borne deposits are known as eolian deposits.

- Loess: loess soil is formed due to wind effects. The main characteristic of loess is that it does not have stratifications. Instead the whole deposit is one clump of soil.
- Eolian sand (sand dunes): sand dunes are known as eolian sands. Sand dunes are formed in areas that have a desert climate.
- Volcanic ash: volcanic ash is carried away by wind and gets deposited.

1.2.3 Glacial deposits

Ice ages come and go. The last ice age came 10,000 years ago. During ice ages, glaciers would be marching down to the southern part of the world from the north.

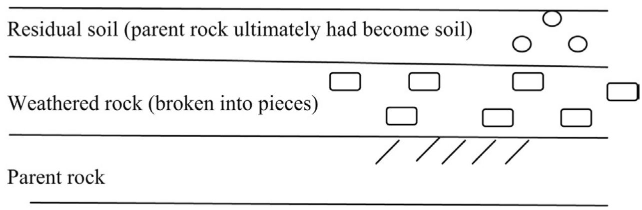


Figure 1.5 Residual soil.

During the last ice age, New York City was under 1000 ft. of ice. The same could be said about London and some other European cities. Glaciers bring in material. When the glaciers melt away, material that was brought in by glaciers would be left behind.

- Glacial till (moraine): glacial till contains particles of all sizes. Boulders to clay particles will be seen in glacial till. When the glacier is moving, it would scoop up any material on the way.
- Glacio-fluvial deposits: during summer months, glaciers would melt. Water from molten glacier would run away with material and deposit along the way. Glacio-fluvial deposits are different from glacial till.
- Glacio-lacustrine deposits: deposits are made by glacial melt water in lakes. Due to low energy in lakes, glacio-lacustrine deposits are mostly silts and clays.
- Glacio-marine deposits: as the name indicates, glacial deposits in oceans are known as glacio-marine deposits.

1.2.4 Colluvial deposits

Colluvial deposits occur due to landslides. The deposits due to landslides contain particles of all sizes. Large and heavy particles would be at the bottom of the mountain.

1.2.5 Residual soil (weathered in situ soil)

Residual soils form when soil or rock, weather at the same location due to chemicals, water, and other environmental elements, without being transported. Another name for residual soil is laterite soil. The main cause behind weathering in residual soils is chemicals (Fig. 1.5).

Quiz

1. Name three sedimentary rock types.
2. Are eolian deposits made by rivers or wind?
3. Are fluvial deposits made by rivers or lakes?
4. Residual soils are formed due to what process?

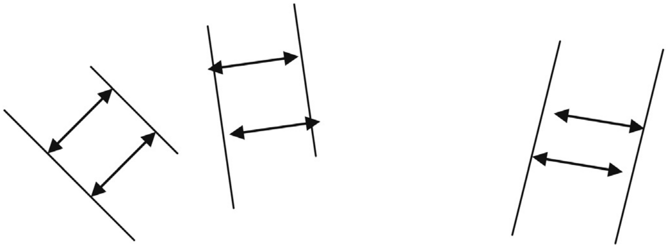


Figure 1.6 Electrochemical bonding of clay particles that gives rise to cohesion.

Answers

1. limestone, conglomerate, and sandstone
2. wind
3. rivers
4. weathered in situ or weathering of soil at the same place of origin

1.3 Site investigation

Soils are of interest to many professionals. Soil chemists are interested in the chemical properties of soil. Geologists are interested in the origin and history of soil strata formation. Geotechnical engineers are interested in the strength characteristics of soil.

Soil strength is dependent on cohesion and friction between soil particles.

1.3.1 Cohesion

Cohesion is developed due to adhesion of clay particles generated by electromagnetic forces. Cohesion is developed in clays and plastic silts while friction is developed in sands and nonplastic silts (Fig. 1.6).

1.3.2 Friction

Sandy particles, when brushed against each other, will generate friction. Friction is a physical process, whereas cohesion is a chemical process. Soil strength generated due to friction is represented by friction angle (φ) (Fig. 1.7).

Cohesion and friction are the most important parameters that determine the strength of soils.

1.3.3 Measurement of friction

Friction angle of soil is usually obtained from correlations available with the standard penetration test value known as SPT (N). Standard penetration test is conducted by

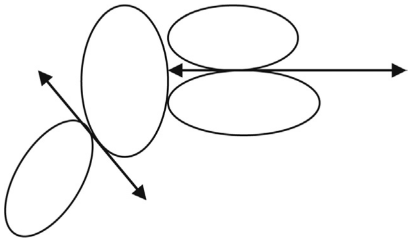


Figure 1.7 Friction in sand particles.

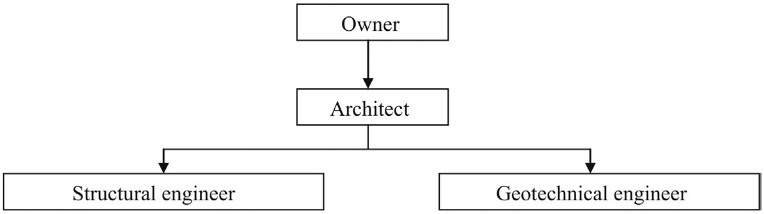


Figure 1.8 Relationship between owner and other professionals.

dropping a 140 lb hammer from a distance of 30 in. onto a split spoon sample and the number of blows required to penetrate 1 ft. is counted. SPT value will be higher in hard soils while it is low in soft clays and loose sand.

1.3.4 Measurement of cohesion

Cohesion of soil is measured by obtaining a Shelby tube sample and conducting a laboratory unconfined compression test.

1.4 Origin of a project

Civil engineering projects originate when a company or a person requires a new facility. A company wishing to construct a new facility is known as the owner. The owner of the new proposed facility would consult an architectural firm to develop an architectural design. The architectural firm would lay out room locations, conference halls, rest rooms, heating and cooling units, and all other necessary elements of a building, as specified by the owner.

Subsequent to the architectural design, structural engineers and geotechnical engineers would enter the project team. Geotechnical engineers would develop the foundation elements, while the structural engineers would design the structure of the building. Usually loading on columns will be provided by structural engineers (Fig. 1.8).

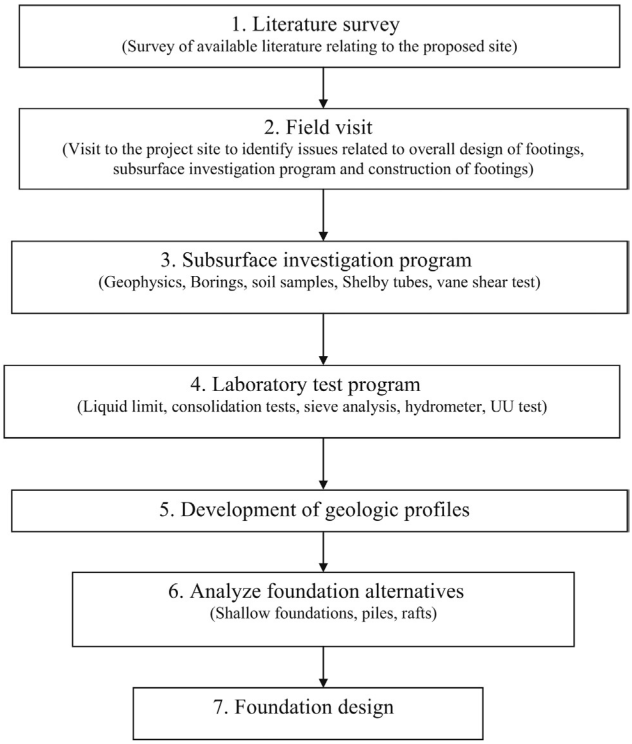


Figure 1.9 Site investigation program.

1.4.1 Geotechnical investigation procedures

After receiving information regarding the project, a geotechnical engineer should gather the necessary information for the foundation design. Usually, the geotechnical engineers would start with a literature survey.

After conducting a literature survey, he or she would make a field visit followed by the subsurface investigation program (Fig. 1.9).

1.4.2 Literature survey

The geotechnical engineer's first step is to conduct a literature survey. There are many sources available to obtain information regarding topography, subsurface soil conditions, geologic formations, and groundwater conditions. Sources for literature survey are local libraries, the Internet, local universities, and national agencies. National agencies usually conduct geological surveys, hydrogeological surveys, and topographic surveys. These surveys usually provide very important information to the geotechnical engineer. Topographic surveys will be useful in identifying depressed regions, streams, marshlands, manmade fill areas, organic soils, and roads. Depressed regions may indicate weak bedrock or settling soil conditions. Construction in marsh

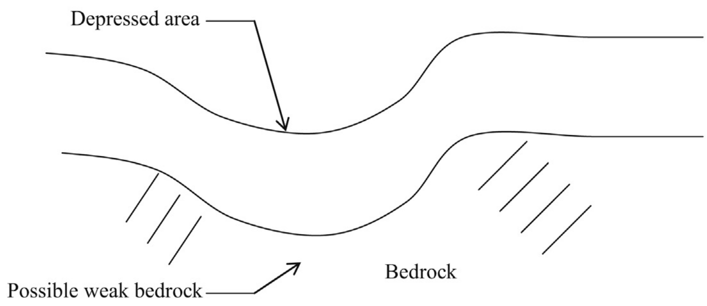


Figure 1.10 Depressed area.

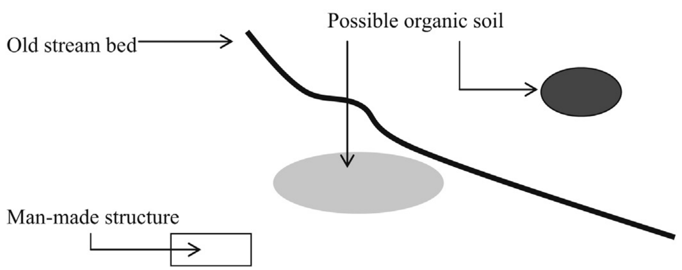


Figure 1.11 Important items in an aerial photograph.

areas will be costly. Roads, streams, utilities, and manmade fills will interfere with the subsurface investigation program or the construction (Fig. 1.10).

1.4.2.1 Adjacent property owners

If there are buildings in the vicinity of the proposed building, the geotechnical engineer may be able to obtain site investigation studies conducted in the past.

1.4.2.2 Aerial surveys

Aerial surveys are done by various organizations for city planning, utility design and construction, traffic management, and disaster management studies. Geotechnical engineers should contact the relevant authorities to investigate whether they have any aerial maps in the vicinity of the proposed project site.

Aerial maps can be a very good source of preliminary information for the geotechnical engineer. Aerial surveys are expensive to conduct and only large-scale projects may have a budget for aerial photography (Fig. 1.11).

Dark patches may indicate organic soil conditions, different types of soil, contaminated soil, low drainage areas, fill areas, or any other oddity that needs the attention of the geotechnical engineer. Darker than usual lines may indicate old stream beds or drainage paths or fill areas for utilities. Such abnormalities can be easily identified from an aerial survey map.

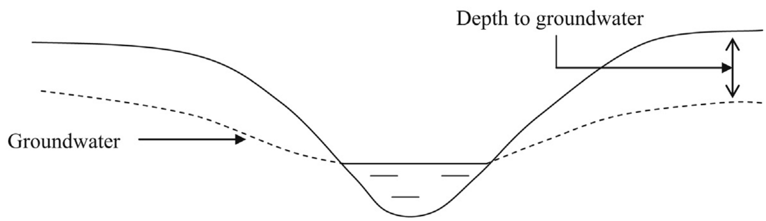


Figure 1.12 Groundwater level near a stream.

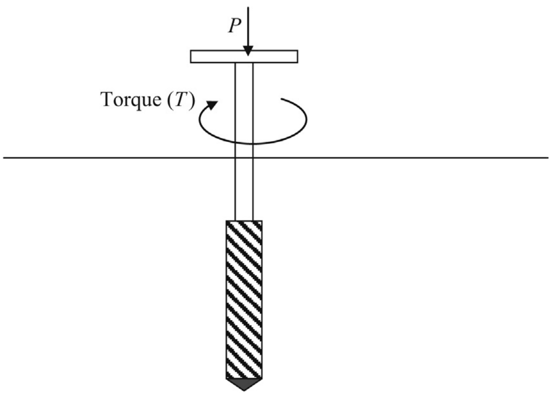


Figure 1.13 Hand auger.

1.4.3 Field visit

After conducting a literature survey, the geotechnical engineer should make a field visit. Field visits would provide information regarding surface topography, unsuitable areas, slopes, hillocks, nearby streams, soft grounds, fill areas, potential contaminated locations, existing utilities, and possible obstructions for site investigation activities. The geotechnical engineer should bring a hand auger to the site so that he or she could observe the soil a few feet below the surface (Fig. 1.12).

Nearby streams could provide excellent information regarding depth to groundwater.

1.4.3.1 Hand augering

Hand augers can be used to obtain soil samples to a depth of approximately 6 ft. depending upon the soil conditions. Downward pressure (P) and torque (T) are applied to the hand auger. Due to the torque and the downward pressure, the hand auger would penetrate the ground. The process stops when human strength is no longer capable of generating enough torque or pressure (Fig. 1.13).

1.4.3.2 Sloping ground

Steep slopes in a site escalate the cost of construction because compacted fill is required. Such areas need to be noted for further investigation (Fig. 1.14).

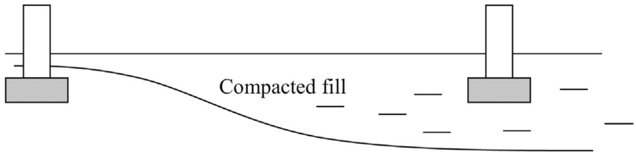


Figure 1.14 Sloping ground.

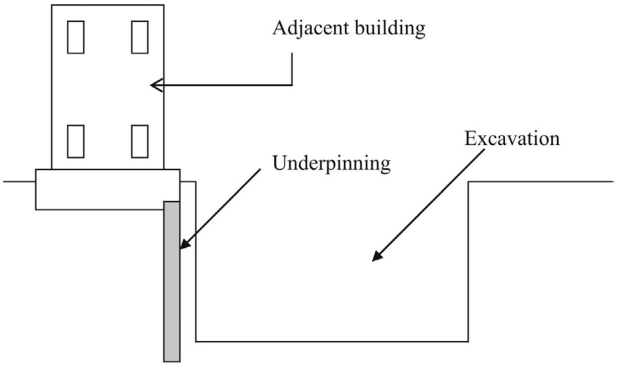


Figure 1.15 Underpinning of nearby building for an excavation.

1.4.3.3 Nearby structures

Nearby structures could pose many problems for proposed projects. It is always good to identify these issues at the very beginning of a project.

Distance to nearby buildings, schools, hospitals, and apartment complexes should be noted. Pile driving may not be feasible if there is a hospital or school close to the proposed site. In such situations, jacking of piles can be used to avoid noise.

If the proposed building has a basement, underpinning of nearby buildings may be necessary (Fig. 1.15).

Other methods such as secant pile walls or heavy bracing can also be used to stabilize the excavation.

Shallow foundations of a new building could induce negative skin friction on the pile foundation of nearby buildings. If compressible soil is present in a site, the new building may induce consolidation. Consolidation of a clay layer may generate negative skin friction in piles of nearby structures (Fig. 1.16).

1.4.3.4 Contaminated soils

Soil contamination is a very common problem in many urban sites. Contaminated soil will increase the cost of a project or in some cases could even kill a project entirely. Identifying contaminated soil areas at early stages of a project is desirable.

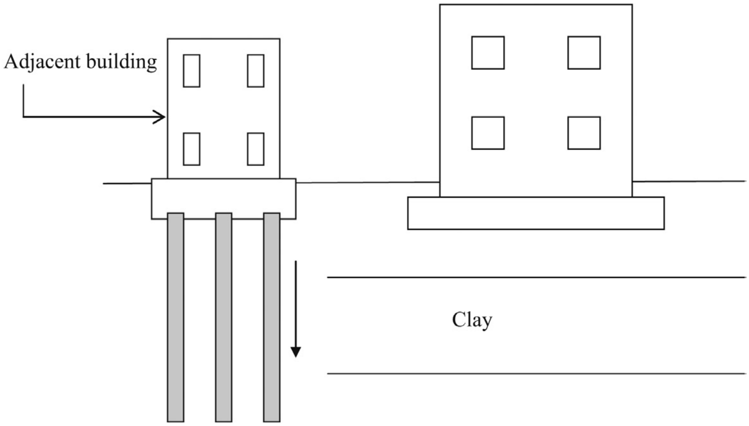


Figure 1.16 Negative skin friction in piles due to new construction.

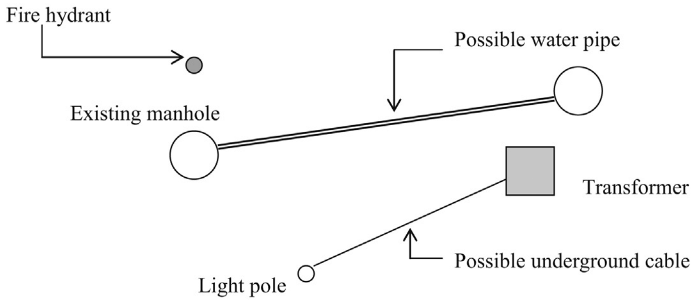


Figure 1.17 Observation of utilities.

1.4.3.5 Underground utilities

It is a common occurrence for drilling crews to accidentally puncture underground power cables or gas lines. Early identification of existing utilities is important. Electrical poles, electrical manhole locations, gas lines, and water lines should be noted so that drilling can be done without breaking any utilities.

Existing utilities may have to be relocated or undisturbed during construction.

Existing manholes may indicate drainpipe locations (Fig. 1.17).

1.4.3.6 Overhead power lines

Drill rigs need to keep a safe distance from overhead power lines during the site investigation phase. Overhead power line locations need to be noted during the field visit.

1.4.3.7 Man-made fill areas

Most urban sites are affected by human activities. Man-made fills may contain soils, bricks, and various types of debris. It is possible to compact some man-made fills so that they could be used for foundations. This is not feasible when the fill material contains compressible soils, tires, or rubber. Fill areas need to be further investigated during the subsurface investigation phase of the project.

1.4.3.8 Field visit checklist

A geotechnical engineer needs to pay attention to issues during the field visit with regard to the following:

- overall design of foundations
- obstructions for the boring program
  (overhead power lines, marsh areas, slopes, poor access may create obstacles for drill rigs)
- issues relating to construction of foundations
  (high groundwater table, access, existing utilities)
- identification of possible man-made fill areas
- nearby structures (hospitals, schools, court houses, etc.)

The geotechnical engineer's checklist for the field visit is shown in Table 1.1.

1.5 Pile foundations versus shallow foundations

Pile foundations are much more expensive than shallow foundations. Hence, all effort is made to avoid piles (Fig. 1.18).

Shallow foundations (also known as shallow footings) are possible only if the bearing soil is present at a shallow depth. Soils that are capable of supporting a foundation are known as bearing soils. Bearing soil can be either sandy or clayey. Dense sand and stiff clay will be able to support shallow footings. Densely packed man-made fill also can be used to construct shallow foundations. Loose sand, soft clay, organic material, peat, and loose man-made fill may not be suitable for shallow foundations.

1.5.1 Soil modification

Soil modification is the process of modifying unsuitable soil to bearing soil. Some of the soil modification methods are as follows:

- compacting with a roller
- dynamic compaction (dropping a heavy load from a height)
- pressure grouting (insert grout into the soil fabric)

There are other methods that have not been mentioned. Some of these methods could be very expensive. For instance pressure grouting is an expensive process. In such cases, piles may be cheaper.

Table 1.1 Field visit checklist

Items	Impacts on site investigation	Impacts on construction	Cost impacts
Sloping ground	May create difficulties for drill rigs	Maneuverability of construction equipment	Due to cut and fill activities
Small hills	May create difficulties for drill rigs	Maneuverability of construction equipment	Due to cut and fill activities
Nearby streams	Groundwater monitoring wells may be necessary	High groundwater may impact deep excavations (pumping)	Due to pumping activities
Overhead power lines	Drill rigs have to stay away from overhead power lines	Construction equipment have to keep a safe distance	Possible impact on cost
Underground utilities (existing)	Drilling near utilities should be done with caution	Construction work	Relocation of utilities
Areas with soft soils	More attention should be paid to these areas during subsurface investigation phase	Possible impact	Possible impact on cost
Contaminated soil	Extent of contamination need to be identified	Severe impacts on construction activities	Severe impact on cost
Man-made fill areas	Broken concrete or wood may pose problems during the boring program	Unknown fill material generally not suitable for construction work	Possible impact on cost
Nearby structures		Due to nearby hospitals and schools some construction methods may not be feasible such as pile driving Excavations for the proposed building could cause problems to existing structures Shallow foundations of new structures may induce negative skin friction in piles in nearby buildings	Possible impact on cost

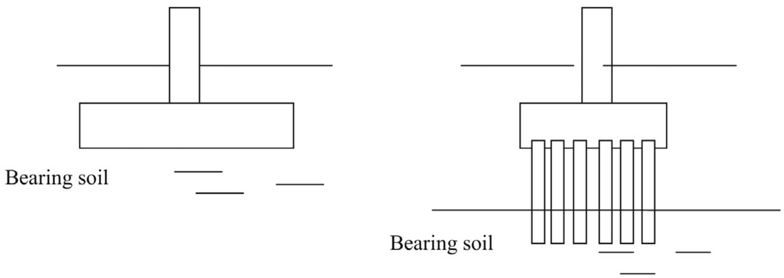


Figure 1.18 Shallow foundation and a pile foundation.

1.6 Subsurface investigation phase

The soil strength characteristics of the subsurface are obtained through a drilling program. In a nutshell, the geotechnical engineer needs the following information for foundation design work:

- soil strata identification (sand, clay, silt, etc.)
- depth and thickness of soil strata
- cohesion and friction angle (two parameters responsible for soil strength)
- depth to groundwater

1.6.1 Soil strata identification

Subsurface soil strata information obtained using drilling.

The most common drilling techniques are as follows:

- augering
- mudrotary drilling

1.6.2 Augering

In the case of augering, the ground is penetrated using augers attached to a rig. The rig applies a torque and a downward pressure to augers. The same principle as in hand augers, is used for penetrating the ground (Fig. 1.19, Plate 1.1).

1.6.3 Mud rotary drilling

In the case of mud rotary drilling, a drill bit, known as roller bit, is used for the penetration. Water is used to keep the roller bit cool so that it will not overheat and stop functioning. Usually drillers mix bentonite slurry (also known as drilling mud) to the water, to thicken the water. The main purpose of the bentonite slurry (also known as mud) is to keep the sidewalls from collapsing (Fig. 1.20, Plates 1.2 and 1.3).

Drilling mud goes through the rod and the roller bit and comes out from the bottom removing cuttings. The mud is captured in a basin and recirculated.

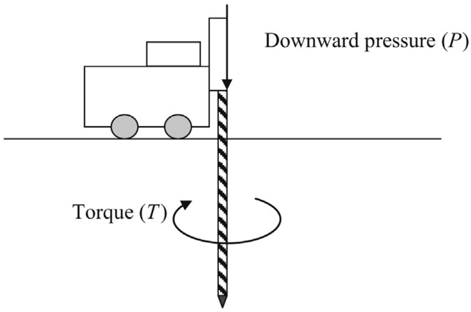


Figure 1.19 Augering.

Plate 1.1 Augers.

1.6.4 Boring program

The number of borings that need to be constructed may sometimes be regulated by local codes. For example, the New York City building code requires one boring per 2500 ft.^{2}. It is important to conduct borings as close as possible to column locations and strip footing locations. In some cases this may not be feasible.

Typically borings are constructed 10 ft. below the bottom level of the foundation.

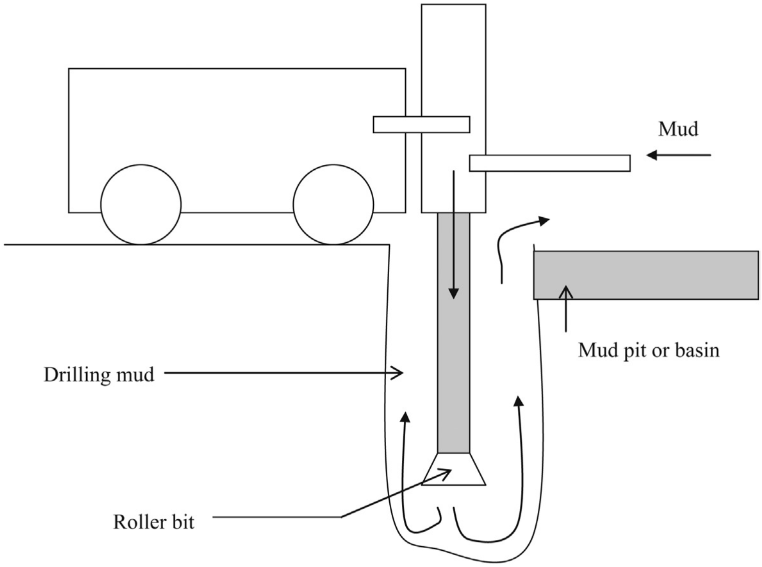


Figure 1.20 Mud rotary drilling.

Plate 1.2 Mud rotary drill rig.

Plate 1.3 Roller bit (Roller bit cut through soil. Water is injected from the hole to keep the roller bit from overheating).

1.6.5 Test pits

In some situations, test pits would be more advantageous than borings. Test pits can provide information down to 15 ft. below the surface. Unlike borings, soil can be visually observed from the sides of the test pit.

1.6.6 Hand digging prior to drilling

Damage of utilities should be avoided during the boring program. Most utilities are rarely deeper than 6 ft. Hand digging the first 6 ft. prior to drilling boreholes is found to be an effective way to avoid damaging utilities. During excavation activities, the backhoe operator is advised to be aware of utilities. The operator should check for fill materials, since in many instances utilities are backfilled with select fill material. It is advisable to be cautious since there could be situations where utilities are buried with the same surrounding soil. In such cases it is a good idea to have a second person present exclusively to watch the backhoe operation.

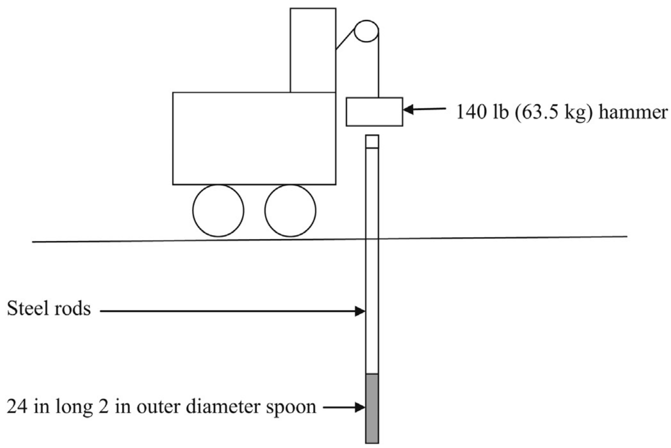


Figure 1.21 SPT hammer mechanism.

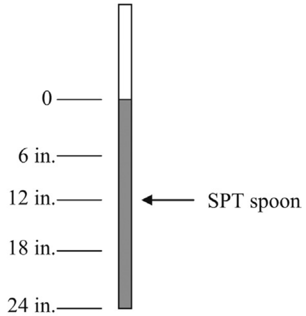


Figure 1.22 SPT spoon.

1.7 Geotechnical field tests

1.7.1 SPT (N) value

During the construction of borings, SPT (N) values of soils are obtained. SPT (N) value provides information regarding the soil strength. SPT (N) value in sandy soils indicates the friction angle in sandy soils and in clay soils indicates the stiffness of the clay stratum.

SPT (N) value is obtained by dropping a 140 lb (63.5 kg) hammer over a distance of 30 in. (760 mm). The drill rods are attached to a hollow cylinder. This hollow cylinder is known as the spoon. The hollow cylinder is 2 in. (51 mm) outer diameter and has a length of 24 in. (Fig. 1.21).

When the hammer is dropped, the spoon would go down. The number of blows required to drive the spoon 24 in. is recorded. Blows are recorded for every 6 in. (Fig. 1.22).

Table 1.2 Correlations with SPT (*N*) value (Bowles, 1988)

SPT (*N*)	Particle sizes	Consistencies	Relative densities	Friction angles (ϕ')	γ_{wet} (pcf)
1–2	Fine	Very loose	0.15	26–28	70–100
2–3	Medium			27–28	
3–6	Coarse			28–30	
3–6	Fine	Loose	0.35	28–30	90–115
4–7	Medium			30–32	
5–9	Coarse			30–34	
7–15	Fine	Medium	0.65	30–34	110–130
8–20	Medium			32–36	
10–25	Coarse			33–40	
16–30	Fine	Dense	0.85	33–28	110–140
21–40	Medium			36–42	
26–45	Coarse			40–50	

As mentioned, blows required for each 6 in. are recorded. SPT value is the addition of blows from 6 to 12 in. + 12 to 18 in.

Let us assume some hypothetical blows attained in a certain site to be:

0–6 in. = 12 blows
6–12 in. = 33 blows
12–18 in. = 41 blows
18–24 in. = 19 blows

Then the SPT value is 33 + 41 = 74.

1.8 SPT (N) and friction angle

Relative density and friction angle are important parameters in the design of piles in sandy soils.

The correlations with SPT (*N*) value are shown in Table 1.2.

1.9 Field tests

1.9.1 Pocket penetrometer

Pocket penetrometers can be used to obtain the stiffness of clay samples. The pocket penetrometer is pressed into the soil sample and the reading is recorded. The reading would indicate the cohesion of the clay sample (Fig. 1.23).

1.9.2 Vane shear test

Vane shear tests are conducted to obtain the cohesion (*C*) value of a clay layer. An apparatus consisting of vanes are inserted into the clay layer and rotated. The torque of

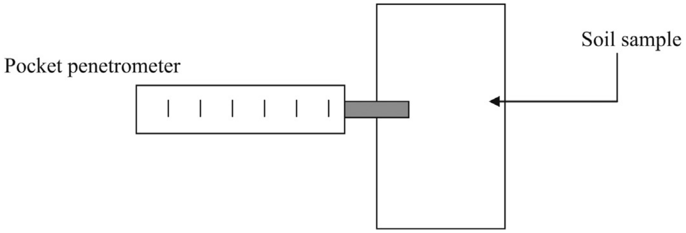


Figure 1.23 Pocket penetrometer.

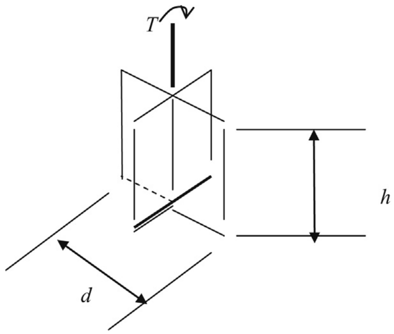


Figure 1.24 Vane shear apparatus.

the vane is measured during the process. Soils with high cohesion values register high torques (Fig. 1.24).

The vane shear test procedure is as follows:

1. a drill hole is made with a regular drill rig.
2. the vane shear apparatus is inserted into the clay.
3. the vane is rotated and the torque is measured.
4. the torque would gradually increase and reach a maximum. The maximum torque achieved is recorded.
5. at failure, the torque would reduce and reach a constant value. This value refers to the remolded shear strength (Fig. 1.25).

The cohesion of clay is given by

$$T = C \times \pi \times \left(\frac{d^2 h}{2 + d^3} 6 \right).$$

Here, T, torque (measured); C, cohesion of the clay layer; d, width of vanes; and h, height of vanes.

The cohesion of the clay layer is obtained by using the maximum torque. Remolded cohesion was obtained by using the torque at failure.

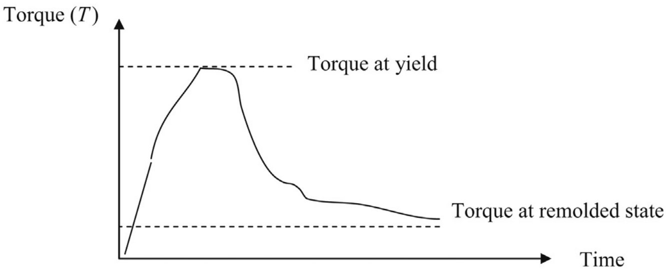


Figure 1.25 Torque versus time curve.

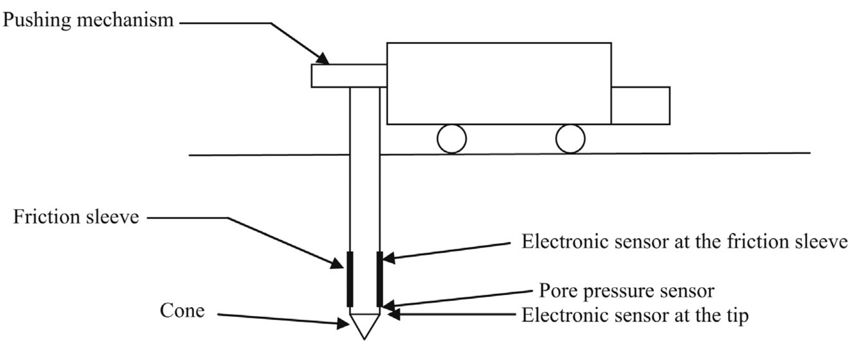


Figure 1.26 CPT testing.

1.9.3 Cone penetration testing

Cone penetration tests (CPTs) are done to identify the soil type. In this test, a cone is pushed into the ground. In the case of SPT tests, a spoon is driven with a hammer. In CPT testing, the cone is pushed instead of being driven (Fig. 1.26).

Fig. 1.26 shows the following items:

- Vehicle: vehicle is required to provide power to the pushing mechanism.
- Pushing mechanism: hydraulic mechanism is used to push the cone into the ground.
- Cone at the tip: cone at the tip measures the tip resistance. There is an electronic sensor just above the tip.
- Friction sleeve: friction sleeve measures the friction in the shaft. There is a sensor that measures the friction in the sleeve.
- Pore pressure sensor: this sensor would measure the pore pressure in the soil.

The cone used is 37.5 mm in diameter and has an angle of 60 degrees. The friction sleeve is 133.7 mm. The surface area of the skin friction sleeve is 15,000 mm^2 (Fig. 1.27).

The perimeter area of the skin friction sleeve = $\pi D \times$ length = $\pi \times 35.7 \times 133.7$ = 15,000 mm^2.

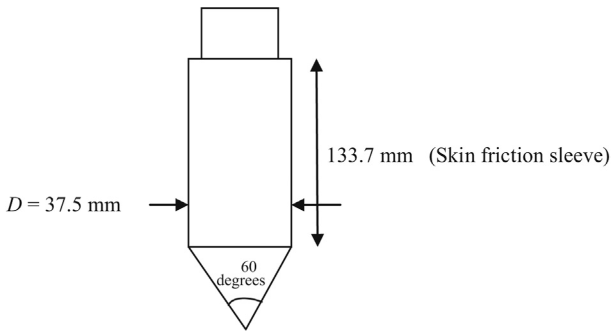


Figure 1.27 CPT apparatus.

Measurements obtained are as follows:

- depth to the cone
- tip resistance
- skin friction in the sleeve
- pore pressure

Resistance to cone penetration comes from two processes:

1. resistance at the tip
2. resistance in the sleeve due to skin friction

Clay soils have high skin friction and sandy soils have high tip resistance. This is a general statement and values are dependent on individual soils. Very stiff clay may have higher tip resistance than a loose sand layer. Also, clay soils will have high excess pore pressure compared to sandy soils.

1.9.4 Friction ratio

Friction ratio is the ratio between skin friction versus tip resistance expressed as a percentage.

$$\text{Friction ratio}\,(f_R) = \frac{f}{Q_u \times 100}.$$

Here, f, skin friction in the sleeve measured using electronic sensors (tsf); and Q_u, tip resistance measured using electronic sensors (tsf).

The following general statements can be said of CPT data:

1. Gravelly sand. very low friction ratio and very high tip resistance.
2. Sand. low friction ratio moderate tip resistance.
3. Sandy silt or silty sand. moderate friction ratio and moderate tip resistance.
4. Clays. high friction ratio and low tip resistance.
5. Peat and organic clays. very low friction ratio and very low tip resistance.

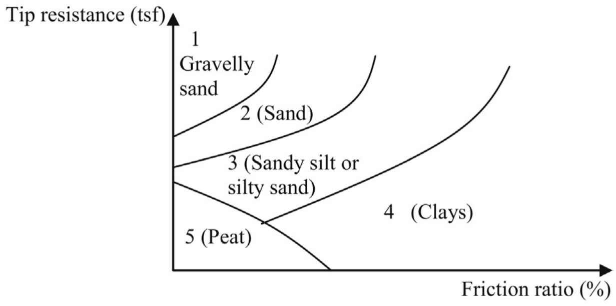


Figure 1.28 Tip resistance vs. friction.

The mentioned soil types (1–5) can be represented in a graph as shown in Fig. 1.28.

Note that the representation is very general in nature and should not be used without supporting data. It is very possible that stiff clay could be area 3 or 2. Generally, a friction ratio of 1–2% is considered to be very low and 10–15% is considered to be very high. Also, a tip resistance of 1 to 1–2 tsf is in the lower category and 8–10 in the higher category.

Practice problem

The following CPT data were obtained from a site. Predict the soil types for each layer (Fig. 1.29).

Solution

Layer 1: tip resistance is very low and friction ratio very high. This layer could be identified as a clay layer.

Layer 2: tip resistance is moderate and friction ratio also moderate. This layer could be identified as a sandy silt or silty sand layer.

Layer 3: tip resistance is low and friction ratio is high. This layer could be identified as a clay layer.

Layer 4: tip resistance is moderate and friction ratio is low. This layer can be identified in the silty sand category.

1.10 Pressure meter testing

In this test, a hole is predrilled. Then a pressure meter is inserted and inflated. The force required to inflate the pressuremeter and the strain are recorded (Fig. 1.30).

When the pressuremeter is inflated, stress in the pressure meter and the strain in soil are recoded. Two methods are typically used.

1.10.1 The equal pressure increment method

In this method, pressure in the pressuremeter is increased at equal intervals. If the pressure of the pressuremeter is 10 tsf and the increment is 2 tsf, the following pressure values will be maintained: 10, 12, 14, 16, and 18 tsf.

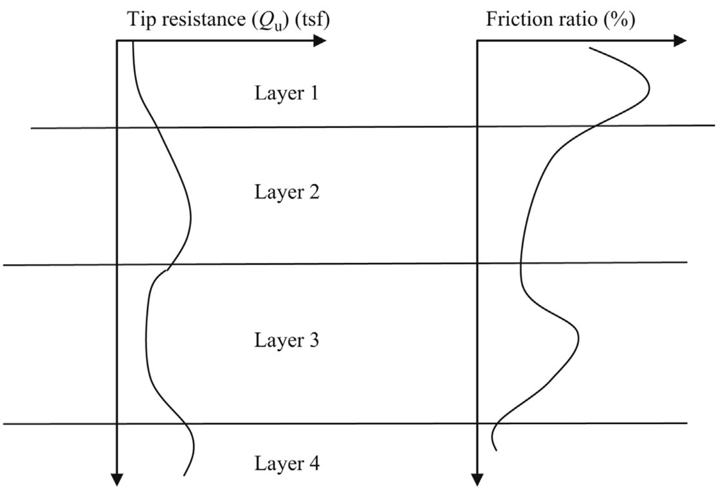


Figure 1.29 CPT data in layered soil.

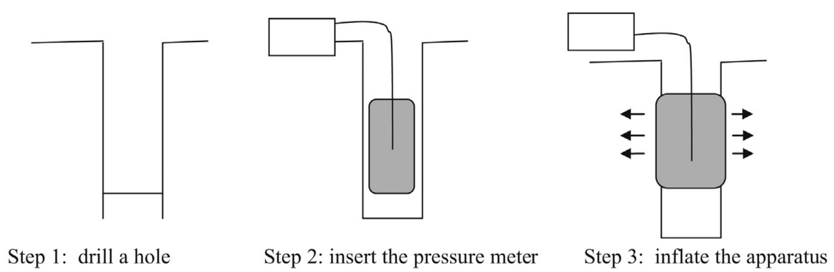


Figure 1.30 Pressuremeter test procedure.

1.10.2 The equal volume increment method

In this method, the volume of the pressure meter is increased at equal intervals. If the volume of the pressure meter is 10 in.^{3} and the increment is 2 in.^{3}, the following volume values will be maintained: 10, 12, 14, 16, and 18 in.^{3}

A typical graph obtained from a pressuremeter test is shown in Fig. 1.31.

When the volume of the pressuremeter is increased, pressure inside the pressuremeter will go up. Initially when the pressuremeter starts to expand, there is a slight gap between the soil and the pressuremeter. Also, the soil at the wall of the borehole is disturbed. The initial region of the graph is not of much use. After this, an approximately straight line graph is seen. This region is named nearly elastic or pseudoelastic. Soils are rarely fully elastic. After this, the plastic region is reached. Failure load is known as p_L.

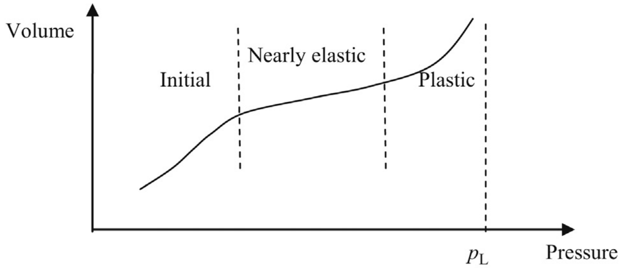


Figure 1.31 Pressuremeter graph.

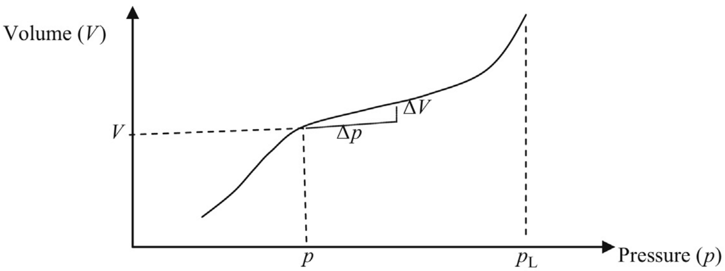


Figure 1.32 Pressure meter graph analysis.

The following equation is used to calculate a parameter known as pressuremeter modulus:

$$E_{pm} = (1+\nu)2V\left(\frac{\Delta p}{\Delta V}\right).$$

Here, E_{pm}, pressuremeter modulus; ν, Poisson's ratio; V, volume of the pressuremeter at a pressure of p; Δp, pressure increment; ΔV, volume increment; and p_L, failure pressure.

See Fig. 1.32 for further explanation.

In Fig. 1.32, $\Delta p/\Delta V$ is the inverse gradient of the graph. Once the graph is obtained, E_{pm} can be computed.

What is the purpose of computing E_{pm}?

There are correlations between E_{pm} and various soil types. Hence, soils can be identified using pressuremeter test data as shown in Table 1.3.

Table 1.3 is valid only for overconsolidated soils. Most soils are overconsolidated. Normally consolidated soils are extremely rare.

Table 1.3 Pressuremeter table (obtained from Terzaghi, Peck, and Mesri)

	E_{pm}/p_L
Clay	>16
Silt	>14
Sand	>12
Sand and gravel	>10

Practice problem

Pressuremeter test was done and the following data were obtained. At a pressure of 4000 psf, volume of the pressuremeter was measured to be 13 in.^{3} Pressure is slightly increased by 1000 psf and the volume increase was measured to be 0.25 in.^{3} Poisson's ratio of soil is known to be 0.3. Failure stress (p_L) is obtained from the graph and found to be 8500 psf. How would you characterize the soil?

Solution

Step 1: Write down the known values.

$$p = 4000\text{ psf}; \quad V = 13\text{ in.}^3$$
$$\Delta p = 1000\text{ psf}; \quad \Delta V = 0.4\text{ in.}^3, \quad p_L = 8500\text{ psf}$$

$$E_{pm} = (1+\nu)2V\left(\frac{\Delta p}{\Delta V}\right)$$
$$E_{pm} = (1+0.3)2\times 13\left(\frac{1,000}{0.25}\right) = 135,200$$

$$\left(\frac{E_{pm}}{p_L}\right) = \left(\frac{135,200}{8,500}\right) = 15.9$$

The value 15.9 is greater than 14. Hence the soil is most likely silt.
Note: E_{pm}/p_L is unitless. Make sure the same units are maintained for Δp and p_L.

1.10.3 SPT–CPT correlations

In the United States, SPT is used extensively. On the other hand, CPT is popular in Europe.

Europeans have developed many geotechnical design methods using CPT data.

The following correlation between SPT and CPT can be used to convert CPT values to SPT number (Table 1.4). Q_c, CPT value measured in bars (1 bar = 100 kPa); N, SPT value; D_{50}, size of the sieve that would pass 50% of the soil.

Table 1.4 **SPT–CPT correlations for clays and sands (Robertson et al., 1983)**

Soil types	Mean grain size (D_{50}) (mm)	Q_c/N
Clay	0.001	1
Silty clay	0.005	1.7
Clayey silt	0.01	2.1
Sandy silt	0.05	3.0
Silty sand	0.10	4.0
Sand	0.5	5.7
	1.0	7.0

Example

SPT tests were done on sandy silt with a D_{50} value of 0.05 mm. Average SPT (N) value for this soil is 12. Find the CPT value.

Solution

From Table 1.4, for sandy silt with a D_{50} value of 0.05 mm,

$$\left(\frac{Q_c}{N}\right) = 3.0$$

$N = 12$; hence, $Q_c = 3 \times 12 = 36$ bars = 3600 kPa.

1.10.4 Standard CPT device

The standard cone has a base area of 10 cm^2 and an apex angle of 60 degrees.

1.10.5 Standard SPT device

A Donut hammer with a weight of 140 lb and a drop of 30 in. Donut hammers have an efficiency of 50–60%.

1.10.6 Dilatometer testing

Dr Silvano Marchetti in 1975 developed the dilatometer test apparatus. The dilatometer blade has a cross-sectional area of about 14 cm^2 and can be pushed into soil with a rig. The dilatometer blade has a membrane attached to it. Once the dilatometer blade is in place where the test has to be done, the membrane is inflated. When the membrane is inflated, soil would be stressed. Stronger soils would obviously provide a higher resistance (Fig. 1.33).

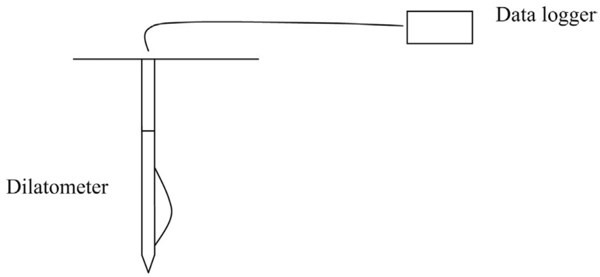


Figure 1.33 Dilatometer.

Tests can be successfully performed in all penetrable soils, including clay, silt, and sand. The dilatometer is not good for use in gravelly soil.

References

Bowles, J., 1988. Foundation Analysis and Design. McGraw Hill Book Company, NY.
Robertson, P.K., et al., 1983. SPT-CPT correlations, ASCE Geotechnical Engineering Journal, Nov., 109 (11), pp.1449–1459.

Geophysical methods

2

Geophysical methods can be used to obtain soil parameters. Let us look at an example. As you know bats can locate objects using sound waves. The bat would send a sound wave and wait to hear the reflection or the echo. From the echo, the bat can identify a frog (Fig. 2.1).

Geophysical methods are based on the same concept. In the case of ground-penetrating radar (GPR), a radar signal is sent and the reflected signal is recorded. In the case of seismic methods, blows are given to the ground surface with a hammer and the reflected seismic waves are collected.

2.1 Ground-penetrating radar methods

In GPR methods, a radar signal is sent and the reflected signal is recorded. The returned signal is analyzed to obtain the soil profile. Radar is widely used in aircraft identification. Same radar can be used to identify soil profiles. Radar is a radio wave. Radio waves belong to the family of electromagnetic waves. Light, X-rays, and radar belong to the electromagnetic wave family.

2.1.1 General methodology

Radar travels at the speed of light. Light would travel approximately 1 ft./ns. One nanosecond is equal to 10^{-9} s. Let us assume that a radar signal bounces back after 10 ns; we can assume that there is an obstruction 5 ft. below the surface (Fig. 2.2).

$$\text{Velocity of radar} = 2 \times \frac{D}{t}.$$

D, depth to the object; and t, travel time.

Velocity of radar in soil is known. Travel time (t) can be measured. Hence, depth can be calculated. In the given equation, depth is multiplied by 2 to account for the wave to reach a depth of D and come back again. Travel distance of the waveform is $2 \times D$.

Let us look at Fig. 2.3. As mentioned previously, the depth of an object can be computed using the travel time. Fig. 2.3 shows two boundaries (three layers) of different soils. It may not be feasible to tell the exact soil type using GPR data. Hence, borings should be conducted. GPR could be used to properly identify the soil strata boundaries.

Fig. 2.3 shows a GPR survey. GPR lines will curve near a pipe.

Pile Design and Construction Rules of Thumb

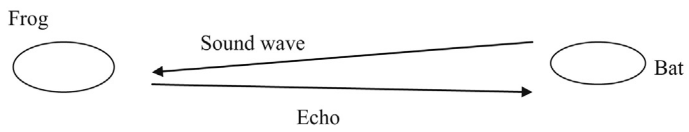


Figure 2.1 Bat and a frog.

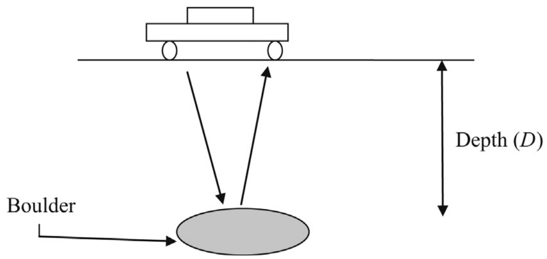


Figure 2.2 Ground-penetrating radar.

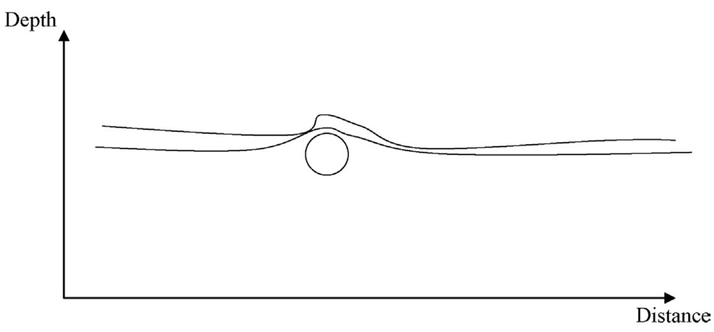


Figure 2.3 GPR example.

2.1.2 Single borehole GPR

In this method, a borehole is drilled. The radar transmitter and the receiver are set at different depths and data are collected (Fig. 2.4).

2.1.3 Procedure

1. Place the radar transmitter and the receiver at any depth. Record the data.
2. Change the depth and repeat the process.

Many different GPR data sets can be obtained in this way. Also, the transmitter and receiver can be placed at different depths.

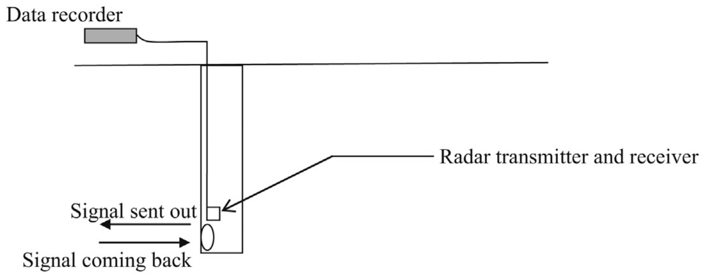


Figure 2.4 GPR in single borehole.

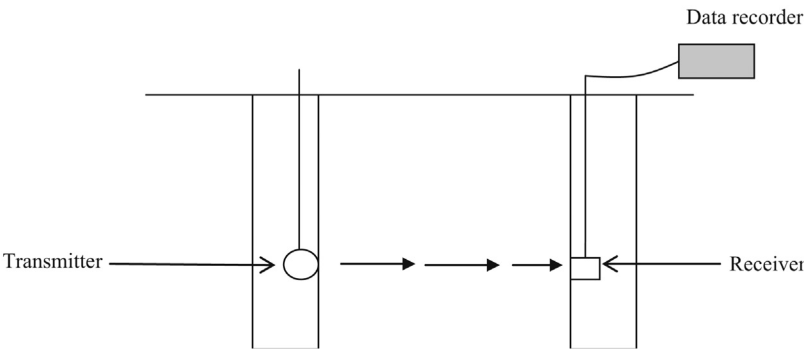


Figure 2.5 Cross-hole method in GPR.

2.1.4 Cross-hole GPR

In this method, two boreholes are needed. The transmitter is placed at one borehole and the receiver is placed at the other borehole (Fig. 2.5).

In Fig. 2.5, the radar transmitter is shown in one borehole. The transmitted waveform is picked up by a receiver placed in a second borehole. From the GPR data it is possible to ascertain the ground conditions between two boreholes.

2.2 Seismic method

The concept in the seismic method is similar to GPR. Instead of a radio signal, a seismic signal is sent out and whatever comes back is recorded. Seismic signal is produced by hammering a piece of wood placed on the ground with a hammer. The seismic signal would travel to seismic sensors. The seismic sensors are attached to a data logger (Fig. 2.6).

Fig. 2.6 shows a piece of wood placed on the ground and hammered. The seismic wave would travel to geophones or seismic sensors. Seismic sensors are attached to a data logger.

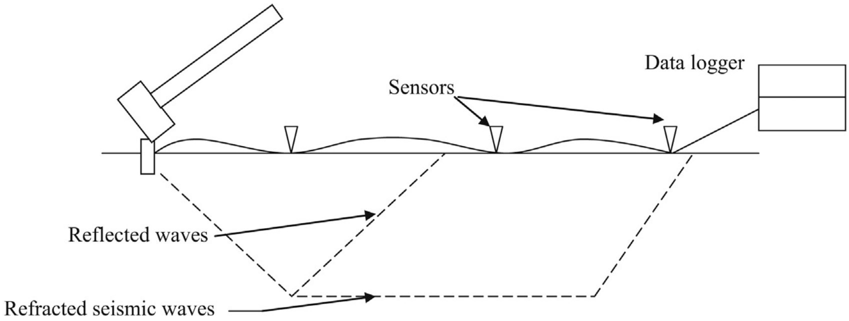


Figure 2.6 Seismic method.

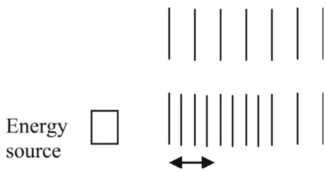


Figure 2.7 Seismic waves.

2.2.1 Reflected seismic waves versus refracted seismic waves

Unlike radar, seismic waves could have two pathways. Reflected path and refracted path.

2.2.2 Seismic P- and S-waves

P-waves are known as body waves. They travel faster than *S*-waves. In a *P*-wave, soil particles move in the same direction as the wave (Fig. 2.7).

Fig. 2.7 (top) shows soil particles prior to seismic energy. Fig. 2.7 (bottom) shows soil particles getting compressed due to seismic energy. The arrow shows the particle movement. The compression wave of soil particles would travel creating a *P*-wave.

2.2.2.1 S-Waves

In the case of *S*-waves, soil particles move perpendicular to the wave direction. *S*-waves are known as shear waves. Shear waves will not occur in air or in water (Fig. 2.8).

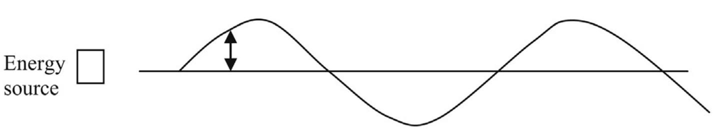


Figure 2.8 Seismic S-waves.

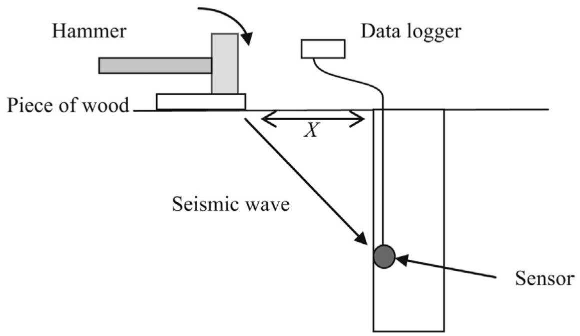


Figure 2.9 Down-hole seismic testing.

The arrow in Fig. 2.8 shows the particle movement. In an *S*-wave, the soil particles move perpendicular to the wave.

2.2.2.2 Surface waves

Surface waves travel along the surface as the name indicates. These waves are known as Rayleigh waves.

2.2.3 Down-hole seismic testing

In this method, a borehole is drilled. Seismic energy is applied at a known distance. Seismic sensors are placed inside the borehole. Data will provide soil strata information (Fig. 2.9).

After obtaining data at a given depth, another set of data is obtained by changing the depth to the sensor.

2.2.4 Cross-hole seismic testing

Seismic cross-hole testing is similar to GPR cross-hole testing (Fig. 2.10).

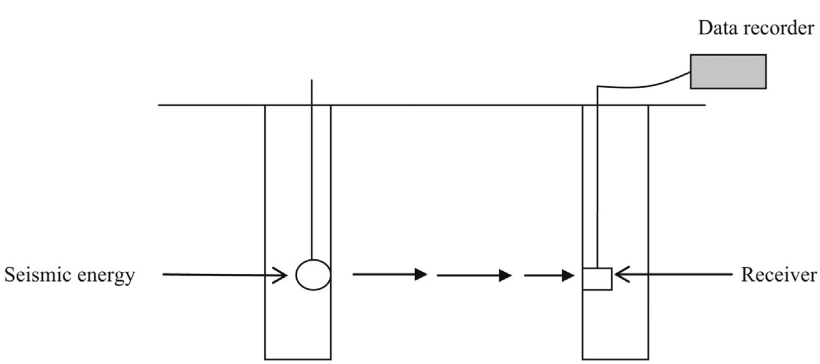


Figure 2.10 Cross-hole seismic testing.

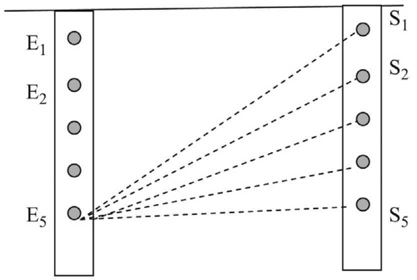


Figure 2.11 Multiple sensors.

Seismic energy is applied from one borehole. Generally, airburst or shock wave is applied. Seismic sensors (receivers) are placed in the other borehole. A new set of data can be obtained by changing the depths of the sensor and energy source (Fig. 2.11).

Cross-hole seismic testing is shown in Fig. 2.11. "E" here indicates seismic energy source, and "S" indicates geophones or sensors. In this case, a number of geophones are located along the depth of the borehole.

Groundwater

3

3.1 Introduction

Water existing in the atmosphere in clouds and air is known as meteoric water. Surface water could be fresh water or oceanic water. Groundwater is part of the water cycle. Groundwater seeps into streams and then gets evaporated. Evaporated water falls back to earth and seeps back to the ground (Fig. 3.1).

Man-made parking lots and cities prevent infiltration of water to the ground. Water falling on a big city would be transported to drains and then through storm pipes. It will be then taken to the ocean or large rivers. This would bypass groundwater (Fig. 3.2).

3.1.1 Magmatic water

When volcanoes erupt, magma flows. Water in magma gets released to the atmosphere and to the ground. Magmatic water can be identified by its mineral content.

3.1.2 Connate water

In a previous chapter, we discussed the formation of sedimentary rocks. During formation of sedimentary rocks, water gets trapped inside the rock. This water has not been in contact with the atmosphere for a long period of time.

3.1.3 Metamorphic water

Metamorphic rocks are formed due to high pressure and high temperature. During the formation of this rock, water gets trapped inside metamorphic rocks. The chemical composition of metamorphic water is different from that of connate water.

3.1.4 Juvenile water

Magmatic water, connate water, or metamorphic water newly entering the water cycle is known as juvenile water.

3.2 Vertical distribution of groundwater

The zone above the groundwater level is known as the vadose zone. The vadose zone is also known as the zone of aeration. The vadose zone is divided into soil–water zone, intermediate vadose zone, and capillary zone (Fig. 3.3).

Pile Design and Construction Rules of Thumb

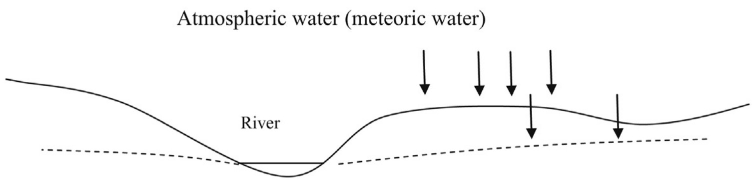


Figure 3.1 Water cycle.

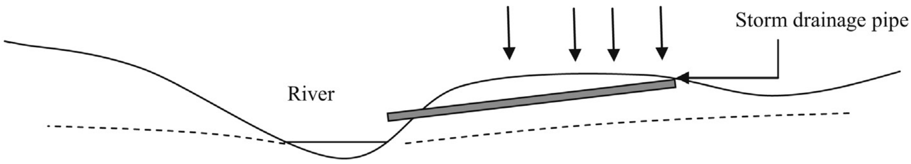


Figure 3.2 Storm drainage.

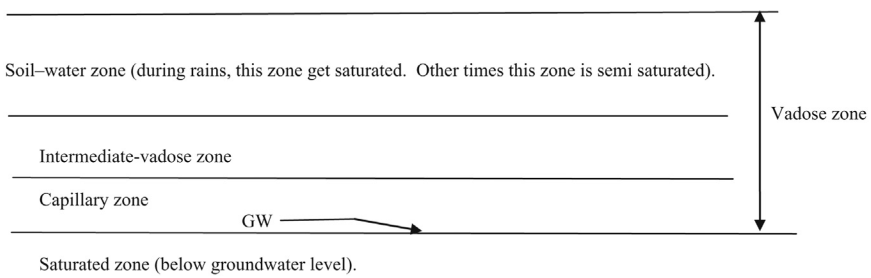


Figure 3.3 Groundwater zones.

3.2.1 Soil–water zone

This zone is exposed to the atmosphere. During rains this zone gets saturated. At other times, this zone is semisaturated.

3.2.2 Intermediate vadose zone

Water in this zone depends on soil type. Some soils would be able to store significant amounts of water.

3.2.3 Capillary zone

Due to capillary action, groundwater rises up. The amount of water in the capillary zone depends on the soil type.

3.3 Aquifers, aquicludes, aquifuges, and aquitards

3.3.1 Aquifer

Water-bearing soil and rock formations are known as aquifers. Aquifers are capable of absorbing water and also transmitting water. Typically sandy soils and sedimentary rock formations are considered to be aquifers. Aquifers are of two types. They are confined aquifers and unconfined aquifers (Fig. 3.4).

Confined aquifers are not open to the atmosphere. Confined aquifers sometimes may be under pressure. When a confined aquifer is under pressure, it is known as an *artesian aquifer*.

Fig. 3.4 shows water flowing to the surface without a pump. This is due to a confined aquifer under pressure.

3.3.2 Aquiclude

Aquicludes can absorb water but would not yield an appreciable quantity of water.

3.3.3 Aquitard

Aquitards as in the case of aquicludes, can absorb water but would not yield an appreciable quantity of water. But unlike aquicludes, aquitards can transmit water from adjacent aquifers. Apparently, there is a slight difference between aquicludes and aquitards. The following are the definitions given by the US Geological Survey (USGS):

- Aquiclude: A hydrogeologic unit, which although porous and capable of storing water, does not transmit it at rates sufficient to furnish an appreciable supply for a well or spring. See preferred term confining unit.
- Aquitard: A confining bed that retards but does not prevent the flow of water to or from an adjacent aquifer; a leaky confining bed. It does not readily yield water to wells or springs, but may serve as a storage unit for ground water (AGI, 1980). See preferred term confining unit.

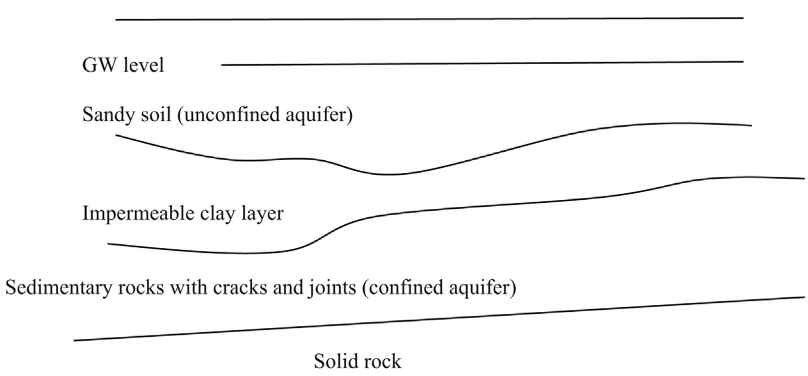


Figure 3.4 Aquifers.

- Confining unit: A hydrogeologic unit of impermeable or distinctly less permeable material bounding one or more aquifers and is a general term that replaces aquitard, aquifuge, aquiclude (AGI, 1980).

Although aquitards do not have much water in store, they can transmit from nearby aquifers. Aquicludes are unable to do so. Also, it seems that the USGS prefers to use the term ‘confining unit’ for aquicludes, aquitards, and aquifuges.

3.3.4 Aquifuges

Aquifuges are not capable of absorbing water. Hence, there is no water to transmit.

Piezometers are used to measure the pore water pressure in soil (Fig. 3.5).

This schematic diagram shows the main items of a piezometer. The tip of the piezometer is equipped with an electronic device known as a pressure transducer that can measure the water pressure. The piezometer tip is backfilled with filter sand. Bentonite seal is placed above the filter sand. On top of the bentonite seal, bentonite cement grout is placed. This is a much cheaper product than bentonite seal.

- Piezometer tip: the tip has an electronic device to measure the pore pressure.
- Filter sand: filter sand allows water to flow to the piezometer tip.
- Bentonite seal: bentonite seal stop water migrating from top aquifers.
- Bentonite cement grout: bentonite cement grout serves the same purpose as the bentonite seal. This is a much cheaper product.

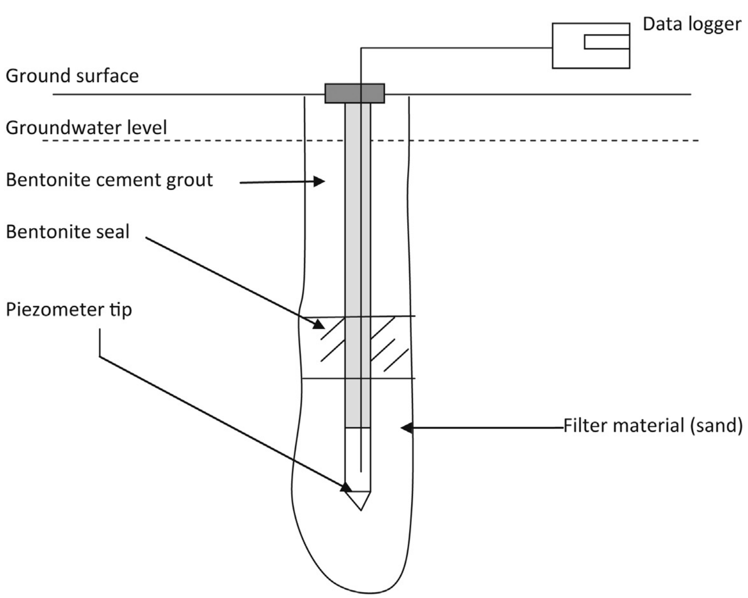


Figure 3.5 Piezometer.

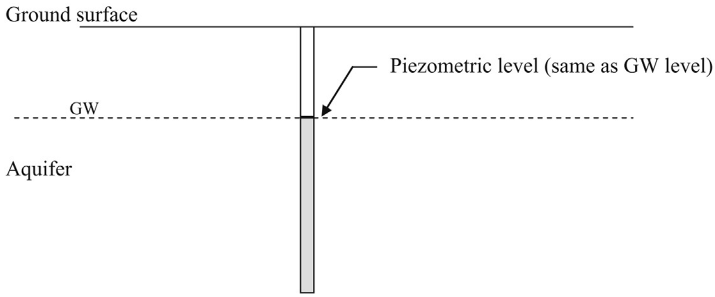


Figure 3.6 Piezometric level.

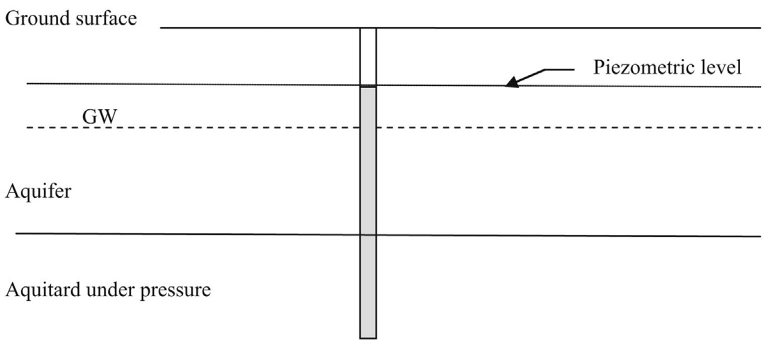


Figure 3.7 Aquitard under pressure.

3.3.5 *Piezometric surface versus groundwater level*

In some situations, the piezometric surface is not the same as the groundwater level. This happens in confined aquifers that are under pressure. Fig. 3.6 shows an aquifer. The groundwater level and piezometric level is the same in aquifers.

3.3.6 *Aquitard under pressure*

Fig. 3.7 shows an aquitard under pressure. In this case, the piezometric level will be higher than the groundwater level.

3.3.7 *Vertical upward groundwater flow*

Fig. 3.8 shows a number of soil layers and their piezometric level.

On looking at Fig. 3.8, one would see that the pressure head in soil layer 4 is higher than the pressure head in soil layer 3. Hence, water will flow from layer 4 to layer 3. Similarly, the pressure head in soil layer 3 is higher than the pressure head in soil layer 2. Hence, water will flow from layer 3 to 2. This is upward vertical flow.

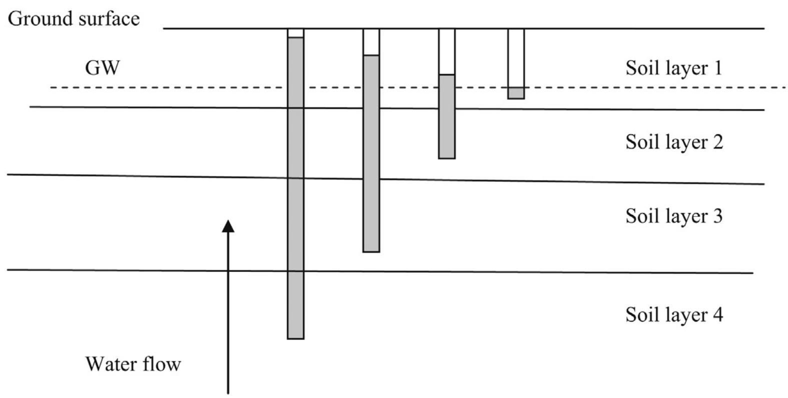


Figure 3.8 Vertical upward flow.

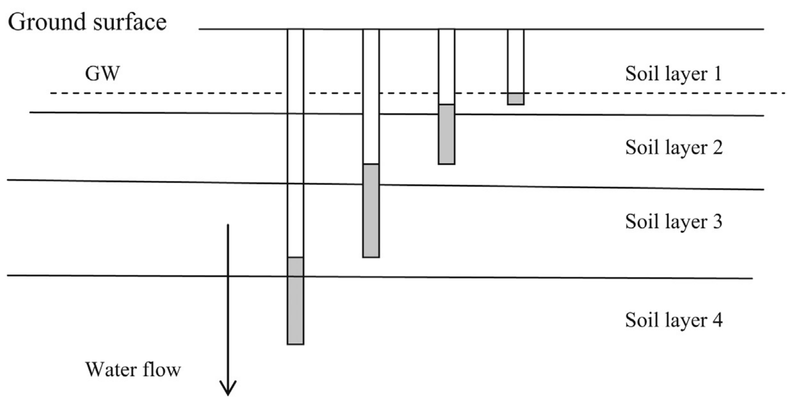


Figure 3.9 Vertical downward flow.

3.3.8 Vertical groundwater flow

Fig. 3.9 shows a number of soil layers and their piezometric level.

3.3.9 Monitoring wells

Monitoring wells are installed to obtain the groundwater elevation. Monitoring wells are typically constructed using PVC pipes (Fig. 3.10).

Slotted section of the PVC is known as the well screen and allows water to flow into the well. If there is no pressure, water level in the well indicates the groundwater level.

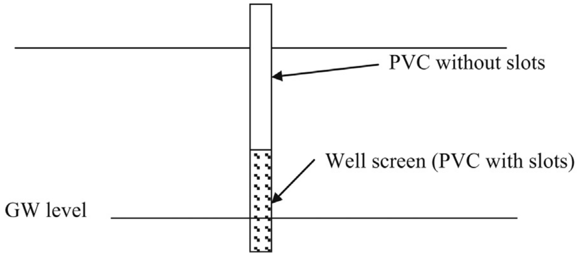


Figure 3.10 Groundwater monitoring well.

3.3.10 Aquifers with artesian pressure

Groundwater in some aquifers can be under pressure. The monitoring wells will register a higher water level than the groundwater level. In some cases, water would spill out from the well due to artesian pressure (Fig. 3.11).

Water level in the well is higher than the actual water level in the aquifer since the aquifer is under pressure (Fig. 3.12).

In Fig. 3.12, an impermeable clay layer is shown lying above the permeable sand layer. The dotted line shows the groundwater level if the clay layer is absent. Due to the impermeable clay layer, groundwater cannot reach the level shown by the dotted

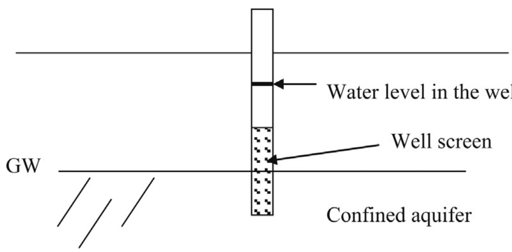


Figure 3.11 Monitoring well in a confined aquifer.

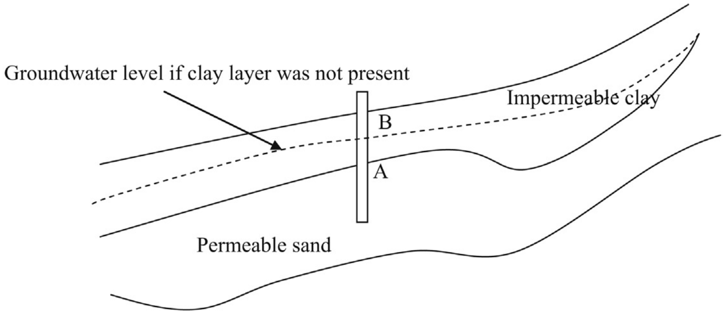


Figure 3.12 Artesian conditions.

line. Hence, groundwater level is confined to the point A in the monitoring well. When a well is installed, the water level will rise to point B, higher than the initial water level due to artesian pressure.

Reference

American Geoscience Institute "Glossary of Geology", 1980, New York.

Foundation types

4

There are a number of foundation types available for geotechnical engineers.

4.1 Shallow foundations

Shallow foundations are the cheapest and most common type of foundations (Fig. 4.1).

Shallow foundations are ideal for situations, when the soil immediately below the footing is strong enough to carry the building loads. In some situations soil immediately below the footing could be weak or compressible. In such situations, other foundation types need to be considered.

4.2 Mat foundations

Mat foundations are also known as raft foundations. Mat foundations, as the name implies, spread like a mat. The building load is distributed in a large area (Fig. 4.2).

4.3 Pile foundations

Piles are used when bearing soil is at a greater depth. In such situations, the load has to be transferred to the bearing soil stratum (Fig. 4.3).

4.4 Caissons

Caissons are nothing but larger piles. Instead of a pile, a group few large caissons can be utilized. In some situations, caissons could be the best alternative (Fig. 4.4).

4.5 Foundation selection criteria

Normally, all attempts are made to construct shallow foundations. This is the cheapest and fastest foundation type. The designer should look into bearing capacity and settlement when considering shallow foundations.

The geotechnical engineer needs to compute the bearing capacity of the soil immediately below the footing. If the bearing capacity is adequate, settlement needs to be computed. Settlement can be immediate or long-term. Immediate and long-term settlements should be computed (Fig. 4.5).

Pile Design and Construction Rules of Thumb

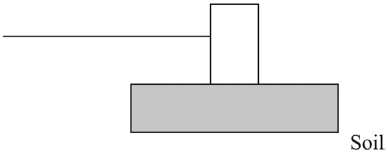


Figure 4.1 Shallow foundation.

Figure 4.2 Mat foundations.

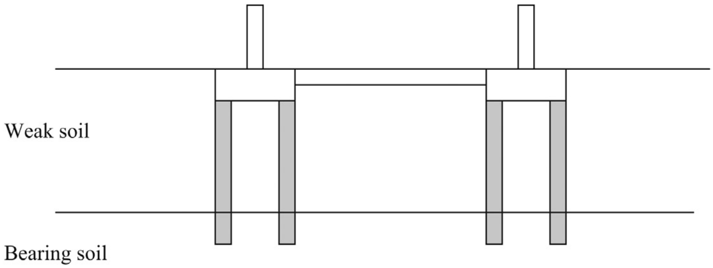


Figure 4.3 Pile foundations.

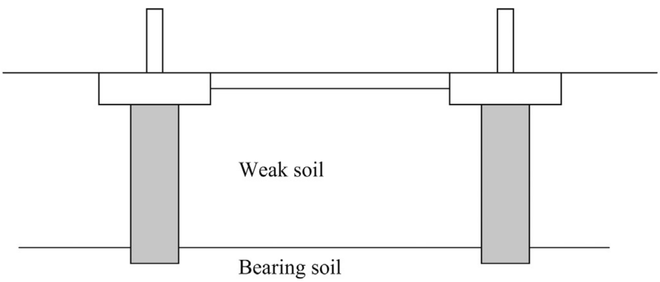


Figure 4.4 Caissons.

The Fig. 4.5 shows a shallow foundation, mat foundation, pile group, and a caisson. A geotechnical engineer needs to investigate the feasibility of designing a shallow foundation due to its cheapness and ease of construction. In the previous situation, it is clear that a weak soil layer just below the new fill may not be enough to support the shallow foundation. Settlement in soil due to loading of the footing also needs to be computed.

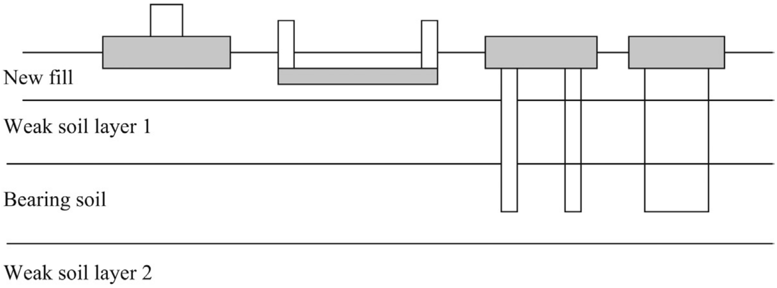


Figure 4.5 Different foundation types.

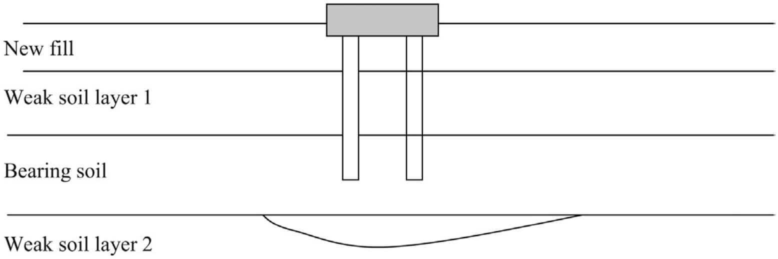


Figure 4.6 Punching failure (soil punching into the weak soil beneath due to pile load).

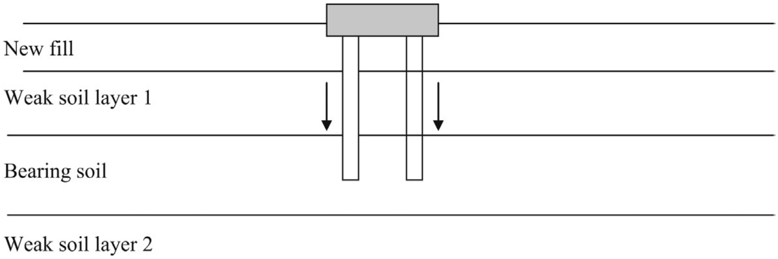


Figure 4.7 Negative skin friction.

If shallow foundations are not feasible, then other options need to be investigated. Mat foundations can be designed to carry large loads in the presence of weak soils. Unfortunately, cost is a major issue with mat foundations. Piles can be installed as shown in the figure ending in the bearing stratum. In this situation, one needs to be careful of the second weak layer of soil below the bearing stratum. Piles could fail due to punching into the weak stratum (Fig. 4.6).

The engineer needs to consider negative skin friction due to the new fill layer. Negative skin friction would reduce the capacity of piles (Fig. 4.7).

Due to the new load of the added fill material, weak soil layer 1 would consolidate and settle. Settling soil would drag the piles down with it. This is known as negative skin friction or down drag.

Pile types

5

All piles can be categorized as displacement piles and nondisplacement piles. Timber piles, closed-end steel pipe piles, and precast concrete piles displace the soil when driven into the ground. These piles are categorized as displacement piles.

Some piles displace soil during installation by a small degree (H-piles, open-end steel tubes, and hollow concrete piles).

5.1 Displacement Piles

Displacement piles can be further categorized into large and small, as discussed.

Large displacement piles	Small displacement piles
Timber piles	Tubular concrete piles
Precast concrete piles	H-Piles
(Reinforced and prestressed)	Open-end pipe piles
Closed-end steel pipe piles	Thin shell type
Jacked-down solid concrete piles	Jacked-down solid concrete cylinders

5.2 Nondisplacement piles

- Augercast piles (hole is augered and concreted). Also known as CFA piles (continuous flight auger). Generally rebars or I-beams are inserted into the hole prior to concreting.
- Steel casing withdrawn after concreting (alpha, delta, Frankie, and Vibrex piles).
- Auguring a hole and placing a thin shell and concreting.
- Drill or augur a hole and placement of concrete blocks inside the hole (Plate 5.1).

5.3 Timber piles

To have a 100-ft. long timber pile, one needs to cut down a tree of 150 ft. or more.

- Timber piles are cheaper than steel or concrete piles.
- Timber piles decay due to living microbes. Microbes need two ingredients to thrive: oxygen and moisture. For timber piles to decay, both ingredients are needed. Below groundwater, there is ample moisture but very little oxygen.

Pile Design and Construction Rules of Thumb

Plate 5.1 Pipe piles after installation.

- Timber piles submerged in groundwater will not decay. Oxygen is needed for the fungi (wood decaying microbes) to grow. Below groundwater level, there is no significant amount of air in the soil. Due to this reason very little decay occurs below the groundwater level.
- Moisture is the other ingredient needed for the fungi to survive. Above groundwater level, there is ample oxygen.
- In states like Nevada, Arizona, and Texas, there is very little moisture above the groundwater level. Since microbes cannot survive without water, timber piles could last for a long period of time.
- This is not the case for states in northern part of the United States. A significant amount of moisture will be present above groundwater level due to snow and rain. Hence, oxygen and moisture are available for the fungi to thrive. Timber piles would decay under such conditions. Creosoting and other techniques should be used to protect timber from decay above groundwater level.
- When the church of St Mark was demolished due to structural defects in 1902, woodpiles driven in 900 AD were in good condition. These old piles were reused to construct a new tower in place of the old church. Venice is a city with very high groundwater level and piles have been underwater for centuries.
- Engineers should consider possible future construction activities that could trigger lowering of the groundwater level (Fig. 5.1).

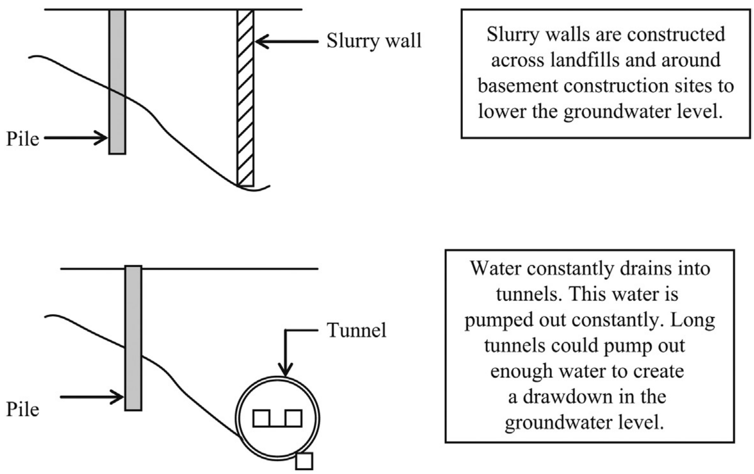


Figure 5.1 Groundwater lowering.

5.3.1 Timber pile decay: biological agents

There are many forms of biological agents that attack timber piles. Timber is an organic substance and nature will not let any organic substance go to waste.

5.3.1.1 Fungi

Fungi belong to the plant kingdom. The main distinction of plants from animals is their ability to generate food on their own. On the other hand, all food types of animals come from plants. On that account, fungi differ from other plants. Fungi are not capable of generating their own food since they lack chlorophyll, the agent that allows plants to generate organic matter using sunlight and inorganic nutrients. Due to this reason, fungi have to rely on organic matter for their supply of food.

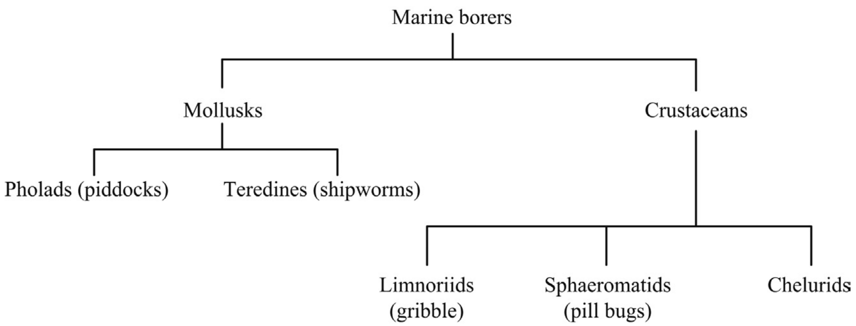
Fungi need organic nutrients (the pile itself), water, and atmospheric air to survive.

5.3.1.1.1 Identification of fungi attack

Woodpiles subjected to fungi attack can be identified by their swollen, rotten, and patched surface. Unfortunately, many piles lie underground and are not visible. Due to this reason, piles that could be subjected to environmental conditions favorable for fungi growth should be treated.

5.3.1.2 Marine borers

Marine borers belong to two families (mollusks and crustaceans). Both groups live in seawater and brackish water. Waterfront structures usually need to be protected from them.



Pholads (piddocks): these are sturdy creatures that can penetrate the toughest of wood. Pholads can hide inside their shells for prolonged periods of time.
Teredines (shipworms): these are commonly known as shipworms due to their worm-like appearance. They typically enter the wood at larvae stage and grow inside the wood.
Limnoriids: these belong to the group of crustaceans, and have seven legs and can grow to a size of 6 mm. They are capable of digging deep into the pile and damage the pile within, without any outward sign.
Sphaeromatids: they create large-sized holes, approximately ½ in. in size and can devastate woodpiles and other marine structures.
Chelurids: they are known to drive out Limnoriids from their burrows and occupy them.

5.3.1.3 Preservation of timber piles

There are three main types of wood preservatives available:

1. creosote
2. oil-borne preservatives
3. water-borne preservatives

These preservatives are usually applied in accordance with the specifications of the American Wood Preserver's Association (AWPA).

Preservatives are usually applied under pressure. Hence the term 'pressure treated' is used (Fig. 5.2).

5.3.2 Shotcrete encasement of timber piles

- Shotcrete is a mixture of cement, gravel, and water. High-strength shotcrete is reinforced with fibers.
- Shotcrete is sprayed onto the top portion of the pile where it could possibly be above groundwater level (Fig. 5.3).

5.3.3 Timber pile installation

Timber piles need to be installed with special care. Timber piles are susceptible to brooming and damage. Any sudden decrease in driving resistance should be investigated.

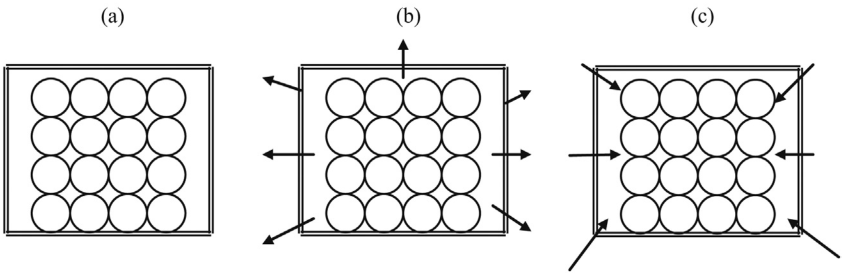


Figure 5.2 Pressure-treatment of timber piles. (a) Timber piles are arranged inside a sealed chamber. (b) Timber piles are subjected to a vacuum. Due to the vacuum, moisture inside piles would be removed. (c) After applying the vacuum, the chamber is filled with preservatives. The preservative is subjected to high pressure until a prespecified volume of liquid is absorbed by the wood.

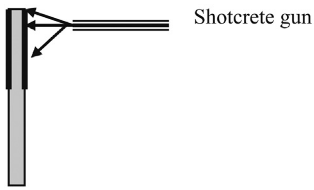


Figure 5.3 Shotcrete encasement on timber piles.

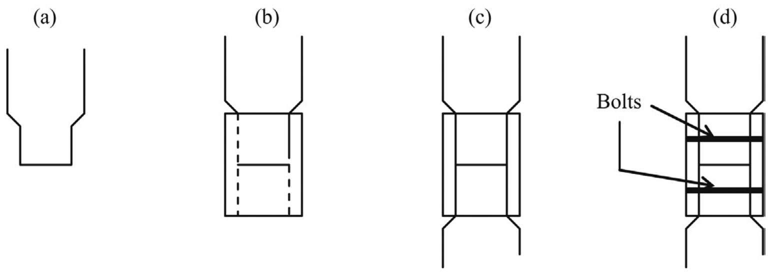


Figure 5.4 Splicing of timber piles. (a) Usually, timber piles are tapered prior to splicing as shown in Fig. 5.1. (b) The sleeve (or the pipe section) is inserted. (c) Bottom pile is inserted. (d) The pipe section is bolted to two piles.

5.3.3.1 Splicing of timber piles

- Splicing of timber piles should be avoided if possible. Unlike steel or concrete piles, timber piles cannot be spliced effectively.
- The usual practice is to provide a pipe section (known as a sleeve) and bolt it to two piles (Fig. 5.4).

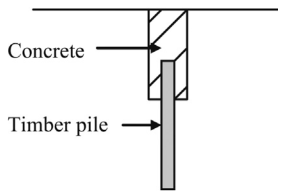


Figure 5.5 Concrete–timber composite pile.

- Sleeve joints are approximately 3–4 ft. in length. As one can see easily, the bending strength of the joint is much lower than the pile. Splice strength can be increased by increasing the length of the sleeve.
- Most building codes require no splicing to be conducted on the upper 10 ft. of the pile since the pile is subjected to high bending stresses at upper levels. If splicing is absolutely required for timber piles in the upper 10 ft. of the pile, it is recommended to construct a composite pile with the upper section filled with concrete. This type of construction is much better than splicing (Fig. 5.5).
- Sleeves larger than the pile diameter: Sleeves larger than the pile may get torn and damaged during driving and should be avoided. If this type of sleeve is to be used, the engineer should be certain that the pile sleeve would not be driven through hard strata.
- Uplift piles: Timber splices are extremely vulnerable to uplift (tensile) forces and should be avoided. Other than sleeves, steel bars and straps are also used for splicing.

5.4 Steel 'H' piles

- Timber piles cannot be driven through hard ground.
- Steel 'H' piles are essentially end-bearing piles. Due to limited perimeter area, 'H' piles cannot generate much frictional resistance.
- Corrosion is a major problem for steel 'H' piles. The corrosion is controlled by adding copper into steel.
- 'H' Piles are easily spliced. 'H' Piles are ideal for highly variable soil conditions.
- 'H' Piles could bend under very hard ground conditions. This is known as 'dog legging' and the pile installation supervisor needs to make sure that the piles are not out of plumb.
- H-Piles can get plugged during the driving process (Fig. 5.6).
- If the H-pile is plugged, end bearing would increase due to larger area. On the other hand, skin friction would become smaller due to lesser wall area.

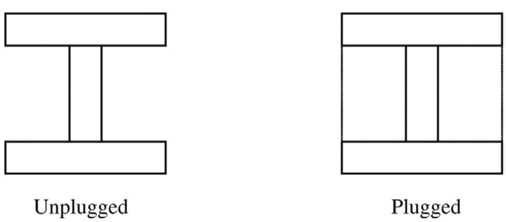


Figure 5.6 Plugging of H-piles.

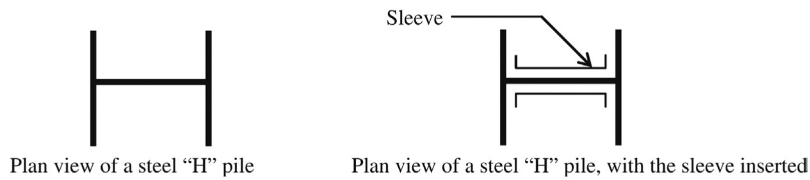


Figure 5.7 Splicing of H-piles.

- When H-piles are driven, both analyses should be done (unplugged and plugged) and the lower value used for design.
    - Unplugged: low end bearing, high skin friction
    - Plugged: low skin friction, high end bearing

5.4.1 Splicing of H-piles

- Step 1: the sleeve is inserted into the bottom part of the H-pile as shown and bolted to the web (Fig. 5.7).
- Step 2: the top part of the pile is inserted into the sleeve and bolted.

5.4.2 Guidelines for splicing (international building code)

The International Building Code states that splices shall develop not less than 50% of the pile bending capacity. If the splice is occurring in the upper 10 ft. of the pile, eccentricity of 3 in. should be assumed for the column load. The splice should be capable of withstanding the bending moment and shear forces due to a 3-in. eccentricity.

5.5 Pipe piles

- Pipe piles are available in many sizes. Twelve-inch diameter pipe piles have a range of thicknesses.
- Pipe piles can be driven either open end or closed end. When driven open end, the pipe is cleaned with a jet of water.

5.5.1 Closed-end pipe piles

- Closed-end pipe piles are constructed by covering the bottom of the pile with a steel plate.
- In most cases pipe piles are filled with concrete. In some cases, pipe piles are not filled with concrete to reduce the cost. If pipe piles are not filled with concrete, then a corrosion protection layer should be applied.
- If a concrete-filled pipe pile gets corroded, most of the load-carrying capacity of the pile would remain intact due to the concrete. On the other hand, an empty pipe pile would lose significant amount of its load-carrying capacity.
- Pipe piles are good candidates for batter piles.
- The structural capacity of pipe piles is calculated based on concrete strength and steel strength. The thickness of the steel should be reduced to account for corrosion. (Typically reduced by 1/16 in. to account for corrosion) (Fig. 5.8).

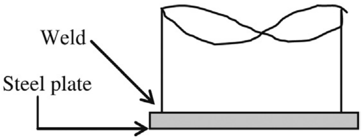


Figure 5.8 A pipe pile is covered with an end cap. The end cap is welded as shown.

Plate 5.2 Closed-end pipe piles.

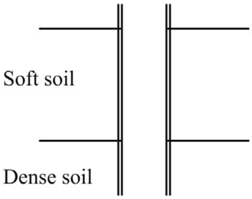
- In the case of close-end driving, soil heave can occur. There are occasions where open-end piles also generate soil heave. This is due to plugging of the open end of the pile with soil.
- Pipe piles are cheaper than steel H-piles or concrete piles (Plate 5.2).

5.5.2 Open-end pipe piles

- Open-end pipe piles are driven and soil inside the pile is removed by a water jet (Fig. 5.9, Plate 5.3).

5.5.2.1 Ideal situations for open-end pipe piles

- Soft layer of soil followed by a dense layer of soil.



- *Site condition*: Soft layer of soil followed by a hard layer of soil.
- Open-end pipe piles are ideal for this situation. After driving to the desired depth, soil inside the pipe is removed and concreted.
- Closed-end pipe piles also could be considered to be ideal for this type of situation.

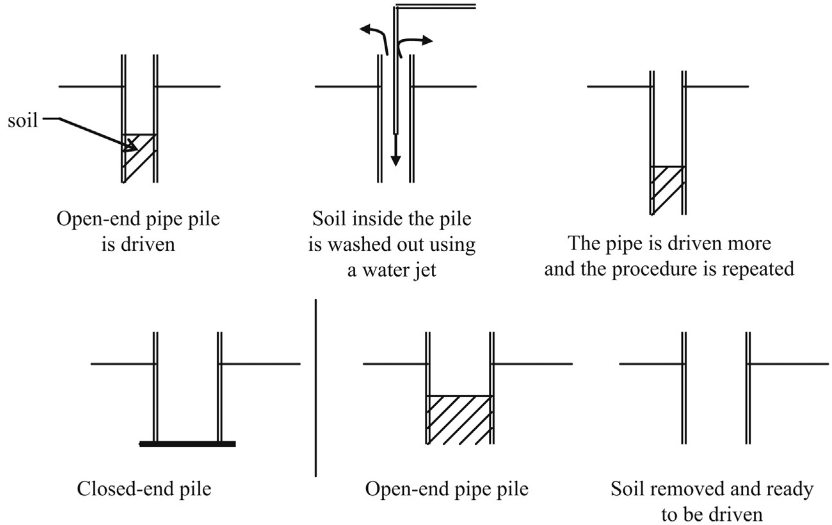
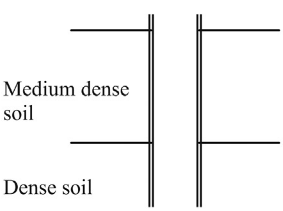


Figure 5.9 Open-end pipe piles are easier to drive through hard soils than closed-end pipe piles.

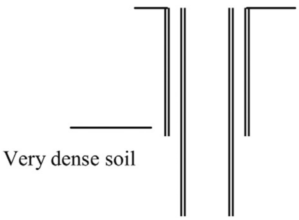
Plate 5.3 Open-end pipe piles.

- Medium dense layer of soil followed by a dense layer of soil.



- *Site condition*: Medium dense layer of soil followed by a hard layer of soil.
- Open-end pipe piles are ideal for this situation as well.
- Closed-end pipe piles may not be a good choice, since driving closed-end pipe piles through medium dense soil layer may be problematic.
- It is easier to drive open-end pipe piles through a dense soil layer than closed end pipe piles.

5.5.2.2 Telescoping



- Usually telescoping is conducted, when very dense soil is encountered.
- In such situations, it may not be possible to drive a larger pipe pile.
- Hence, a small diameter pipe pile is driven inside the original pipe pile.

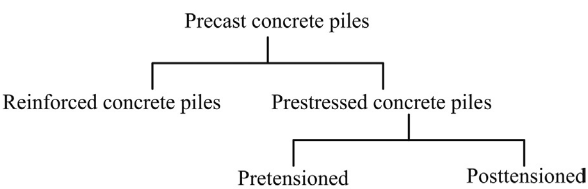
- Due to the smaller diameter of the telescoping pipe pile, end-bearing capacity of the pile would reduce. To accommodate the loss, the length of the telescoping pile should be increased.

5.5.2.3 Splicing of pipe piles

Pipe piles are spliced by fitting a sleeve. The sleeve would fit into the bottom section of the pile as well as the top section (Fig. 5.10).

5.6 Precast concrete piles

Precast concrete piles could be either reinforced concrete piles or prestressed concrete piles.



5.6.1 Reinforced concrete piles

Reinforced concrete piles are constructed by reinforcing the concrete as shown in Fig. 5.11.

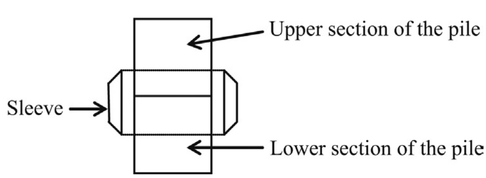


Figure 5.10 Splicing of pipe piles.

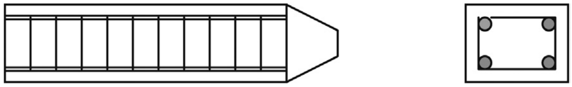


Figure 5.11 Reinforced concrete piles.

5.6.2 Prestressed concrete piles

Please refer Figs. 5.12 and 5.13 for details about prestressed concrete piles.

5.6.3 Hollow-tubular section concrete piles

- Most hollow tubular piles are posttensioned to withstand tensile stresses.
- Hollow tubular concrete piles can be driven either closed end or open end. A cap is fitted at the end for closed-end driving.
- These piles are not suitable for dense soils.
- Splicing is expensive.
- It is a difficult and expensive process to cutoff these piles. It is very important to know the depth to the bearing stratum with reasonable accuracy.

5.6.4 Driven cast-in-place concrete piles

- Step 1: steel tube is driven first.
- Step 2: soil inside the tube is removed by water jetting.
- Step 3: reinforcement cage is set inside the casing.
- Step 4: the empty space inside the tube is concreted while removing the casing.

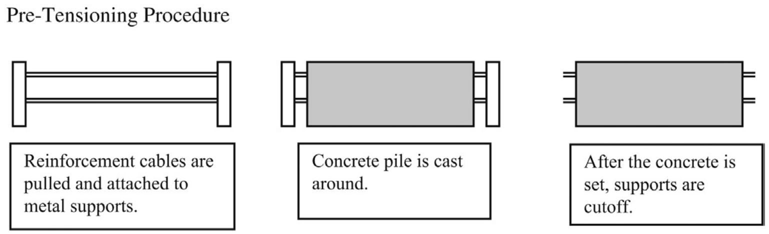


Figure 5.12 Pretensioning procedure.

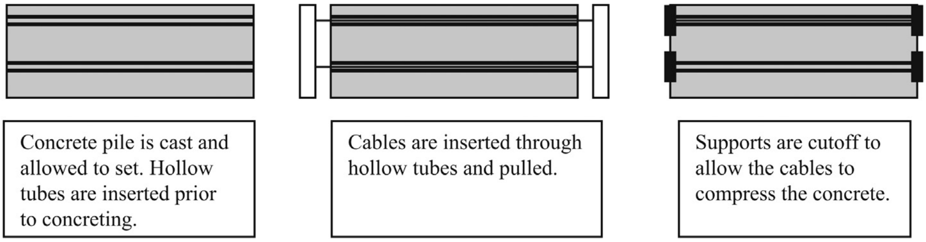


Figure 5.13 Posttensioning procedure.

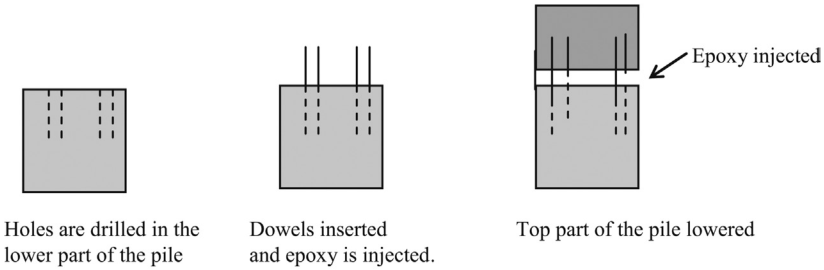


Figure 5.14 Splicing of concrete piles.

5.6.5 *Splicing of concrete piles*

Splicing of prestressed concrete piles is not an easy task.

- Holes are drilled in the bottom section of the pile.
- Dowels are inserted into the holes and filled with epoxy.
- Holes are driven in the upper section of the pile as well.
- The top part is inserted into the protruding dowels.
- Epoxy grout is injected into the joint (Fig. 5.14).

5.7 Augercast piles (continuous flight auger piles)

Augercast piles are constructed by drilling a hole and then filling the hole with concrete and a rebar casing. Augercast piles are also known as continuous flight auger piles.

5.7.1 *Construction methodology*

Step 1: drill a hole with augers (Fig. 5.15)
Step 2: install a casing
A casing is installed in most cases to make sure that the hole will not collapse after withdrawing the augers. The casing is pushed into the hole.

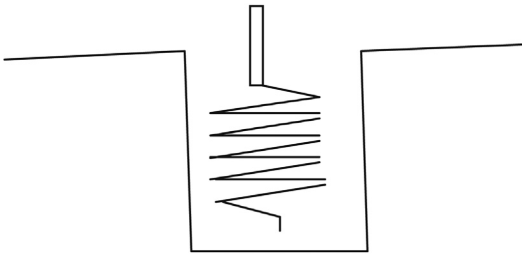


Figure 5.15 Augers.

Push the casing into the hole. This is done with an auger drill rig. Drill rigs are capable of pushing a casing into the ground after the hole is augered (Fig. 5.16) (Plate 5.4).
Step 3: install a rebar cage, steel rod or I-beam inside the casing (Fig. 5.17) (Plates 5.5–5.7).
Step 4: concrete the hole (Fig. 5.18).
Step 5: build the pile cap and columns on top of the augercast pile (Fig. 5.19).

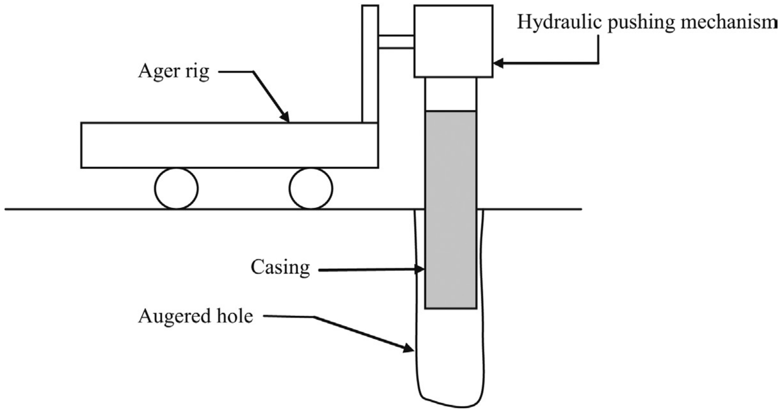


Figure 5.16 Installation of casing.

Plate 5.4 Casings are pushed into the ground.

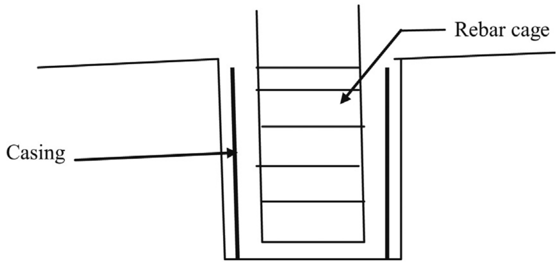


Figure 5.17 Install a rebar cage in the hole inside the casing.

Plate 5.5 Rebar cages.

Basically augercast piles are concrete columns built in the ground. Casing is provided to keep the hole from collapsing when the augers are withdrawn. Casings are left in the hole or removed during concreting. I should mention here that casings are expensive and can be reused. Also, the soil–concrete bond is higher in most cases compared to soil–casing bond. Next we will look into casing removal type augercast piles.

5.7.2 Casing removal type

The procedure is the same as the previous one except that the casing is removed (Fig. 5.20).

Plate 5.6 Rebar cage inside an augercast pile. Casing is also seen in the picture.

Plate 5.7 Instead of the rebar cage, designers can install I-beams.

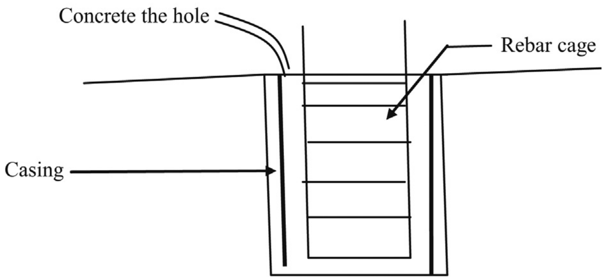


Figure 5.18 Concrete the hole.

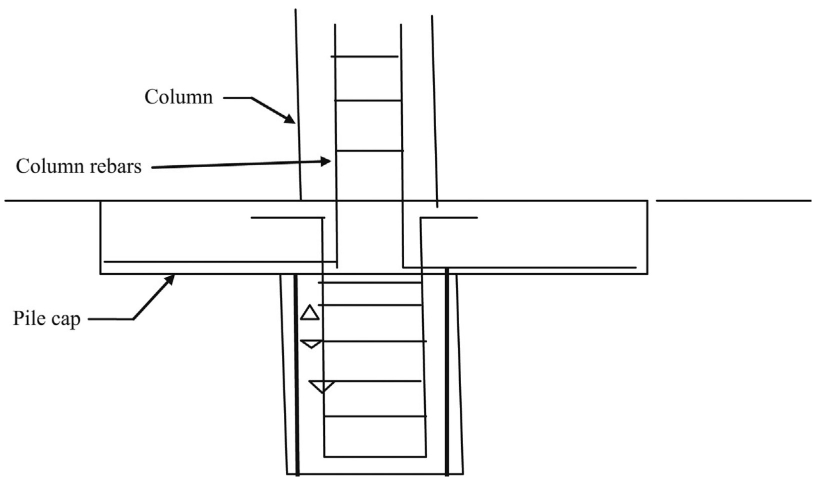


Figure 5.19 Build the column on top of the augercast pile.

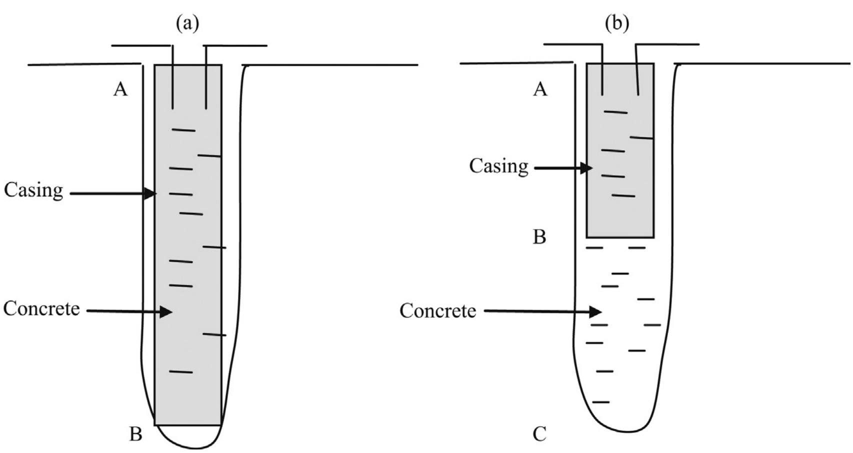


Figure 5.20 Cased augercast pile versus casing removal type.

Fig. 5.20a shows an augercast pile with a casing. Fig. 5.20b shows an augercast pile with the casing partially removed. If the soil is stiff enough, the casing can be completely removed.

Let us look at the skin friction in the two piles shown.

5.7.3 Skin friction in cased augercast pile

In the cased pile, skin friction from point A to point B is between the soil and casing material. Casings are always made of metal.

5.7.4 Skin friction in partially cased augercast pile

In the partially cased pile, skin friction from point A to point B is between the soil and casing material. The skin friction between point B and point C is between the soil and concrete.

Skin friction between metal and soil is low compared to skin friction between soil and concrete. Concrete would travel into soil pores and generate a bond. Therefore, it is better to remove the casing. But in most situations the soil near the ground surface is very weak and could collapse into the hole. Due to this reason a portion of the casing is left in the hole. If the geotechnical engineer determines that the soil is strong enough to stay open without collapsing, the casing can be fully removed.

5.8 Frankie piles

The following major components are needed for Frankie piles (Fig. 5.21): pile hammer, casing, mandrel, bottom cap, reinforcement cage, and concreting method.

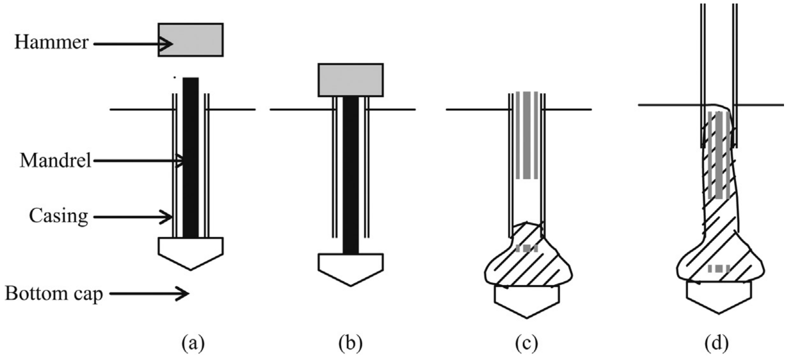
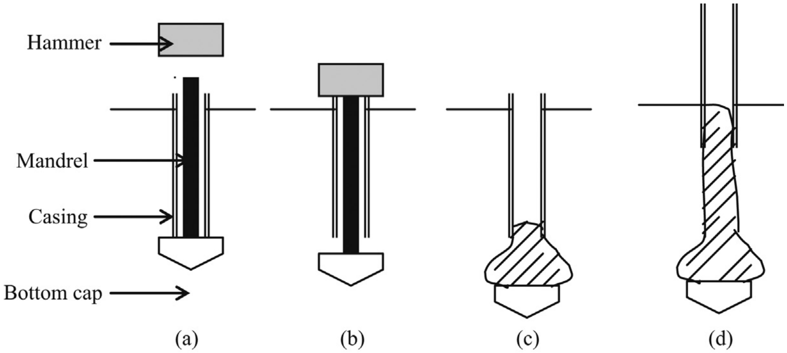


Figure 5.21 Frankie pile installation. (a) The casing is driven to the desired depth using an internal mandrel. (b) The bottom cap is broken off by heavy hammer drops. (c) Mandrel is removed and reinforcement cage is inserted. (d) Casing is lifted and concreted.



Following major components are needed for Delta piles.
Pile hammer, Casing, Mandrel, Bottom cap, Concreting method

A: The casing is driven to the desired depth using an internal mandrel.
B: The bottom cap is broken off by heavy hammer drops.
C: Mandrel is removed.
D: Casing is lifted and concreted. (No reinforcement cage)

Figure 5.22 Delta pile installation. (a) The casing is driven to the desired depth using an internal mandrel. (b) The bottom cap is broken off by heavy hammer drops. (c) Mandrel is removed. (d) Casing is lifted and concreted (no reinforcement cage).

5.9 Delta piles

The following major components are needed for Delta piles (Fig. 5.22): pile hammer, casing, mandrel, bottom cap, and concreting method.

5.10 Vibrex piles (casing removal type)

The following major components are needed for Vibrex piles (Fig. 5.23): vibrating hammer, casing, bottom cap, reinforcement cage, and concreting method.

5.11 Compressed base type

In this pile type, larger base area is created. In most cases it is not easy to estimate the bearing area of the base (Fig. 5.24).

5.12 Precast piles with grouted base

Grouted base piles are constructed by auguring a hole and grouting the bottom under pressure. A precast pile (steel, concrete or timber) is inserted into the grouted base (Fig. 5.25).

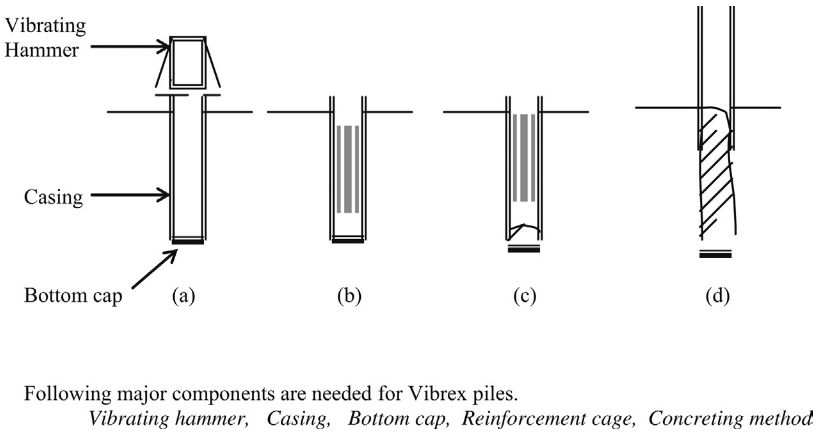


Figure 5.23 Vibrex pile installation. (a) The casing is driven to the desired depth using a vibrating hammer. (b) Reinforcement cage is inserted. (c) Bottom cap is removed. A removal mechanism is provided at the tip. (d) Casing is lifted and concreted.

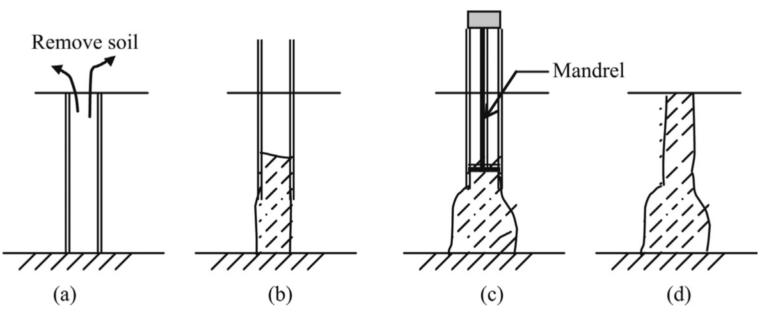


Figure 5.24 Compressed base type pile installation. (a) Insert a casing to the hard stratum and remove soil inside the casing. (b) Lift the casing and concrete. (c) Apply pressure using a hammer and an internal mandrel. (d) Concrete rest of the pile.

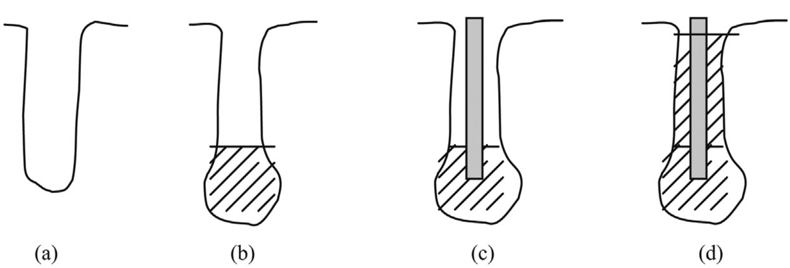


Figure 5.25 Precast piles with grouted base installation. (a) Augur a hole. (b) Grout the base under pressure. (c) Insert the precast pile. (d) Grout the annulus of the hole.

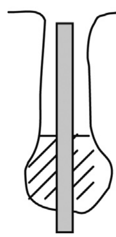


Figure 5.26 Pile penetrates the grouted base and fails (punching failure).

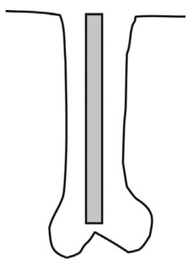


Figure 5.27 Grouted base fails (splitting failure).

5.12.1 Capacity of grouted base piles

Failure mechanisms of grouted base piles (Figs. 5.26 and 5.27).

5.13 Mandrel driven piles

Theory: Mandrels are used to drive a thin shell into the soil. The mandrel is later withdrawn and the shell is concreted (Fig. 5.28).

Note: Mandrel driven piles are not suitable for unpredictable soil conditions. It is not easy to increase the length of the mandrel if the pile had to be driven to a longer depth.

5.14 Composite piles

Piles that consist of two or more different types of piles are known as composite piles.

5.14.1 Pipe pile/timber pile composite

Method 1:

It is a well-known fact that timber piles would decay above groundwater. Due to this reason, steel (or concrete) pipe piles are used above groundwater level (Fig. 5.29).

Step 1: drive the pipe pile below the groundwater level.

Step 2: drive the timber pile inside the steel or concrete pipe pile.

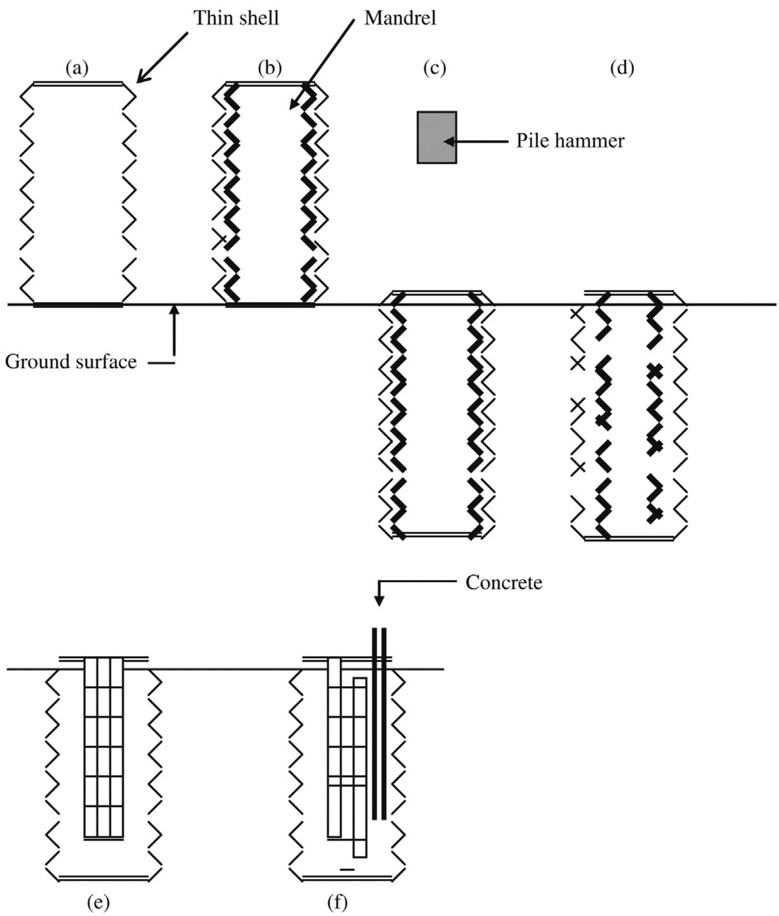


Figure 5.28 Mandrel driven pile installation. (a) Thin shell. (b) Insert the mandrel into the thin shell. The mandrel is fitted into the corrugations of the thin shell. The mandrel is a solid object that can be driven into the ground with a pile hammer. (c) Drive the mandrel and the thin shell into the ground. The mandrel will drag down the thin shell with it. (d) Collapse the mandrel. Mandrels are collapsible. After the mandrel is collapsed, it is removed from the hole. (e) Insert the reinforcement cage. (f) Concrete the thin shell.

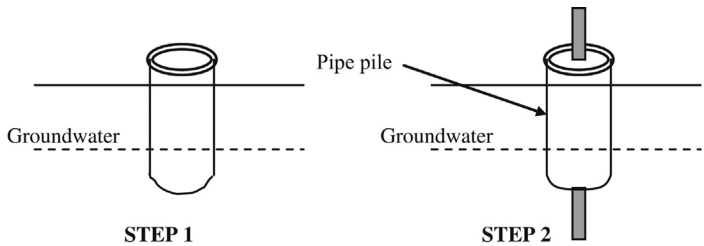


Figure 5.29 Pipe pile/timber pile composite.

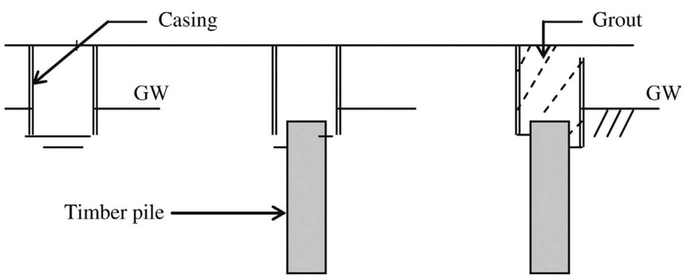


Figure 5.30 Pipe pile/timber pile composite – method 2.

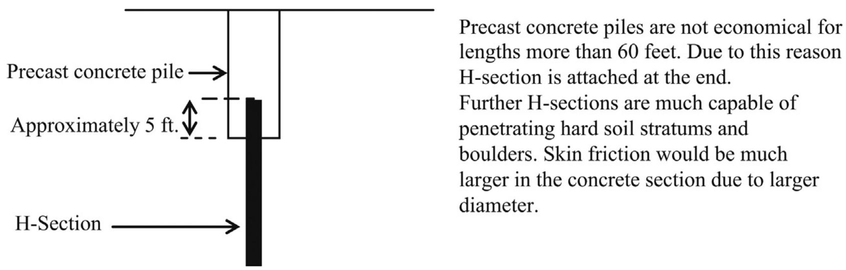


Figure 5.31 Precast concrete piles with H-section.

Step 3: concrete the annulus between pipe pile and the timber pile.

Method 2:

- A steel pipe is driven into the ground and soil inside the pipe is removed. A timber pile is driven inside the casing.
- The casing is grouted (Fig. 5.30).

5.14.2 Precast concrete piles with H-section

Precast concrete piles are not economical for lengths more than 60 ft. Due to this reason, an H-section is attached at the end. Further H-sections are more capable of penetrating hard soil strata and boulders. Skin friction would be much larger in the concrete section due to larger diameter (Fig. 5.31).

5.14.3 Uncased concrete and timber piles

- A steel casing is driven. The soil is removed and timber pile is installed (Fig. 5.32a–c).
- Concreting is done while the casing is lifted (Fig. 5.32d and e).
- The timber pile is installed below the groundwater level (Fig. 5.32).

5.15 Fiber-reinforced plastic piles

- Fiber-reinforced plastic (FRP) piles are becoming increasingly popular due to their special properties.

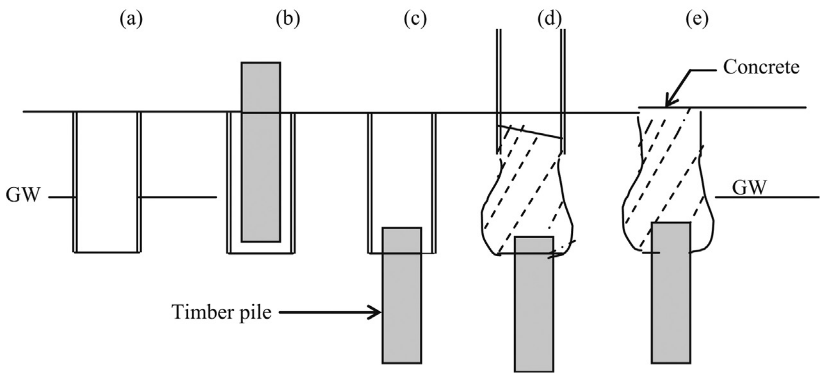


Figure 5.32 Uncased concrete and timber piles.

- Timber piles deteriorate over time and are also vulnerable to marine borer attack. Plastic piles are not vulnerable to marine borer attack.
- Corrosion is a major problem for steel piles.
- Concrete piles may deteriorate in marine environments.
- For an instance, a plastic pile with a steel core could be used in highly corrosive environments. The plastic outer layer would protect the inner core from marine borer attack and corrosion.

5.15.1 Materials used

Fiberglass and HDPE plastic are the most popular materials used.

5.15.2 Types of FRP piles

5.15.2.1 Plastic pile with a steel core

A steel core provides rigidity and compressive strength to the FRP pile. In some instances, plastic could peel off from the steel core (Fig. 5.33).

5.15.2.2 Reinforced plastic piles

Most of these piles are made of recycled plastic. Instead of the steel core, steel rebars are also used to provide rigidity and strength.

5.15.2.3 Fiberglass pipe piles

These piles consist of a fiberglass pipe and a concrete core. Typically fiberglass piles are driven and filled with concrete.

5.15.2.4 Plastic lumber

Plastic lumbers consist of recycled HDPE and fiberglass.

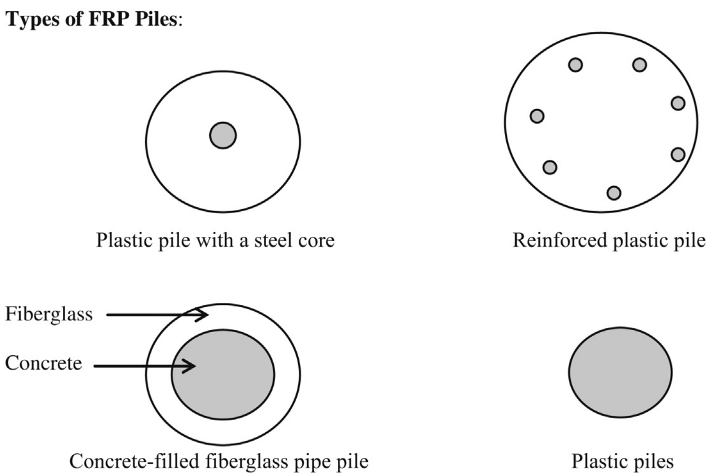


Figure 5.33 Types of FRP piles.

5.15.3 Use of wave equation for plastic piles

Wave equation could be used for drivability analysis of plastic piling by using adjusted piling parameters.

Selection of piles

6

Not every site is the same. Some sites are in the city and closer to schools and hospitals. In such sites, driven piles may not be suitable. Pile driving creates large noise and vibrations. In such situations, augercast piles should be considered. Important issues need to be taken into account during the selection of piles, as shown below:

- noise
- vibrations
- pile capacity (skin friction and end bearing)
- obstructions
- pile splicing
- budget
- chemicals in soil
- durability

Pile capacity comes from skin friction and end bearing. Most engineers prefer to construct end-bearing piles, extended to the bedrock. This is the safest type of pile, since in most cases the bedrock is solid. The integrity of the bedrock should be confirmed through rock coring. Friction piles that rely on skin friction for the capacity generally tend to have high settlement values. Skin friction in sandy soils is due to *friction* between the pile surface and the soil. Skin friction in clay soils is due to *adhesion* between the pile material and clay. Skin friction in sandy soils increases with depth. After a certain depth, increase in skin friction tends to taper off. It is generally accepted that adhesion does not increase with depth. Contrary to the traditional view there is some increase in adhesion with depth. This issue will be discussed later.

Some sites contain obstructions such as concrete debris, boulders, man-made fill, and glacial till.

Another issue that designers should be mindful about, is the splicing of piles. If the depth of the pile cannot be properly determined prior to driving, piles have to be spliced. Steel piles can be easily spliced. Timber piles and concrete piles cannot be spliced easily. In a later chapter, different splice methods will be discussed (Plate 6.1).

Chemicals in soil affect the pile material. Timber piles and steel piles may not be suitable in marine environments. Concrete piles may not be suitable in soils with high sulfur content.

Funds are not unlimited. In some cases, the budget could be very tight. The most suitable piles for the project may be too expensive. Hence, the next best-suited pile has to be selected.

Plate 6.1 Special rock cutting cutter head (these can cut through boulders).

Scenario 1

The following soil profile is available at a site. The geotechnical engineer is assigned to design a foundation for a 20-story building. He or she has ruled out shallow footings due to unpredictable fill material in the site. The depth of bedrock is known with good accuracy. Noise is not an issue since the site is located in a remote area. What piles are most suited for this project? (Fig. 6.1)

Solution

Since noise is not an issue, driven piles can be considered. Timber piles can be ruled out for a 20-story building since they may not have enough capacity. Piles resting on bedrock are the best solution. In this case, the bedrock is only 16 m below the ground surface. Hence, it is a good idea to design end-bearing piles resting on bedrock. The top 5 m consists of many obstructions (boulders, concrete debris, etc.). Pipe piles (open end and closed end) may have trouble going through boulders. Hence, these piles may not be good candidates for this project.

Another option is augering a hole down to 5 m and then installing pipe piles (Figs. 6.2–6.5).

6.1 H-sections

H-sections are significantly better in going through boulders and concrete debris. H-piles should also be seriously considered. H-piles are sturdy and can drive through obstructions.

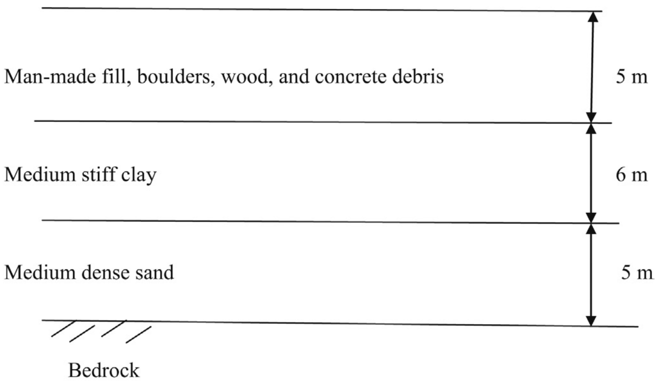


Figure 6.1 Soil profile.

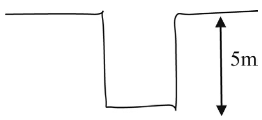


Figure 6.2 Auger a hole down to 5 m using a strong cutting auger.

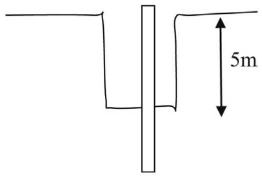


Figure 6.3 Drive pipe piles.

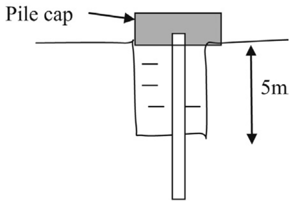


Figure 6.4 Backfill and construct the pile cap.

H-piles driven all the way to the bedrock are a valid option to consider.

6.2 Concrete piles

In this case, the depth of the bedrock is known with good accuracy. Hence, concrete piles can be made to proper length. Driving shoes can be attached to concrete piles to bust through obstructions.

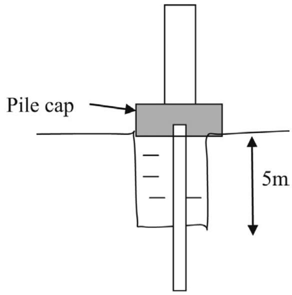


Figure 6.5 Install the column.

6.3 Augercast piles

Augercast piles are a very good option when there are obstructions. There are special rock-crushing augers that can be used to go through boulders. Drill a hole all the way down to bedrock and place a rebar cage and concrete (Plate 6.2).

Plate 6.2 H-Pile installed in the ground.

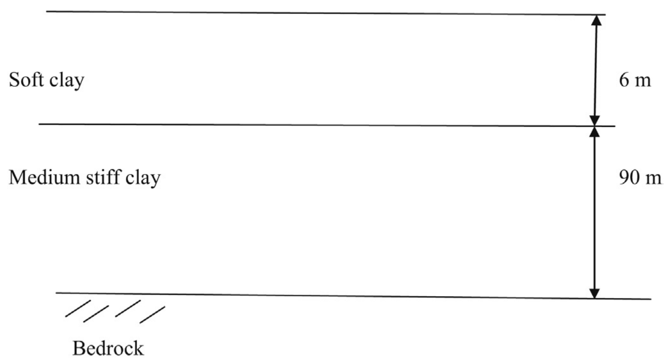


Figure 6.6 Soil profile and bedrock.

Scenario 2

A geotechnical engineer is assigned to design a foundation for a 20-story building. He or she has ruled out shallow footings due to soft clay at the top. The depth of the bedrock is not known with good accuracy. Noise is not an issue since the site is located in a remote area. What piles are most suited for this project? (Fig. 6.6)

Solution
The bedrock is too deep to reach. Hence, friction piles would be a better option. Friction piles get their capacity mostly due to skin friction. Since noise is not an issue, driven piles can be considered.

6.4 Open- and closed-end pipe piles

Since there are no obstructions or hard soil, open- or closed-end pipe piles can be driven to the desired depth. Desired depth is dependent upon skin friction between the soil and the pile. Unit skin friction between steel and soil is less compared to concrete and soil.

6.5 Concrete piles

Unit skin friction in concrete piles would be higher compared to steel. Hence, concrete piles are an attractive option.

6.6 Augercast piles

Soft clay and some medium stiff clay can collapse during the withdrawal of augers. Hence, casing may be needed. Casing can be withdrawn during concreting. Concrete or grout will go into the pores of soil and develop very high skin friction between the pile and the soil.

6.7 H-piles

H-Piles have a large surface area. Hence, skin friction will be high. On the other hand H-piles are made of steel. Steel–soil unit skin friction is low compared to concrete or timber (Fig. 6.7).

Soil and pile contact area is shown with dotted lines. This area can be reduced due to soil plugging (Fig. 6.8).

During driving, soil can get plugged as shown. When this happens, the contact area between the soil and pile will be reduced. Therefore, skin friction also will be reduced. On the other hand, unit skin friction between soil–soil may be higher than steel and soil.

Scenario 3

A site contains loose sand to a depth of 7 m followed by soft clay layer of 5 m. Shallow foundations have been ruled out for the site. There are nearby existing buildings. Noise is not an issue. The soil profile for the site is shown in Fig. 6.9. The centerline of columns in the existing building is 3 m from the proposed building. Since the bedrock is too deep, friction piles ending in medium dense sand is recommended.

Solution

The adjacent building is too close for displacement piles. When piles are driven, soil will be displaced. Displaced soil would move and can damage the piles in the existing building (Fig. 6.10).

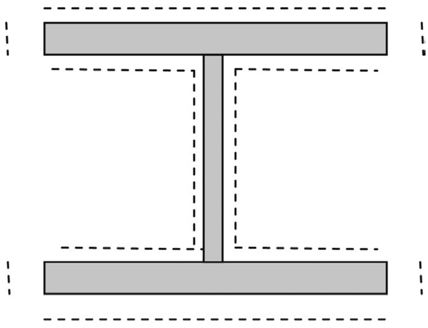


Figure 6.7 Soil–pile contact area in H-piles.

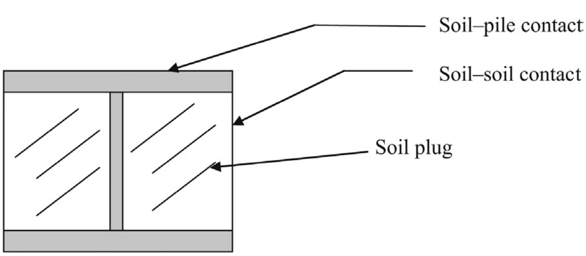


Figure 6.8 Soil plugging.

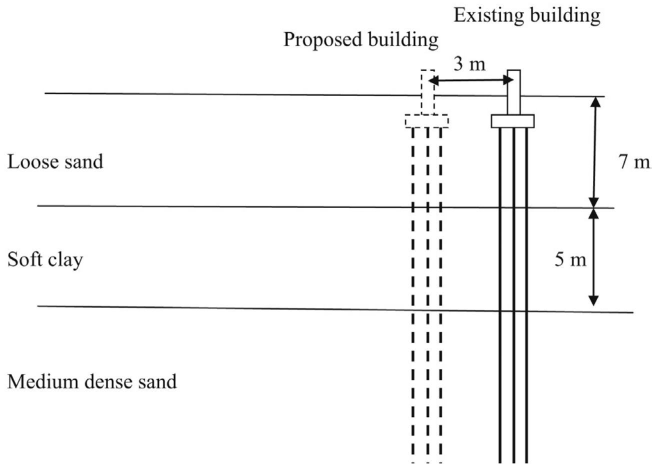


Figure 6.9 Pile configuration.

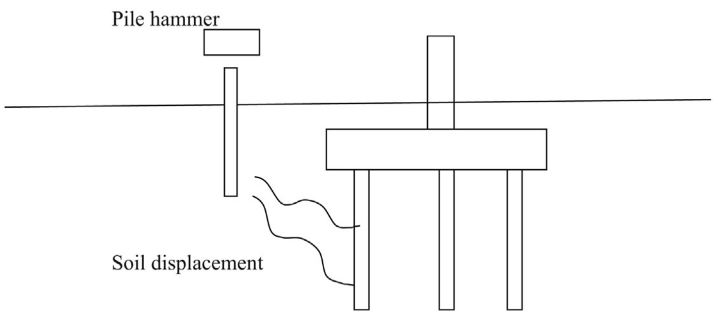


Figure 6.10 Soil displacement.

Due to this reason, displacement piles or driven piles can be ruled out. Augercast piles are best suited for this project. If driven piles were to be used, H-piles would be the best type of driven piles for the project. H-Piles displace only a small quantity of soil. However, if soil plugging can occur in H-piles, soft clay could get plugged in the H-pile. If soil plugging occurs, large amounts of soil will be displaced. This in turn can damage the existing piles in the nearby building.

Static and dynamic analysis

7

7.1 Pile design in sandy soils (static analysis)

The modified version of the Terzaghi bearing capacity equation is widely used for pile design (Terzaghi et al., 1996).

The third term or the density term in the Terzaghi bearing capacity equation is negligible in piles and hence usually ignored. Lateral earth pressure coefficient (K) is introduced to compute the skin friction of piles.

$$P_{\text{ultimate}} = \underbrace{(\sigma'_t \times N_q \times A)}_{\text{End bearing term}} + \underbrace{(K\,\sigma'_v \times \tan\delta \times A_p)}_{\text{Skin friction term}}$$

P_{ultimate}, ultimate pile capacity; σ'_t, effective stress at the tip of the pile; N_q, bearing factor coefficient; A, cross-sectional area of the pile at the tip; K, lateral earth pressure coefficient (use the Table 7.1 for K values); σ'_v, effective stress at the perimeter of the pile (σ'_v varies with the depth; usually, σ'_v value at the midpoint of the pile is obtained); tanδ, friction angle between pile and soil (see Table 7.1 for δ values), and A_p, perimeter area of the pile.

For round piles, $A_p = (\pi \times d) \times L$ (d, diameter; and L, length of the pile).

7.1.1 Description of terms

7.1.1.1 Effective stress (σ')

When a pile is driven, effective stress of the existing soil around and below the pile would change. Fig. 7.1 shows effective stress prior to driving a pile.

Effective stress after driving the pile is shown in Fig. 7.2. When a pile is driven, the soil around the pile and below the pile would be compacted.

Due to disturbance of soil during the pile-driving process, σ'_{aa} and σ'_{bb} cannot be accurately computed. Usually, an increase in effective stress due to pile driving is ignored.

7.1.1.2 N_q (bearing capacity factor)

Many researchers have provided techniques to compute bearing capacity factors. End bearing capacity is a function of friction angle, dilatancy of soil, and relative density. All these parameters are lumped into N_q. Different methods of obtaining the N_q value will be discussed.

Pile Design and Construction Rules of Thumb

Table 7.1 Friction angle versus N_q (NAVFAC DM 7.2, 1984)

ϕ	26	28	30	31	32	33	34	35	36	37	38	39	40
N_q (for driven piles)	10	15	21	24	29	35	42	50	62	77	86	120	145
N_q (for bored piles)	5	8	10	12	14	17	21	25	30	38	43	60	72

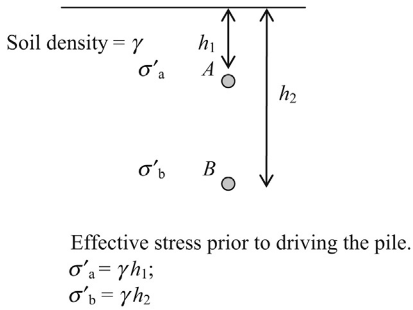


Figure 7.1 Effective stress in a pile.

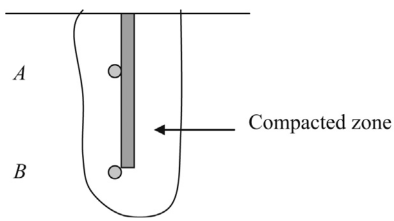


Figure 7.2 Compacted zone.

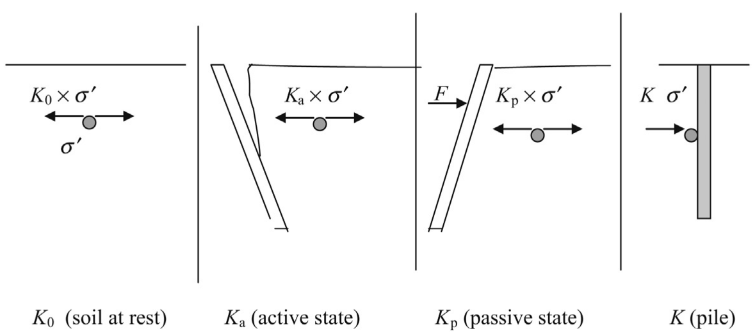


Figure 7.3 PSoil pressure near piles.

7.1.1.3 K (lateral earth pressure coefficient)

Prior to discussing the lateral earth pressure coefficient related to piles, it is necessary to investigate lateral earth pressure coefficients in general (Fig. 7.3).

7.1.1.3.1 K_0:– in situ soil condition

For at rest condition, horizontal effective stress is given by $K_0 \times \sigma'$(K_0 = lateral earth pressure coefficient at rest and σ' is the vertical effective stress).

7.1.1.3.2 K_a:– active condition

In this case, soil is exerting the minimum horizontal effective stress, since soil particles have room to move (K_a, active earth pressure coefficient). K_a is always smaller than K_0.

7.1.1.3.3 K_p:– passive condition

In this case, soil is exerting the maximum horizontal effective stress, since soil particles have been compressed (K_p, passive earth pressure coefficient). K_p is always greater than K_0 and K_a.

7.1.1.3.4 K: soil near piles

Soil near a driven pile would be compressed. In this case, soil is definitely exerting more horizontal pressure than the in situ horizontal effective stress (K_0). Since K_p is the maximum horizontal stress that can be achieved, K should be in between K_0 and K_p.

$$K_a < K_0 < K\,(\text{pile condition}) < K_p$$

Hence $K = (K_0 + K_a + K_p)/3$ can be used as an approximation.
Equations for K_0, K_a, and K_p are

$$K_0 = 1 - \sin\varphi$$

$$K_a = 1 - \tan^2\left(45 - \frac{\varphi}{2}\right)$$

$$K_p = 1 + \tan^2\left(45 + \frac{\varphi}{2}\right)$$

7.1.1.4 tanδ (wall friction angle)

Friction angle between pile material and soil δ decides the skin friction. Friction angle (δ) varies with the pile material and soil type. Various agencies have conducted laboratory tests and have published δ values for different pile materials and soils (Fig. 7.4).

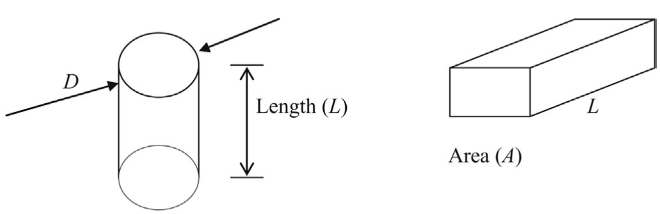


Figure 7.4 Perimeter surface area of a pile.

7.1.1.5 A_p (perimeter surface area of the pile)

Perimeter surface area of a circular pile = πDL

Perimeter surface area of a rectangular pile = AL

Skin friction acts on the perimeter surface area of the pile.

7.2 Equations for end bearing capacity in sandy soils

The following equations have been proposed to compute the end bearing capacity of piles in sandy soils.

7.2.1 API method (American Petroleum Institute, 1984)

$$q = N_q \sigma_t'$$

Here, q is end bearing capacity of the pile (units same as σ_v'), σ_t', is effective stress at pile tip (maximum effective stress allowed for the computation is 240 kPa). N_q = 8–12 for loose sand, N_q = 12–40 for medium sand, and N_q = 40 for dense sand.

7.2.2 Martin et al. (1987)

$$q = CN\left(\frac{MN}{\mathrm{m}^2}\right)$$

Here, q is the end bearing capacity of the pile (units same as σ_v'), N is SPT value at pile tip, $C = 0.45$ for pure sand, and $C = 0.35$ for silty sand.

7.2.3 NAVFAC DM 7.2 (1984)

$$q = \sigma_t' \times N_q \times A$$

Here, q, end bearing capacity of the pile (units same as σ_v'); A, cross sectional area of the pile at the tip; and σ_t', effective stress at pile tip.

7.2.4 Bearing capacity factor (N_q)

In Table 7.1, the N_q value is lower in bored piles. This is expected. During pile driving, the soil just below the pile tip would be compacted. Hence it is reasonable to assume a higher N_q value for driven piles.

Note: if water jetting is used, ϕ should be limited to 28 degrees. This is due to the fact that water jets tend to loosen the soil. Hence higher friction angle values are not warranted.

7.3 Equations for skin friction in sandy soils

Numerous techniques have been proposed to compute the skin friction in piles in sandy soils. These different methodologies are briefly discussed in this chapter.

7.3.1 Driven piles

7.3.1.1 McClelland (1974)

McClelland (1974) suggested the following equation:

$$S = \beta \sigma'_{\mathrm{v}} A_{\mathrm{p}}$$

Here, S, skin friction; σ_{v}', effective stress; σ_{v}' changes along with the length of the pile. Hence σ_{v}' at the midpoint of the pile should be taken; A_{p}, perimeter surface area of the pile; $\beta = 0.15–0.35$ for compression; and $\beta = 0.10–0.25$ for tension (for uplift piles).

7.3.1.2 Meyerhoff (1976) (driven piles)

Meyerhoff (1976) suggested following equations for driven piles:

$$S = \beta \sigma'_{\mathrm{v}} A_{\mathrm{p}}$$

Here, S, skin friction of the pile; σ_{v}', effective stress at the midpoint of the pile; $\beta = 0.44$ for $\phi' = 28$ degrees, $\beta = 0.75$ for $\phi' = 35$ degrees, and $\beta = 1.2$ for $\phi' = 37$ degrees.

7.3.1.3 Meyerhoff (1976) (bored piles)

Meyerhoff (1976) suggested the following equations for bored piles:

$$S = \beta \sigma'_{\mathrm{v}} A_{\mathrm{p}}$$

Here, S, skin friction of the pile; σ_{v}', effective stress at the midpoint of the pile; $\beta = 0.10$ for $\phi' = 33$ degrees, $\beta = 0.20$ for $\phi' = 35$ degrees, and $\beta = 0.35$ for $\phi' = 37$ degrees.

Table 7.2 Skin friction angle between pile material and surrounding sandy soils (NAVFAC DM 7.2, 1984)

Pile type	δ
Steel piles	20 degrees
Timber piles	¾ ϕ
Concrete piles	¾ ϕ

7.3.1.4 Kraft and Lyons (1974)

$$S = \beta \sigma'_v A_p$$

Here, S, skin friction of the pile; σ_v', effective stress at the midpoint of the pile; $\beta = C \cdot \tan(\phi' - 5)$; $C = 0.7$ for compression, and $C = 0.5$ for tension (uplift piles).

7.3.1.5 NAVFAC DM 7.2 (1984)

$$S = K\sigma'_v \times \tan\delta \times A_p$$

Here, S, skin friction of the pile; and σ_v', effective stress at the midpoint of the pile.

7.3.2 Pile skin friction angle (δ)

In Table 7.2, δ is the skin friction angle between pile materials and surrounding sandy soils. Usually smooth surfaces tend to have lesser skin friction compared to rough surfaces.

7.3.3 Lateral earth pressure coefficient (K)

In Table 7.3, lateral earth pressure coefficient is less in uplift piles compared to regular piles. Tapered piles tend to have the highest K value.

Table 7.3 Lateral earth pressure coefficient in uplift and regular piles (NAVFAC DM 7.2, 1984)

Pile type	K (piles under compression)	K (piles under tension: uplift piles)
Driven H-piles	0.5–1.0	0.3–0.5
Driven displacement piles (round and square)	1.0–1.5	0.6–1.0
Driven displacement tapered piles	1.5–2.0	1.0–1.3
Driven jetted piles	0.4–0.9	0.3–0.6
Bored piles (less than 24″ diameter)	0.7	0.4

7.3.4 Average K method

Earth pressure coefficient K can be averaged from K_a, K_p, and K_0.

$$K = \frac{(K_0 + K_a + K_p)}{3}$$

Equations for K_0, K_a, and K_p are

$K_0 = 1 - \sin \varphi$
$K_a = 1 - \tan^2(45 - \varphi/2)$
$K_p = 1 + \tan^2(45 + \varphi/2)$

7.4 Design examples

Design example 1

A single pile in uniform sand layer (no groundwater present): 0.5-m-diameter, 10-m-long round steel pipe pile is driven into a sandy soil stratum as shown in Fig. 7.5. Compute the ultimate bearing capacity of the pile.

Step 1: compute the end bearing capacity.

$$\boxed{Q_{\text{ultimate}} = \underbrace{\left[\sigma'_t \times N_q \times A\right]}_{\text{End Bearing Term}} + \underbrace{\left[K \cdot \sigma'_p \times \tan\delta \times (\pi \times d) \times L\right]}_{\text{Skin Friction Term}}}$$

End bearing term = $\sigma'_t \times N_q \times A$ (σ'_t, effective stress at the tip of the pile)

$\sigma'_t = \gamma \cdot$ depth to the tip of the pile = 17.3 × 10 = 173 kN/m^2.

Find "N_q" using Table 7.1.

For a friction angle of 30, N_q = 21 for driven piles.

$$\text{End bearing capacity} = \sigma'_t \times N_q \times A$$
$$= 173 \times 21 \times \left(\frac{\pi d^2}{4}\right) = 713.3\,\text{kN}$$

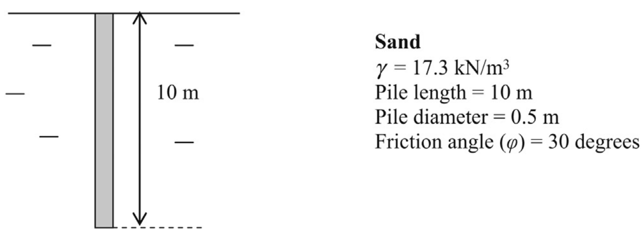


Figure 7.5 Pile information.

Step 2: computation of the skin friction.

Skin friction term = $K \cdot \sigma'_p \times \tan\delta \times (\pi \times d) \times L$.

Obtain the K Value

From Table 7.1, for driven round piles K value lies between 1.0 and 1.5. Hence, assume $K = 1.25$.

Obtain the σ'_p (effective stress at the perimeter of the pile)

The effective stress along the perimeter of the pile varies with the depth. Hence, obtain the σ'_p value at the midpoint of the pile.

The pile is 10 m long. Hence, use the effective stress at 5 m below the ground surface.

σ'_p (midpoint) = $5 \times \gamma = 5 \times 17.3 = 86.5$ kN/m^2.

Obtain the skin friction angle (δ)

From Table 7.2, the skin friction angle for steel piles is 20 degrees.

Find the skin friction of the pile

$$\begin{aligned}\text{Skin friction} &= K\sigma'_p \times \tan\delta \times (\pi \times d) \times L \\ &= 1.25 \times 86.5 \times (\tan 20) \times (\pi \times 0.5) \times 10 \\ &= 618.2\ \text{kN}\end{aligned}$$

Step 3: compute the ultimate bearing capacity of the pile.

$Q_{ultimate}$, ultimate bearing capacity of the pile; $Q_{ultimate}$, end bearing capacity + skin friction; and $Q_{ultimate}$, 713.3 + 618.2 = 1331.4 kN.

Assume a factor of safety of 3.0.

Hence, allowable bearing capacity of the pile = $Q_{ultimate}$/FOS = 1331.4/3.0

Allowable pile capacity = 443.8 kN.

Note: 1 kN = 0.225 Kips. Hence, allowable capacity of the pile is 99.8 Kips.

Design example 2

A single pile in uniform sand layer (groundwater present): a 0.5-m-diameter, 10-m-long round concrete pile is driven into a sandy soil stratum as shown in Fig. 7.6. Groundwater is located 3 ft. below the surface. Compute the ultimate bearing capacity of the pile.

Step 1: compute the end bearing capacity.

$$Q_{ultimate} = \underbrace{\left(\sigma'_t \times N_q \times A\right)}_{\text{End bearing term}} + \underbrace{\left(K\sigma'_v \times \tan\delta \times (\pi \times d) \times L\right)}_{\text{Skin friction term}}$$

End bearing term = $\sigma'_t \times N_q \times A$ (σ'_t, effective stress at the tip of the pile); $\sigma'_t = 17.3 \times 10 = 173$ kN/m^2.

Find N_q using Table 7.1.

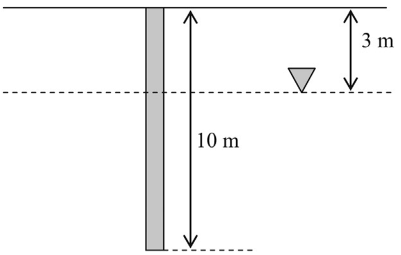


Figure 7.6 Pile information.

For a friction angle of 30, $N_q = 21$ for driven piles.

$$\text{End bearing capacity} = \sigma_t' \times N_q \times A = 173 \times 21 \times \frac{\pi d^2}{4} = 713.3\,\text{kN}.$$

Step 2: computation of the skin friction.

Skin friction term = $K\sigma'_v \times \tan\delta \times (\pi \times d) \times L$.

Obtain the K value

From Table 7.3, for driven round piles, K value lies between 1.0 and 1.5. Hence, assume $K = 1.25$.

Obtain the σ_v' (effective stress at the perimeter of the pile)

The effective stress along the perimeter of the pile varies with the depth. Hence, obtain the σ_p' value at the midpoint of the pile.

The pile is 10 m long. Hence, use the effective stress at 5 m below the ground surface.

σ'_p (midpoint) = $5 \times \gamma = 5 \times 17.3 = 86.5$ kN/m^2.

Obtain the skin friction angle (δ)

From Table 7.2, the skin friction angle for steel piles is 20 degrees.

Find the skin friction of the pile

$$\begin{aligned}\text{Skin friction} &= K\sigma_v' \times \tan\delta \times (\pi \times d) \times L \\ &= 1.25 \times 86.5 \times (\tan 20) \times (\pi \times 0.5) \times 10 \\ &= 618.2\ \text{kN}.\end{aligned}$$

Step 3: compute the ultimate bearing capacity of the pile.

$Q_{ultimate}$, ultimate bearing capacity of the pile, $Q_{ultimate}$, end bearing capacity + skin friction; and $Q_{ultimate}$, 713.3 + 618.2 = 1331.4 kN.

Assume a factor of safety of 3.0.

Hence, allowable bearing capacity of the pile = $Q_{ultimate}$/FOS = 1331.4/3.0

Allowable pile capacity = 443.8 kN.

Note: 1 kN is equal to 0.225 Kips. Hence, allowable capacity of the pile is 99.8 Kips.

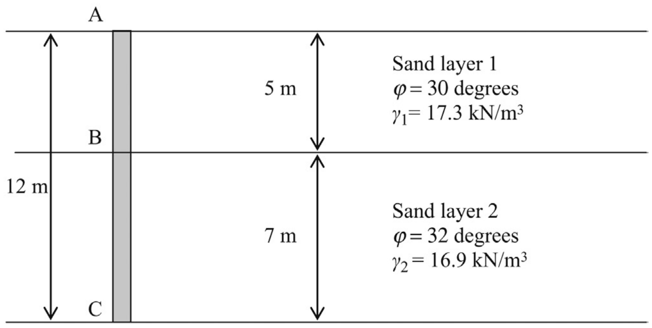


Figure 7.7 Soil strata.

Design example 3

Multiple sand layers with no groundwater present: a 0.5-m-diameter, 12-m-long round concrete pile is driven into a sandy soil stratum as shown in Fig. 7.7. Compute the ultimate bearing capacity of the pile.

Solution

Step 1: Compute the end bearing capacity.

$$Q_{\text{ultimate}} = \underbrace{\left(\sigma_t' \times N_q \times A\right)}_{\text{End bearing term}} + \underbrace{\left(K\,\sigma_v' \times \tan\delta \times A_p\right)}_{\text{Skin friction term}}$$

End bearing term = $\sigma'_t \times N_q \times A$ (σ'_t, effective stress at the tip of the pile)

$\sigma'_t = \gamma_1 \cdot 5 + \gamma_2 \cdot 7$

$\sigma'_t = 17.3 \times 5 + 16.9 \times 7 = 204.8$ kN/m^2.

Find "N_q" using Table 7.1.

For a friction angle of 32, $N_q = 29$ for driven piles.

The φ value of the bottom sand layer is used to find N_q, since the tip of the pile lies on the bottom sand layer.

$$\begin{aligned}\text{End bearing capacity} &= \sigma_t' \times N_q \times A \\ &= 204.8 \times 29 \times \left(\frac{\pi d^2}{4}\right) \\ &= 1166.2 \text{ kN}\end{aligned}$$

Step 2: computation of the skin friction.

The skin friction of the pile needs to be done in two parts:

- Skin friction of the pile portion in sand layer 1 (A to B).
- Skin friction of the pile portion in sand layer 2 (B to C).

Compute the skin friction of the pile portion in sand layer 1 (A to B)

Skin friction term = $K \cdot \sigma'_p \times \tan\delta \times A_p$; $A_p = (\pi \times d) \times L$.

Obtain the K value

From Table 7.3, for driven round piles the K value lies between 1.0 and 1.5.

Hence, assume $K = 1.25$.

Obtain the σ'_v (effective stress at the perimeter of the pile).

Obtain the σ'_v value at the midpoint of the pile in sand layer 1.

σ'_v (midpoint) = $2.5 \times (\gamma_1) = 2.5 \times 17.3$ kN/m^2.
σ'_v (midpoint) = 43.3 kN/m^2.

Obtain the skin friction angle (δ)

From Table 7.2, the skin friction angle (δ) for concrete piles is ¾ φ.

δ = ¾ × 30 degrees = 22.5 degrees (friction angle of layer 1 is 30 degrees).

Skin friction in sand layer 1 = $K\sigma'_p \times \tan\delta \times (\pi \times d) \times L$
= $1.25 \times 43.3 \times (\tan 22.5) \times (\pi \times 0.5) \times 5$
= 176.1 kN.

Step 3: find the skin friction of the pile portion in sand layer 2 (B to C).

Skin friction term = $K\sigma'_v \times \tan\delta \times (\pi \times d) \times L$.
Obtain the σ'_v (effective stress at the perimeter of the pile).
Obtain the σ'_v value at the midpoint of the pile in sand layer 2.
σ'_v (midpoint) = $5 \times \gamma_1 + 3.5 \times \gamma_2$
σ'_v (midpoint) = $5 \times 17.3 + 3.5 \times 16.9$ kN/m^2 = 145.7 kN/m^2.

Obtain the skin friction angle (δ)

From Table 7.2, the skin friction angle (δ) for concrete piles is ¾ φ.

δ = ¾ × 32 degrees = 24 degrees (friction angle of layer 2 is 32 degrees).

Skin friction in sand layer 2 = $K\sigma_p' \times \tan\delta \times (\pi \times d) \times L$
= $1.25 \times 145.7 \times (\tan 24) \times (\pi \times 0.5) \times 7$
= 891.6 kN.

$P_{ultimate}$ = end bearing capacity + skin friction in layer 1 + skin friction in layer 2.

- End bearing capacity = 1166.2 kN
- Skin friction in sand layer 1 = 176.1 kN
- Skin friction in sand layer 2 = 891.6 kN

$P_{ultimate}$ = 2233.9 kN.

One can see that the bulk of the pile capacity comes from the end bearing. Then, the skin friction in layer 2 (bottom layer). Skin friction in the top layer is very small. One of the reasons for this is that the effective stress acting on the perimeter of the pile is very low in the top layer. This is due to the fact that effective stress is directly related to depth.

Hence, allowable bearing capacity of the pile = $P_{ultimate}$/FOS = 2233.9/3.0

Allowable pile capacity = 744.6 kN.

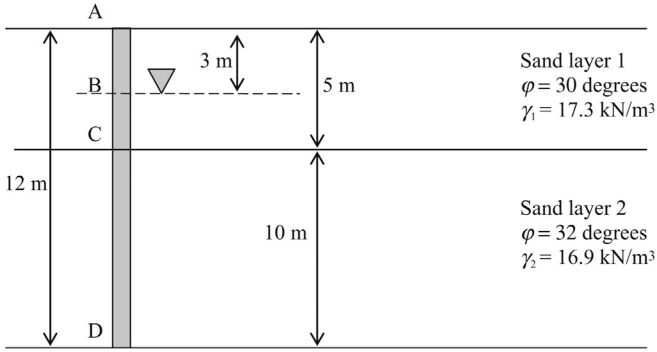


Figure 7.8 Soil strata information.

Design example 4

Multiple sand layers with groundwater present a 0.5-m diameter, 15-m long round concrete pile that is driven into a sandy soil stratum, as shown in Fig. 7.8. Groundwater is 3 m below the surface. Compute the ultimate bearing capacity of the pile.

Solution

Step 1: compute the end bearing capacity.

$$Q_{\text{ultimate}} = \underbrace{\left(\sigma'_t \times N_q \times A\right)}_{\text{End bearing term}} + \underbrace{\left(K\sigma'_v \times \tan\delta \times A_p\right)}_{\text{Skin friction term}}$$

End bearing term $= \sigma'_t \times N_q \times A$ (σ'_t = effective stress at the tip of the pile).
$\sigma'_t = \gamma_1 \cdot 3 + (\gamma_1 - \gamma_w) \cdot 2 + (\gamma_2 - \gamma_w) \cdot 10$
$\sigma'_t = 17.3 \times 3 + (17.3 - 9.8) \times 2 + (16.9 - 9.8) \cdot 10 = 137.9\ \text{kN/m}^2$.

Find N_q using Table 7.1.

For a friction angle of 32, $N_q = 29$ for driven piles.

The φ value of the bottom sand layer is used to find N_q, since the tip of the pile lies on the bottom sand layer.

End bearing capacity $= \sigma'_t \times N_q \times A$

$$= 137.9 \times 29 \times (\pi d^2/4) = 785.2\ \text{kN}$$

Step 2: computation of the skin friction.

The skin friction of the pile needs to be done in three parts:

- Skin friction of the pile portion in sand layer 1 above groundwater (A to B).
- Skin friction of the pile portion in sand layer 1 below groundwater (B to C).
- Skin friction of the pile portion in sand layer 2 below groundwater (C to D).

Step 3: compute the skin friction of the pile portion in sand layer 1 above groundwater (A to B).

Skin friction term $= K \cdot \sigma'_v \times \tan\delta \times (\pi \times d) \times L$.

Obtain the K value

From Table 7.3, for driven round piles, K value lies between 1.0 and 1.5.

Hence, assume $K = 1.25$.

Obtain the σ'_v (effective stress at the perimeter of the pile)

Obtain the σ'_p value at the midpoint of the pile in sand layer 1 above groundwater (A to B).

σ'_v (midpoint) = $1.5 \times (\gamma_1) = 1.5 \times 17.3$ kN/m^2.
σ'_v (midpoint) = 26 kN/m^2.

Obtain the skin friction angle (δ)

From Table 7.2, the skin friction angle (δ) for concrete piles is $\frac{3}{4}\varphi$.

$\delta = \frac{3}{4} \times 30$ degrees = 22.5 degrees (friction angle of layer 1 is 30 degrees).
Skin friction in sand layer 1 (A to B)
$= K \cdot \sigma'_v \times \tan\delta \times (\pi \times d) \times L$
$= 1.25 \times 26 \times (\tan 22.5) \times (\pi \times 0.5) \times 3$
= 63.4 kN.

Step 4: find the skin friction of the pile portion in sand layer 1 below groundwater (B to C).

Skin friction term = $K\sigma'_v \times \tan\delta \times (\pi \times d) \times L$.
Obtain the σ'_v (effective stress at the perimeter of the pile):
σ'_v (midpoint) = $3 \times \gamma_1 + 1.0\,(\gamma_1 - \gamma_w)$
σ'_v (midpoint) = $3 \times 17.3 + 1.0 \times (17.3 - 9.8)$ kN/m^2
= 59.4 kN/m^2.
Skin friction in sand layer 1 (B to C) = $K\sigma_p' \times \tan\delta \times (\pi \times d) \times L$
$= 1.25 \times 59.4 \times (\tan 22.5) \times (\pi \times 0.5) \times 2$
= 96.6 kN.

Step 5: find the skin friction in sand layer 2 below groundwater (C to D).

Skin friction term = $K \cdot \sigma'_v \times \tan\delta \times (\pi \times d) \times L$.
Obtain the σ'_p (effective stress at the midpoint of the pile):
σ'_v (midpoint) = $3 \times \gamma_1 + 2.0\,(\gamma_1 - \gamma_w) + 5 \times (\gamma_2 - \gamma_w)$
σ'_v (midpoint) = $3 \times 17.3 + 2.0 \times (17.3 - 9.8) + 5 \times (16.9 - 9.8)$ kN/m^2
= 102.4 kN/m^2.
From Table 7.2, the skin friction angle (δ) for concrete piles is $\frac{3}{4}\varphi$.
$\delta = \frac{3}{4} \times 32$ degrees = 24 degrees (friction angle of layer 2 is 32 degrees).
Skin friction in sand layer 2 (C to D) = $K\sigma_v' \times \tan\delta \times (\pi \times d) \times L$
$= 1.25 \times 102.4 \times (\tan 24) \times (\pi \times 0.5) \times 10$
= 895.2 kN.

$P_{ultimate}$ = end bearing capacity + skin friction in layer 1 (above GW) + skin friction in layer 1 (below GW) + skin friction in layer 2 (below GW).

End bearing capacity	= 785.2 kN
Skin friction in layer 1 (above GW) (A to B)	= 63.4 kN
Skin friction in layer 1 (below GW) (B to C)	= 96.6 kN
Skin friction in layer 2 (below GW) (C to D)	= 895.2 kN

$P_{ultimate}$ = 1840.4 kN.

- In this case, bulk of the pile capacity comes from end bearing and the skin friction in the bottom layer.

  Hence, allowable bearing capacity of the pile = P_{ultimate}/FOS = 1840.4/3.0.
  Allowable pile capacity = 613.5 kN.

7.5 Parameters that affect end bearing capacity

The following parameters affect the end bearing capacity:

- effective stress at pile tip
- friction angle at pile tip and below (ϕ')
- the dilation angle of soil (ψ)
- shear modulus (G)
- Poisson's ratio (ν)

Most of these parameters have been bundled into the bearing capacity factor (N_q). It is known that friction angle decreases with depth. Hence N_q, which is a function of the friction angle, also would reduce with depth. Variation of other parameters with depth has not been researched thoroughly.

Experimental evidence indicates that increase of end bearing capacity slows down with depth.

Fig. 7.9 by Randolph et al. (1994) is an attempt to formulate end bearing capacity of a pile with regard to relative density (RD) and effective stress.

RD = relative density (see chapter: SPT and Friction Angle to obtain the relative density value using SPT (N) values).

- As per Fig. 7.9, end bearing capacity tapers down with increasing effective stress.

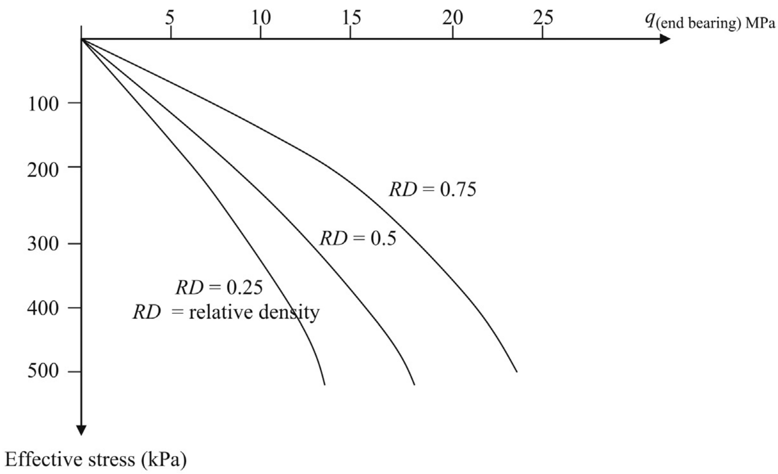


Figure 7.9 Effective stress, end bearing, and relative density (RD, Randolph et al., 1994).

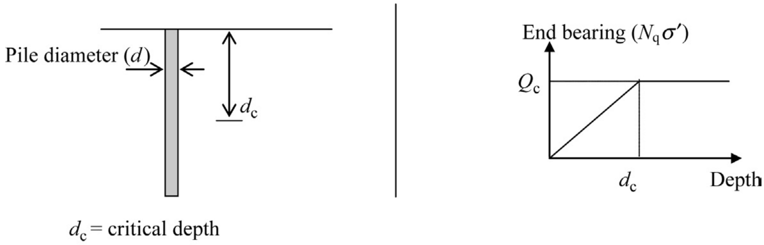


Figure 7.10 Critical depth for end bearing.

7.6 Critical depth for end bearing capacity (sandy soils)

- Pile end bearing capacity in sandy soils is related to effective stress. Experimental data indicate that end bearing capacity does not increase with depth indefinitely.
- Due to lack of a valid theory, engineers use the same critical depth concept adopted for skin friction for end bearing capacity (Fig. 7.10).
- As shown in the figure, the end bearing capacity was assumed to increase till the critical depth. d_c = critical depth; Q_c = end bearing at critical depth; σ'_c = effective stress at critical depth.

The following approximations were assumed for the critical depth:

- critical depth for loose sand = 10*d* (*d* is the pile diameter or the width)
- critical depth for medium dense sand = 15*d*
- critical depth for dense sand = 20*d*

Critical depth for end bearing is the same as the critical depth for skin friction?

Since the critical depth concept is a gross approximation that cannot be supported by experimental evidence, the question is irrelevant. It is clear that there is a connection between end bearing capacity and skin friction since the same soil properties act in both cases such as effective stress, friction angle, and relative density. On the other hand it should be noted that two processes are vastly different in nature.

7.7 Critical depth for skin friction (sandy soils)

Vertical effective stress (σ') increases with depth. Hence, the skin friction should increase with depth indefinitely. In reality, skin friction *will not* increase with depth indefinitely.

It was believed that skin friction would become a constant at a certain depth. This depth was named critical depth (Fig. 7.11).

- As shown in the figure, the skin friction was assumed to increase till the critical depth and then maintain a constant value. d_c = critical depth, S_c = skin friction at critical depth ($K \cdot \sigma'_c \cdot \tan\delta$), and σ'_c = effective stress at critical depth.

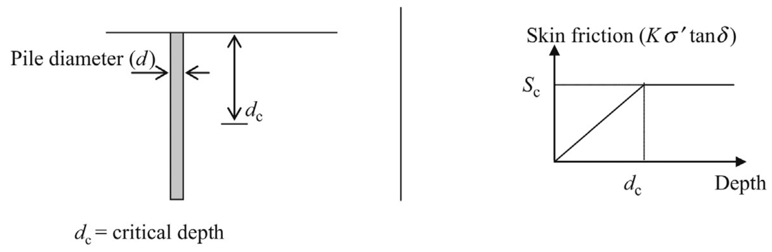


Figure 7.11 Critical depth for skin friction.

The following approximations were assumed for the critical depth:

- critical depth for loose sand = 10*d* (*d* is the pile diameter or the width)
- critical depth for medium dense sand = 15*d*
- critical depth for dense sand = 20*d*

This theory does not explain recent precise pile load test data. According to recent experiments, it is clear that skin friction will not become a constant abruptly as once believed.

7.7.1 Experimental evidence for critical depth

Fig. 7.12 shows typical variation of skin friction with depth in a pile.

- As one can see, the experimental data do not support the old theory with a constant skin friction below the critical depth.
- The skin friction tends to increase with depth and just above the tip of the pile attain its maximum value. Skin friction would drop rapidly after that.
- Skin friction does not increase linearly with depth as once believed.
- It should be noted here that no satisfactory theory exists at the present to explain the field data.
- Due to lack of a better theory, the critical depth theory of the past is still being used by engineers.

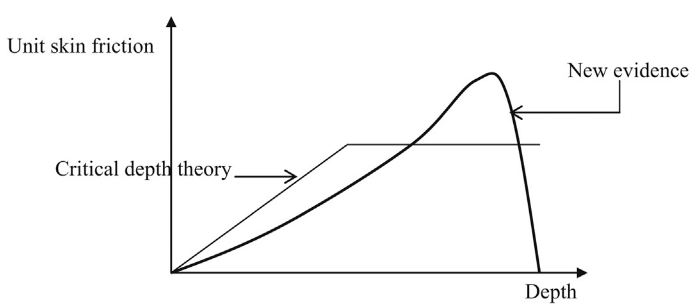


Figure 7.12 Depth versus unit skin friction (Randolph et al., 1994).

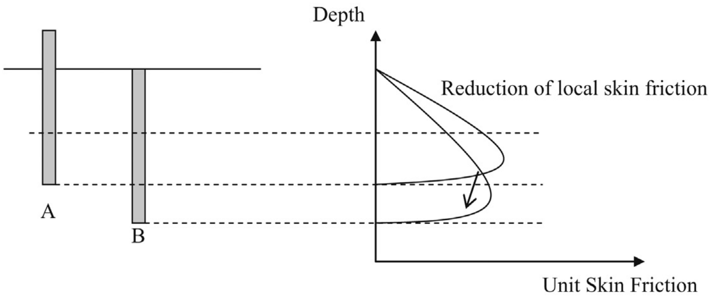


Figure 7.13 Skin friction variation.

7.7.2 Reasons for limiting skin friction

The following reasons have been put forward to explain why skin friction does not increase with the depth indefinitely as suggested by the skin friction equation (Unit skin friction = $K \cdot \sigma' \tan\delta$; $\sigma' = \gamma d$):

1. The aforementioned K value is a function of the soil friction angle (ϕ'). Friction angle tends to decrease with depth. Hence, K value decreases with depth (Kulhawy et al., 1983).
2. The aforementioned skin friction equation does not hold true at high stress levels due to readjustment of sand particles.
3. Reduction of local shaft friction with increasing pile depth (Fig. 7.13) (Randolph et al., 1994).

   Assume a pile was driven to a depth of 10 ft. and unit skin friction was measured at a depth of 5 ft. Then assume the pile was driven further to a depth of 15 ft. and unit skin friction was measured at the same depth of 5 ft. It has been reported that unit skin friction at 5 ft. is less in the second case (Fig. 7.13).

According to Fig. 7.13, local skin friction reduces when the pile is driven further into the ground.

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Design of driven piles

8

8.1 Pile design in sandy soils (dynamic analysis)

Dynamic analysis is another technique to evaluate the capacity of a pile. Presently many engineers prefer to use static analysis. Dynamic analysis is based on pile driving data. There are many dynamic equations available. The accuracy of these equations has been debated. The most popular formulae are:

- Engineering news formula
- Danish formula
- Hiley's formula (more complicated than the first two formulas)

Dynamic formulae are better suited for cohesionless soils. These comments were made by Peck et al.

> *All dynamic analysis formulas are unsound because their neglect of the time dependant aspects of the dynamic phenomena. Hence, except where well-supported empirical correlations under a given set of physical and geological conditions are available, the use of formulas apparently superior to the Engineering News Formula is not justifiable.*

In addition to neglecting the time-dependent aspects, pile dynamic formulas ignore soil parameters and pile type as well.

8.1.1 Engineering news formula

The energy generated by the hammer during free fall = $W_h \times H$. W_h; weight of the hammer; and H; height of fall of the hammer (ft.).

The energy absorbed by the pile = $Q_u \times S$. Q_u, ultimate pile capacity and; S, penetration in feet during the last blow to the pile (normally average penetration during the last five blows, is taken).

Note: the work energy is defined as (force × distance).

During pile driving, the pile has to penetrate by overcoming a force of Q_u (ultimate pile capacity). Hence, work energy is given by $Q_u \times S$.

$$W_h \times H = Q_u \times S.$$

There is elastic compression in the pile and pile cap. These inefficiencies are represented by a constant (C).

Hence, $W_h \times H = Q_u \times (S + C)$; here C is an empirical constant.

$C = 1$ in. (0.083 ft.) for drop hammers and $C = 0.1$ in. (0.0083 ft.) for air hammers.

$$\text{Engineering news formula} \rightarrow Q_u = \frac{W_h \times H}{(S + C)}$$

- Units of H, S, and C should be the same.

8.1.2 Design example

A timber pile was driven by an air hammer weighing 2 tons with a free fall of 3 ft. The average penetration (S) of the last five blows was found to be 0.2 in. Use the Engineering news formula to obtain the ultimate pile capacity (Q_u).

$$\text{Engineering news formula} \rightarrow Q_u = \frac{W_h \times H}{(S + C)}$$

Convert all units to tons and inches. The 'C' value for steam hammers is 0.1 in.
$W_h = 2$ tons, $H = 3 \times 12$ in., $S = 0.2$ in., and $C = 0.1$ in.

$$Q_u = \frac{2 \times (3 \times 12)}{(0.2 + 0.1)} = 240 \text{ tons}$$

Q (allowable) = 240/6 = 40 tons; normally a FOS of 6 is used.

8.1.3 Danish formula

$$Q_u = \frac{\eta W_h \times H}{(S + 0.5 S_0)} \qquad \eta = \text{efficiency of the hammer}$$

Not all the energy is transferred from the hammer to the pile. A certain amount of energy is lost during the impact. η is used to represent the energy loss.

$$S_0 = \text{elastic compression of the pile} = \frac{2W_h \times H \times L}{A \times E}.$$

Here L, length of the pile; A, cross-sectional area of the pile; and E, modulus of elasticity of the pile.

8.2 Water jetting

Water jetting is the process of providing a high-pressure water jet at the tip of the pile.

Water jetting would ease the pile driving process byloosening the soilcreating a lubricating effect on the side walls of the pile

Water jetting is required in sandy soils.

8.3 Driving stresses

- When a pile is driven, the pile will be subjected to tensile and compressive stresses. The pile needs to be designed to withstand driving stresses (compressive and tensile).
- Driving stresses are calculated using the wave equation.
- Computer programs, such as GRLWEAP by Goble, Rausche, Likins and Associates, can be used to compute the driving stresses.
- Maximum tensile stress and maximum compressive stress during pile driving should be less than the tensile strength and compressive strength of the pile, respectively.
- Driving stresses are dependent on the stroke of the hammer and driving resistance of soil.

Hammer energy = driving resistance × set
Hammer energy = weight of hammer × stroke

8.3.1 Example

For a 12 in. concrete pile, the following maximum tensile stresses are provided by the wave equation for a single acting air hammer with a rating of 26,000 lbs ft. (Fig. 8.1).

According to the aforementioned data, the pile driving operator should not use a stroke of 36 in. (Fig. 8.2).

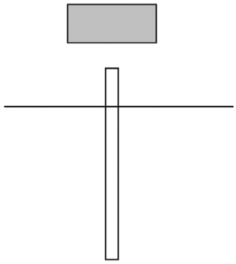


Figure 8.1 Pile set. Set in the distance, pile would penetrate per each blow.

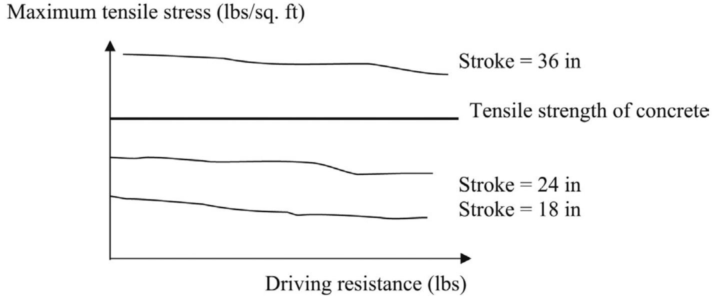


Figure 8.2 Maximum tensile stress versus driving resistance.

8.3.2 Maximum allowable driving stresses

AASHTO recommends the following maximum allowable driving stresses:

- steel piles–0.90 f_y (compression and tension): f_y = steel yield stress
- concrete piles–0.85 f_c' (compression); 0.70 f_y (for steel reinforcements under tension during driving): f_c' = 28 day concrete strength
- prestressed concrete piles (normal environment)–[0.85 $f_c' - f_{pe}$] (compression); $(f_c' + f_{pe})^{1/2}$ (tension): f_{pe} = prestress force
- prestressed concrete piles (corrosive environment): f_{pe} (tension)
- timber piles–3 × allowable working stress (compression); 3 × allowable working stress (tension)

Note: see the chapter on "Timber Piles" to obtain the allowable working stress for a given species of timber.

8.4 Pile design in clayey soils

8.4.1 Skin friction and end-bearing resistance

Piles generate resistance through two processes:

- end-bearing resistance
- skin friction

When the pile is loaded, the total load will be taken by skin friction at the initial stages (Fig. 8.3).

- Assume the pile shown has an ultimate skin friction capacity of 600 kN and ultimate end bearing capacity of 400 kN.
- Hence total ultimate capacity of the pile is 1000 kN. Assuming a factor of safety of 3.0, design capacity of the pile is 1000/3.0 kN = 333 kN.
- Load versus settlement was recorded for the pile and shown in the Fig. 8.3. According to Fig. 8.3, when the total load was 400 kN, settlements were approximately 2 mm. Pile resistance due to skin friction was 390 kN and end bearing was only 10 kN. Total = 400 kN; skin friction = 390 kN; end bearing = 10 kN; and s = 2 mm.

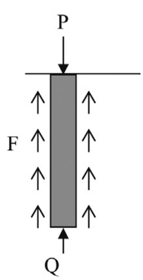


Figure 8.3 Pile in clay soil.

- When "P" was increased to 615 kN, values were: total = 615 kN; skin friction = 600 kN; end bearing = 15 kN; s = 3 mm.
  When total load was increased beyond 615 kN, skin friction did not increase anymore. For example, at a total load of 900 kN, following values were obtained: total = 900 kN; skin friction = 600 kN; end bearing = 300 kN; s = 15 mm. As described, skin friction did not change.
- The pile had reached its ultimate skin friction at 600 kN. Skin friction will not increase anymore. For this pile, the *ultimate skin friction* is developed at a settlement of 3 mm. Any additional load after the first 600 kN, would be taken fully by end bearing.
- Generally, piles generate skin friction at a very slight settlement while full end bearing resistance occurs at a very high settlement.

8.4.2 End bearing versus skin friction (typical example)

- Most of the load was absorbed by the skin friction during early stages of the loading process.
- After 600 kN, resistance due to skin friction did not increase. The ultimate skin friction was achieved at a settlement of 3 mm.
- Ultimate end bearing resistance occurred at a settlement of 25 mm.

Many engineers estimate the total ultimate capacity of the pile and divide it by a factor of safety. In some instances not much attention is paid to the development process of skin friction and end bearing. In the example, given ultimate total resistance was 1000 kN. Assuming a factor of safety value 3, one would obtain an allowable pile capacity of 333 kN (Fig. 8.4).

At 333 kN, almost all the load is taken by skin friction (Fig. 8.4). At this load, end-bearing resistance is negligible. Hence, one could question the logic of dividing the total ultimate resistance by a factor of safety.

8.4.3 Case study: foundation design options

D'Appolonia and Lamb describe construction of several buildings at MIT.

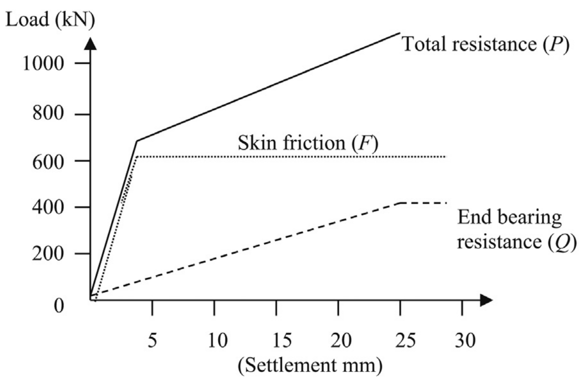
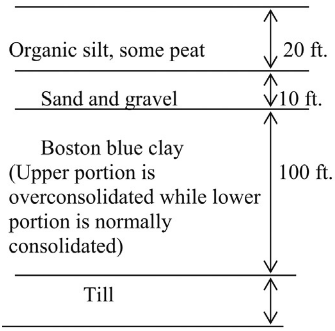


Figure 8.4 Load versus settlement.

8.4.3.1 General soil conditions



- Organic silt was compressible and cannot be used for shallow foundations.
- Sand and gravel was medium dense and can be used for shallow foundations.
- Upper portion of Boston blue clay was overconsolidated. Lower portion was normally consolidated.
- Normally consolidated clays settle appreciably more than over-consolidated clays.
- Till and shale can be used for end bearing piles.

Note: all clays start as normally consolidated clays. Overconsolidated clays had been subjected to higher pressures in the past than existing in situ pressures. This happens mainly due to glacier movement, fill placement, and groundwater change. On the other hand, normally consolidated soils are presently experiencing the largest pressure they have ever experienced. Due to this reason, normally consolidated soils tend to settle more than overconsolidated clays.

8.4.3.2 Foundation option 1

Shallow footing placed on compacted backfill (Fig. 8.5).

- Organic silt was excavated to the sand and gravel layer.
- Compacted backfill was placed and footing was constructed.
- This method was used for light loads.

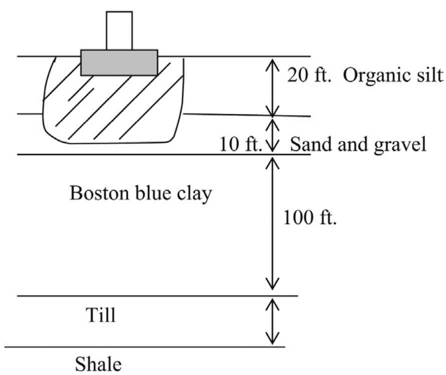


Figure 8.5 Option 1.

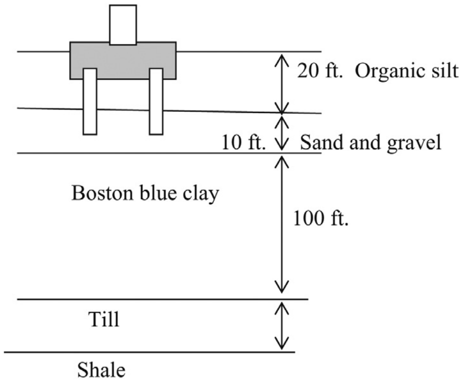


Figure 8.6 Option 2.

- It has been reported that Boston blue clay would settle by more than 4 in. when subjected to a stress of 400 psf. Hence, engineers had to design the footing, so that the stress on Boston blue clay was less than 400 psf.

8.4.3.3 Foundation option 2

Timber piles ending on sand and gravel layer have been shown here (Fig. 8.6).

- Foundations were placed on timber piles ending in sand and gravel layer.
- Engineers had to make sure that underlying clay layer was not stressed excessively due to piles.
- This option was used for light loads.

8.4.3.4 Foundation option 3

Timber piles ending in Boston blue clay layer have been shown here (Fig. 8.7).

- Some foundations were placed on timber piles ending in Boston blue clay.
- This method was found to be a mistake, since huge settlements occurred due to consolidation of the clay layer.

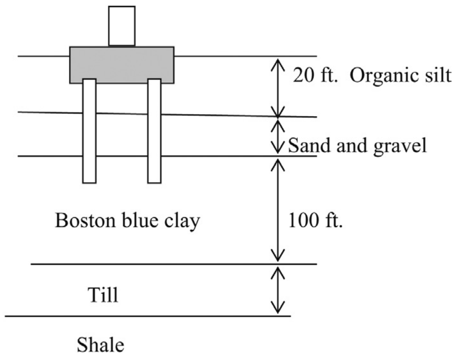


Figure 8.7 Option 3.

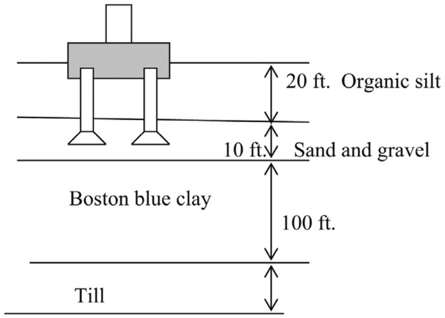


Figure 8.8 Option 4.

8.4.3.5 Foundation option 4

Belled piers ending in sand and gravel have been shown here (Fig. 8.8).

- Foundations were placed on belled piers ending in sand and gravel layer.
- It is not easy to construct belled piers in sandy soils.

8.4.3.6 Foundation option 5

Deep piles ending in till or shale have been shown here (Fig. 8.9).

- Foundations were placed on deep concrete pipe piles ending in till or shale.
- These foundations were used for buildings with 20–30 stories.
- Their performance was found to be excellent. Settlement readings in all buildings were less than 1 in.
- Close-ended concrete-filled pipe piles were used. These piles were selected over H-piles due to their lower cost.
- During pile driving, adjacent buildings underwent slight upheaval. After completion of driving, buildings started to settle (significant settlement in adjacent buildings within 50 ft. of piles was noticed).

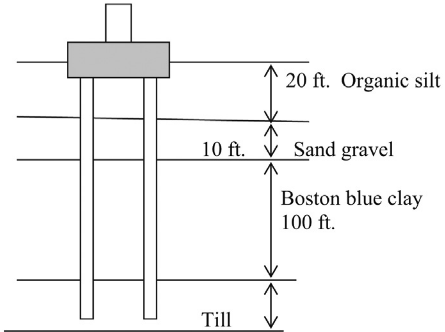


Figure 8.9 Option 5.

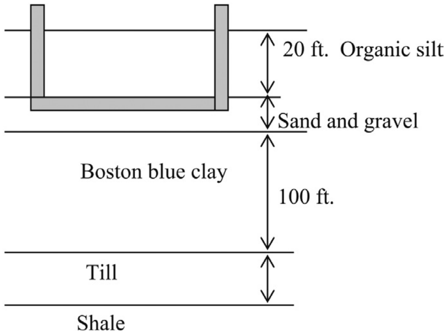


Figure 8.10 Option 6.

- Measured excess pore water pressures exceeded 40 ft. of water column, 15 ft. away from the pile. Excess pore pressures dropped significantly after 10–40 days.
- Due to upheaval of buildings, some piles were preaugered down to 15 ft. prior to driving. Preaugering reduced the generation of excess pore water pressures. In some cases, excess pore water pressures were still unacceptably high.
- Piles in a group were not driven at the same time. After one pile was driven, sufficient time was allowed for the pore water pressures to dissipate prior to driving the next pile.
- Another solution was to drive the pipe piles open end and then cleanout the piles. This was found to be costly and time consuming.

8.4.3.7 Foundation option 6

Floating foundations placed on sand and gravel (rafts) have been shown here (Fig. 8.10).

- Floating foundations were placed on sand and gravel.
- Settlement of floating foundations were larger than deep piles (option 5).
- Settlement of floating foundations varied from 1.0 to 1.5 in.
- Performance of rafts was inferior to deep piles. Average settlements in raft foundations were slightly higher than deep footings.
- There was a concern that excavations of rafts could create settlements in adjacent buildings. This was found out to be a false alarm since adjacent buildings did not undergo any major settlement due to braced excavations constructed for raft foundations.
- When the cost of driving deep piles was equal to the cost of raft foundations, it was desirable to construct raft foundations, since rafts would give a basement.
- Basement might not be available in the piling option, unless it would be specially constructed with additional funds.

8.5 Structural design of piles

8.5.1 Timber pile design

8.5.1.1 Quality of timber piles

The engineer needs to make sure that the timber is of good and sound quality, free from decay, damage during transportation, and insect attack.

8.5.1.2 Knots

Timber piles contain knots and need to be observed. Local codes may provide guidelines on acceptable and nonacceptable knots. Acceptable knot size is dependent upon the type of timber. Typically knots greater than 4 in. are considered to be unacceptable. Knots grouped together (cluster knots) may be undesirable and such piles should be rejected.

8.5.1.3 Holes

Timber piles may contain holes that are larger than the acceptable size. Local codes should be referred to for the acceptable size of holes. Typically holes greater than ½ in. may be considered to be undesirable.

8.5.1.4 Preservatives

No portion of the pile should be exposed above the groundwater unless the pile is treated with a preservative. Untreated piles usually last a very long period of time below groundwater level. Piles are susceptible to decay above groundwater level.

Creosote is widely used and other preservatives, such as chlorophenols and napthelenates are also used to treat piles. Some preservatives are not suitable for piles located in marine environments. The pile designer should investigate the preservative type for the environment in which the piles are located. Timber species also affect the preservative type.

8.5.2 Piles in marine environments

In a study done by Grall (1992) many bacteria, algae, and other unicellular organisms get attached to piles in the ocean. The following is the account by Grall summarized by the author.

> *When a piling is driven to the bottom of the bay, life takes up residence almost immediately. Bacteria, algae and protozoans cover the submerged surface. This "slime" provides a foothold for larger creatures to attach themselves in succession. In the summer ivory barnacles, bryzoans and sun sponges get attached themselves to the pile. Bright patches of algae such as sea lettuce soon arrive, followed by hydroids and bulbous sea squirts. Tube builder amphipods construct tunnels of mud and detritus for protection and for a niche on the crowded pile. Almost every underwater part of the piling is covered with various species each looking for food, shelter and a place to propagate.*

Eventually the pile would fail and drop to the ocean floor taking the animals and the structure it supports as well. Hence, timber piles need to be preserved with extra care when used in marine environments.

Table 8.1 Timber properties

Timber species	Compression parallel to grains (psi)	Bending stress (psi)	Modulus of elasticity (psi)
Southern Pine	1200	2400	1.5×10^6
Douglas Fir	1250	2450	1.5×10^6

8.5.3 Allowable stresses in timber

Allowable stress in timber piles are dependent on the timber species. ASTM D25 provides allowable stresses in various timber species. A few examples are shown in Table 8.1 (Fig. 8.11).

8.5.4 Straightness criteria

Piles are made of timber and may not be as straight as a steel H-pile. Straightness criteria for timber piles should be established by the engineer. Local codes may provide minimum criteria required. The rule of the thumb methods such as drawing a straight line from one end to the other to make sure that the line is within the pile is popular (Tables 8.2 and 8.3).

8.5.5 Allowable working stress for round timber piles

The allowable working stress of round timber piles depends on the wood species. Table 8.4 was provided by AASHTO.

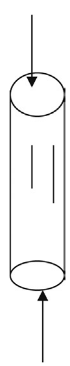


Figure 8.11 Compression parallel to grains.

Table 8.2 Friction angle, SPT "N" values, and relative density

Soil type	SPT (N_{70} value)	Consistency	Friction angle (ϕ)	Relative density (D_r)
Fine sand	1–2	Very loose	26–28	0–0.15
	3–6	Loose	28–30	0.15–0.35
	7–15	Medium	30–33	0.35–0.65
	16–30	Dense	33–38	0.65–0.85
	<30	Very dense	<38	<0.85
Medium sand	2–3	Very loose	27–30	0–0.15
	4–7	Loose	30–32	0.15–0.35
	8–20	Medium	32–36	0.35–0.65
	21–40	Dense	36–42	0.65–0.85
	<40	Very dense	<42	<0.85
Coarse sand	3–6	Very loose	28–30	0–0.15
	5–9	Loose	30–33	0.15–0.35
	10–25	Medium	33–40	0.35–0.65
	26–45	Dense	40–50	0.65–0.85
	<45	Very dense	<50	<0.85

Table 8.3 Soil properties

Soil type	SPT (N_{70} value)	Consistency	Total density
Fine sand	1–2	Very loose	70–90 pcf (11–14 kN/m^3)
	3–6	Loose	90–110 pcf (14–17 kN/m^3)
	7–15	Medium	110–130 pcf (17–20 kN/m^3)
	16–30	Dense	130–140 pcf (20–22 kN/m^3)
	<30	Very dense	<140 pcf < 22 kN/m^3
Medium sand	2–3	Very loose	70–90 pcf (11–14 kN/m^3)
	4–7	Loose	90–110 pcf (14–17 kN/m^3)
	8–20	Medium	110–130 pcf (17–20 kN/m^3)
	21–40	Dense	130–140 pcf (20–22 kN/m^3)
	<40	Very dense	<140 pcf < 22 kN/m^3
Coarse sand	3–6	Very loose	70–90 pcf (11–14 kN/m^3)
	5–9	Loose	90–110 pcf (14–17 kN/m^3)
	10–25	Medium	110–130 pcf (17–20 kN/m^3)
	26–45	Dense	130–140 pcf (20–22 kN/m^3)
	<45	Very dense	<140 pcf < 22 kN/m^3

8.5.6 Timber pile case study: Parakkum building, Colombo, Sri Lanka

The site contained 5 ft. of loose fine sand followed by 15 ft. of soft clay. Fine sand had an average SPT "*N*" value of 3 and the cohesion of soft clay, was found to be 500 psf. Medium dense coarse sand was encountered below the clay layer, which

Table 8.4 **Allowable working stress**

Timber species	Allowable unit working stress compression parallel to grain for normal duration of loading (psi)	Timber species	Allowable unit working stress compression parallel to grain for normal duration of loading (psi)
Ash, white	1200	Hickory	1650
Beech	1300	Larch	1200
Birch	1300	Hard Maple	1300
Chestnut	900	Oak (red and white)	1100
Southern Cypress	1200	Pecan	1650
Tidewater red Cypress	1200	Pine, Lodgepole	800
Douglas Fir, inland	1100	Pine, Norway	850
Douglas Fir coast type	1200	Pine, Southern	1200
Elm, rock	1300	Pine, Southern dense	1400
Elm, soft	850	Poplar, yellow	800
Gum (black and red)	850	Redwood	1100
Eastern Hemlock	800	Spruce, Eastern	850
West Coast Hemlock	1000	Tupelo	850

had an average SPT "N" value of 15. Groundwater was found to be at 8 ft. below the surface. Due to loose fine sand and soft clay, shallow foundations were considered to be risky. The decision was made to drive piles to the medium dense sand layer. Timber piles were selected due to the lesser cost compared to other types of piles. Static analysis was conducted with 8-in.-diameter 30-ft.-long piles.

Assume density of all soils to be 110 pcf.

The main reasons for selecting timber piles:

- no boulders or any other obstructions were found in the overburden soil.
- bearing stratum (medium dense sand) was within the reach of typical length of timber piles.

Piles were driven with a 20,000 lbs ft. hammer till 30 blows per ft.

Conduct a static analysis and the dynamic analysis of the pile capacity (Fig. 8.12).

8.5.6.1 Static analysis

Step 1: find the friction angle from SPT "N" values.
Loose fine sand ($N = 3$), from Table 8.2 $\phi = 28$ degrees.
Coarse sand ($N = 15$), from Table 8.2 $\phi = 35$ degrees.

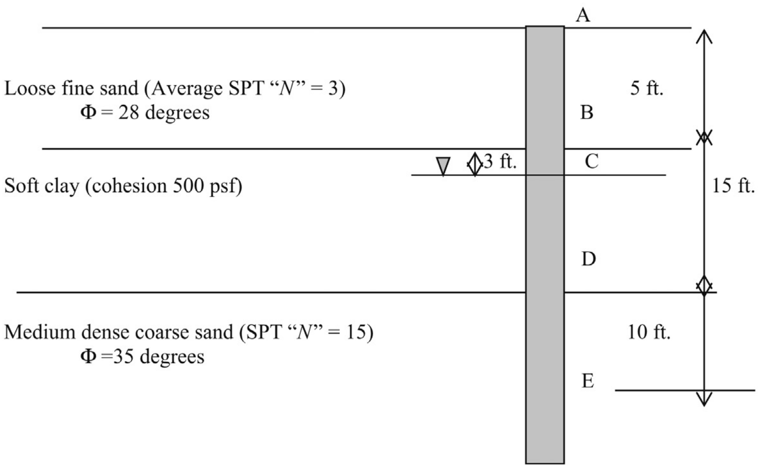


Figure 8.12 Soil strata.

Step 2: find the end-bearing capacity.

Ultimate end-bearing capacity (Q_u) = $q \times N_q \times A$

Here, q = vertical effective stress at the bottom (tip) of the pile,

N_q = bearing capacity factor, and A = cross-sectional area.

Effective stress at bottom of the pile

$$= 110 \times 5 + 110 \times 3 + (110 - 62.4) \times 12 + (110 - 62.4) \times 10$$
$$= 1927 \text{ psf.}$$

Cross-sectional area of the pile = $\pi \times D^2/4 = \pi \times (8/12)^2/4 = 0.35 \text{ ft}^2$.

N_q = 50 (bottom of the pile is located in coarse sand with friction angle of 35 degrees. NAVwFAC DM 7.2 provides N_q = 50).

Ultimate end-bearing capacity = $q \times N_q \times A = 1927 \times 50 \times 0.35 = 33.7$ kips

Step 3: find the skin friction from A to B (loose fine sand):

skin friction from A to B = $K \times q \times \tan \delta \times$ (perimeter surface area)

q = vertical effective stress at the midpoint of the section considered

K = lateral earth pressure coefficient

$q = 110 \times 2.5 = 275$ psf

$K = (K_a + K_0 + K_p)/3$

$K_0 = 1 - \sin \Phi = 1 - \sin 28 = 0.53$

$K_a = \tan^2(45 - \Phi/2) = 0.36$

$K_p = \tan^2(45 + \Phi/2) = 2.77$

$K = (0.36 + 0.53 + 2.77)/3.0 = 1.22$

δ for timber piles = $3/4 \times \Phi$(NAVFAC DM 7.2)

$\delta = 3/4 \times \Phi = 3/4 \times 28 = 21$ degrees.

Skin friction from A to B

$$= K \times q \times \tan \delta \times (\text{perimeter surface area})$$
$$= 1.22 \times 275 \times \tan(21 \text{ degrees}) \times \pi \times (8/12) \times 5 = 1.35 \text{ kips}$$

Step 4: find the skin friction from B to C (soft clay):

skin friction from B to C (soft clay) = $\alpha \times$ cohesion $\times$ (perimeter surface area)

α = adhesion factor = 0.96 (NAVFAC DM 7.2)

cohesion = 500 psf

skin friction from B to C (clay) = $0.96 \times 500 \times \pi \times 8/12 \times 3 = 3.02$ kips

Step 5: find the skin friction from C to D (soft clay):

skin friction from C to D (clay) = $\alpha \times$ cohesion $\times$ (perimeter surface area)

α = adhesion factor = 0.96 (NAVFAC DM 7.2)

cohesion = 500 psf

skin friction from C to D (clay) = $0.96 \times 500 \times \pi \times 8/12 \times 12 = 12.1$ kips

Note: one could have computed the skin friction from B to D directly in this situation since cohesion of saturated clay and unsaturated clay is the same.

Step 6: find the skin friction from D to E (coarse sand):

skin friction from A to B = $K \times q \times \tan\delta \times$ (perimeter surface area)

q = vertical effective stress at the midpoint of the section considered

K = lateral earth pressure coefficient

$q = 110 \times 8 + (110 - 62.4) \times 17 = 1690$ psf

$K = (K_a + K_0 + K_p)/3$

$K_0 = 1 - \sin\Phi = 1 - \sin 35 = 0.43$

$K_a = \tan^2(45 - \Phi/2) = 0.27$

$K_p = \tan^2(45 + \Phi/2) = 3.69$

$K = (0.43 + 0.27 + 3.69)/3.0 = 1.46$

δ for timber piles = $3/4 \times \Phi$ (NAVFAC DM 7.2)

$\delta = 3/4 \times 35 = 3/4 \times 35 = 26$ degrees.

$$\begin{aligned}\text{Skin friction from D to E}\\ &= K \times q \times \tan\delta \times (\text{perimeter surface area})\\ &= 1.46 \times 1690 \times \tan(26\ \text{degrees}) \times \pi \times (8/12) \times 10\\ &= 25.2\ \text{kip.}\end{aligned}$$

Step 7: total capacity:

end-bearing capacity = 33.7 kip

skin friction from A to B = 1.35

skin friction from B to C = 3.02

skin friction from C to D = 12.1

skin friction from D to E = 25.2

total ultimate pile capacity = 75.4 kip.

Total allowable capacity = 25.1 kip (assuming a factor of safety of 3.0).

Step 8: hammer energy is given as 20,000 lbs ft. The piles were driven till 30 blows per foot.

Engineering News record formula:

R = hammer energy/(s + 0.1).

Here, R = ultimate capacity of the pile and s = distance per blow.

30 blows per foot is equal to 1/30 ft. per blow.

s = 1/30 ft. = 0.033.

R = 20,000/(0.033 + 0.1) lbs = 150 kip.

Allowable capacity of the pile (dynamic analysis) = 150/3.0 = 50 kip.

We obtained an allowable capacity of 25 kip from static analysis.

8.5.7 Case study: bridge pile design (timber piles)

8.5.8 Bridge pile design

Timber piles are rarely used for bridge abutments and piers today. This case study gives details of a timber pile project for bridge abutments and piers.

- Center pier is supported on 25 piles as shown in Fig. 8.13.
- Timber piles with diameter = 1 ft., length = 50 ft., pile cap = 15 × 15 ft., pile cap height = 3.5 ft.
- Abutment is supported on seven piles each with diameter of 1 ft. (Fig. 8.14).

8.5.8.1 Soil parameters

- Soil is mostly medium stiff-to-stiff clay. Unconfined compressive strength = 2.3 tsf. Average "*N*" value of the clay soil = 14 (lower values occurred near the surface).

8.5.8.2 Earthquake

- An earthquake of magnitude 6.4 induced significant shaking on the structure. No damage occurred due to the earthquake (peak horizontal and vertical accelerations measured during the earthquake was 0.5*g* and 0.51*g*, respectively).

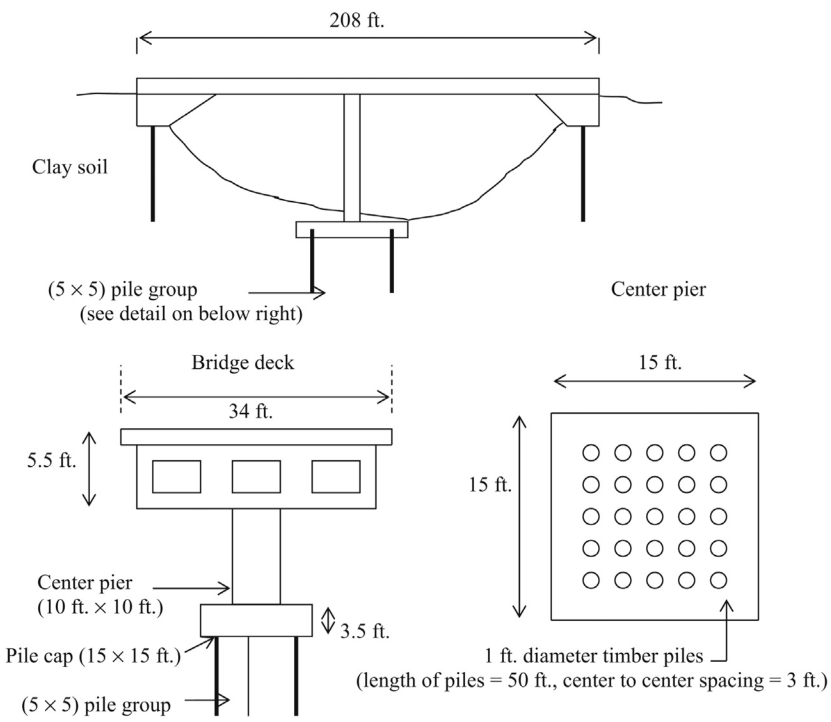


Figure 8.13 Bridge piles.

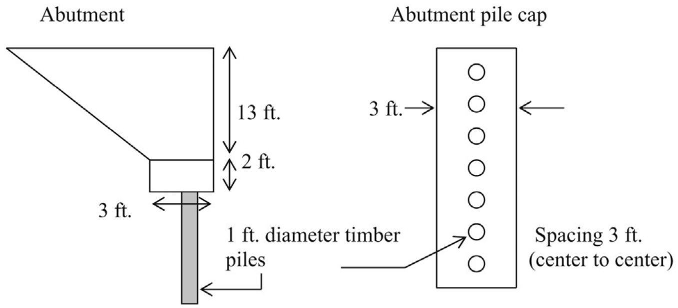


Figure 8.14 Abutment piles.

8.6 Recommended guidelines for pile design

American Society of Civil Engineers(ASCE) has provided the following guidelines for pile foundation design (ASCE, 1997):

8.6.1 Steel piles

- Steel pipe piles should have a minimum yield strength not less than 35,000 psi.
- Structural steel piles should conform to ASTM A36, ASTM A572, and ASTM A588.
- Steel pipe piles should conform to ASTM A252.
- Steel-encased cast-in situ concrete piles should conform to ASTM A252, ASTM A283, ASTM A569, ASTM A570, or ASTM A611.
- The allowable design stress in steel should not be more than 35% of the minimum yield strength of steel.

8.6.2 Minimum dimensions for steel pipe piles

- Pipe piles should have a minimum outer diameter of 8 in.
- Minimum wall thickness of 0.25 in. is recommended for pipe diameters of 14 in or less. Minimum wall thickness of 0.375 in. is recommended for pipe diameters greater than 14 in.
- Steel pipe piles with lesser wall thickness are allowed when the pipe piles are filled with concrete.

8.6.3 Concrete piles

8.6.3.1 Reinforced precast concrete piles

- Diameter or minimum dimension measured through the center should not be less than 8 in.
- Minimum 28 day concrete strength (f_c') = 4,000 psi.
- Minimum yield strength of rebars = 40,000 psi.
- The allowable design stress in concrete should not be more than 1/3 of the minimum concrete strength.
- The allowable design stress in steel should not be more than 40% of the minimum yield strength of steel.

8.6.3.2 Prestressed concrete piles

- Diameter or minimum dimension measured through the center should not be less than 8 in.
- Minimum 28 day concrete strength = 4,000 psi.
- Minimum yield strength of rebars = 40,000 psi.
- The effective prestress should not be less than 700 psi.
- The allowable axial design compressive stress applied to the full cross-section should not exceed 33% of the specified minimum concrete strength minus 27% of the effective prestressed force.

8.6.3.3 Concrete filled shell piles

- Diameter or minimum dimension measured through the center should not be less than 8 in.
- Minimum 28 day concrete strength = 3,000 psi.
- Minimum yield strength of re-bars = 40,000 psi.
- Thin shells less than 0.1 in. thick should not be considered as load-carrying members.
- The allowable design stress in concrete should not be more than 1/3 of the minimum concrete strength.
- The allowable design stress in steel should not be more than 40% of the minimum yield strength of steel.

8.6.3.4 Augered pressure-grouted concrete piles

- Diameter should not be less than 8 in.
- Minimum 28 day concrete strength (f_c') = 3,000 psi.
- Minimum yield strength of rebars = 40,000 psi.
- The allowable design stress in concrete should not be more than 1/3 of the minimum concrete strength.
- The allowable design stress in steel should not be more than 40% of the minimum yield strength of steel.

8.6.3.5 Maximum driving stress

- Maximum driving stress for steel piles = $0.9 f_y$ (for both tension and compression): f_y = yield strength.
- Maximum driving stress for timber piles = $2.5 \times (z)$ (here, z = allowable design strength of timber piles).
- Maximum driving stress for precast concrete piles = $0.85 f_c'$ for compression; $= 3(f_c')^{1/2}$ for tension; f_c' = 28 day concrete strength
- Maximum driving stress for prestressed concrete piles = $(0.85 f_c' - f_{pe})$ for compression (f_{pe} = effective prestress, force).
- Maximum driving stress for prestressed concrete piles = $(3(f_c')^{1/2} + f_{pe})$ for tension (f_{pe} = effective prestress force).

8.7 Uplift forces

- Outside piles in a building are usually subject to uplift forces.
- Total uplift capacity of a pile depends on the skin friction (Fig. 8.15).

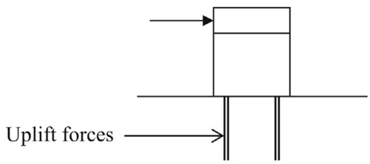


Figure 8.15 Uplift forces.

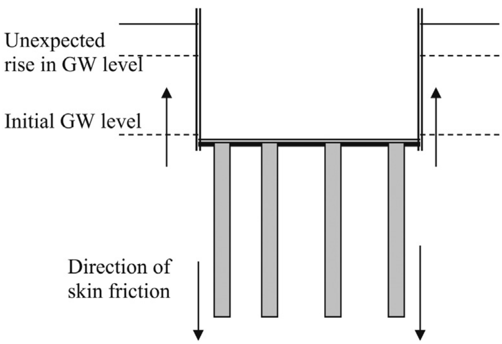


Figure 8.16 Uplift due to groundwater.

Is the skin friction in an uplift pile the same as the skin friction of a loaded pile?

- According to research data provided by Dennis and Olson (1983), there is no significant difference in skin friction between two situations.
- Hence, the table for downward skin friction can be used to compute the skin friction of piles subjected to uplift as well.
- Contrary to Dennis and Olson's contention (1983), many engineers and academics believe that skin friction during uplift is lesser than the skin friction on the pile when the pile is loaded downward. Since there is no consensus on this issue it is suggested to reduce the skin friction obtained using the Olson table by 15%.

8.7.1 Uplift due to high groundwater

- The Archimedes theorem suggests that uplift force due to water is equivalent to the weight of water displaced (Fig. 8.16).
  - As shown in the figure, groundwater level has risen unexpectedly.
  - This would increase the uplift force on the building structure. If the weight of the building is less than the uplift force due to buoyancy, then piles would be called in to resist the uplift force due to buoyancy.
- If the weight of the building is less than the buoyant forces acting upward, then the skin friction on piles would be facing down. The piles are acting as uplift piles.

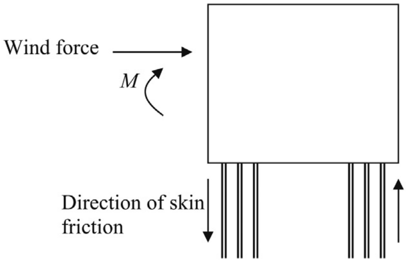


Figure 8.17 Uplift due to wind.

8.7.2 Uplift forces due to wind

- AASHTO recommends only 1/3 of the frictional resistance obtained using static equations for uplift piles.
- Uplift load test procedure can be found in ASTM D-3689 (Fig. 8.17).
- Wind forces would generate a moment on the building.
- Piles on the side of the wind, would be under uplift forces.

8.8 Pile design in expansive soil

Expansive soils swell when water is introduced. Such soils could create major problems for shallow foundations and pile foundations (Figs. 8.18 and 8.19).

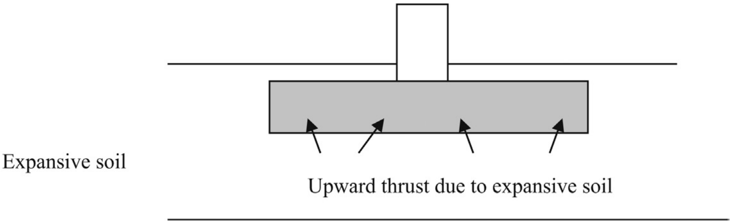


Figure 8.18 Shallow foundation subjected to upward thrust.

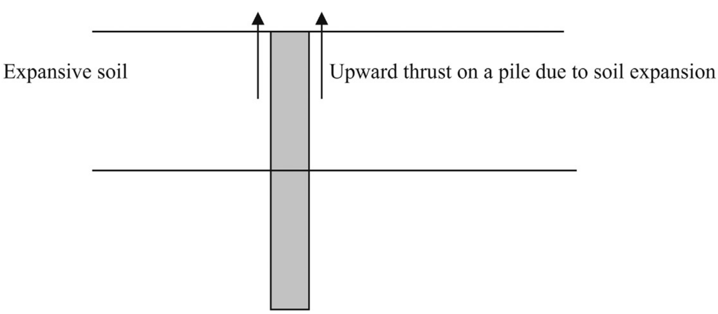


Figure 8.19 Piles in expansive soil.

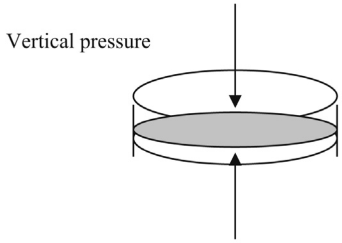


Figure 8.20 Expansion index.

Expansive soils are present in many countries and in some cases, large financial losses have occurred due to their action.

8.8.1 Identification of expansive soils

Expansive soils are mostly clays of silty clays. Not all clays can be considered as expansive soils. If the design engineer suspects the presence of expansive soil, the engineer should obtain the expansive index of the soil.

Expansive index test is described in ASTM D 4829, "Standard Test Method for Expansion Index of Soils." In this test, a soil specimen is compacted into a metal ring so that the degree of saturation is between 40% and 60%. The specimen is placed in a consolidometer and a vertical pressure of 1 psi is applied to the specimen. Next the specimen is inundated with distilled water and the deformation of the specimen is recorded for 24 h. The swell or the expansion of the soil volume is computed (Fig. 8.20 Table 8.5).

8.8.2 Pile design options

The design engineer should identify the expansive soil by conducting expansion index tests. Local codes would provide design guidelines under expansive soil conditions. Typically no action is necessary if the expansion index is less than 20%. If expansion index is between 20% and 50%, the design engineer could ignore the skin friction

Table 8.5 Expansive index

Expansive index	General guidelines
0–20	No special design is needed
20–50	Design engineer should consider effects of expansive soil
50–90	Special design methods to counter expansive soil is needed
90–130	Special design methods to counter expansive soil is needed
>130	Special design methods to counter expansive soil is needed

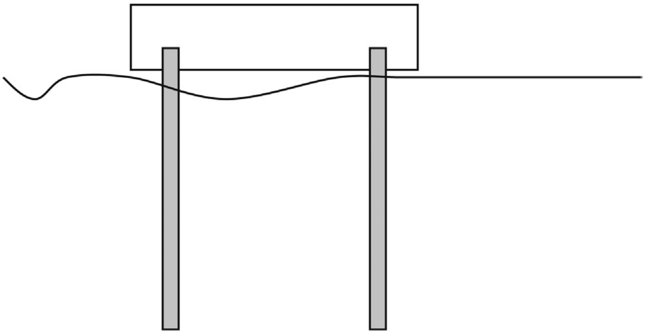


Figure 8.21 Space between pile caps.

developed in expansive soils. If the expansive index is greater than 50%, the pile should be designed against uplift due to expansive soil.

Solutions for expansive soil conditions:

- *excavate and remove expansive soils*: this option is possible only if the expansive soil is limited to a small area of the site.
- *design against the uplift due to expansive soils*: the pile should be able to withstand the uplift caused by expansive soils. The pile should be embedded deep in the soil so that the uplift forces due to expansive soils will be resisted.
- *ignore the positive skin friction in expansive soil section*: the design engineer should ignore any positive skin friction in expansive soils.

8.8.3 Pile caps

Space between pile caps and the expansive soil strata should be provided for the soil underneath to expand (Fig. 8.21).

8.9 Open-ended pipe pile design: semiempirical approach

Open-ended pipe piles are driven in order to reduce driving stresses. During driving, a soil plug would develop inside the pipe pile (Fig. 8.22).

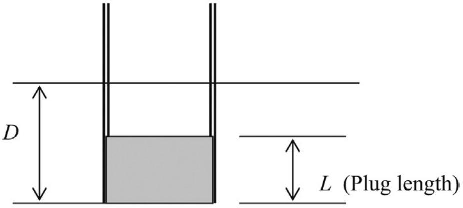


Figure 8.22 Open end pipe piles.

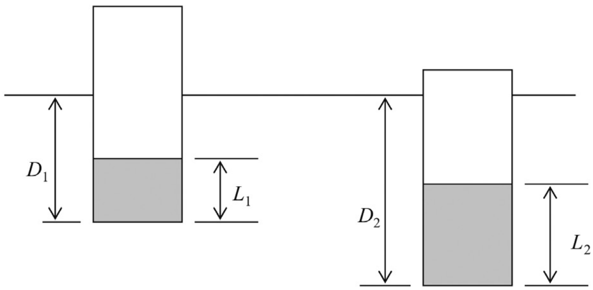


Figure 8.23 Incremental filling ratio.

8.9.1 Plug ratio

Soil plug characteristics strongly affect the bearing capacity of open-ended piles.
Plug ratio = L/D

8.9.2 Incremental filling ratio

Incremental Filling Ratio (IFR) is defined as the increase of the plug length with respect to increase of depth (Fig. 8.23).

$$IFR\% = (L_2 - L_1)/(D_2 - D_1) \times 100$$

- If $L_1 = L_2$ then $IFR = 0$. In this case, soil plug length did not change due to further driving. This happens when the pile is driven through a *soft* soil strata.
- If $L_2 - L_1 = D_2 - D_1$, then $IFR = 1$. In this case, change in plug length is equal to the change in depth (When the pile is driven 1 ft., soil plug length increases by 1 ft.). This happens when the pile is driven through a hard soil stratum.

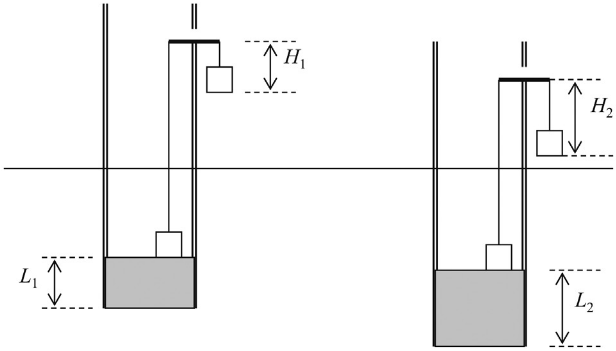


Figure 8.24 Measurement of IFR.

8.9.2.1 Measurement of IFR

- A hole is made through the pile as shown in Fig. 8.24 and two weights are placed.
- It is clear that $H_2 - H_1 = L_2 - L_1$. Hence, by measuring H_1 and H_2, it is possible to compute IFR.

8.9.3 Correlation between PLR and IFR

PLR = Plug length ratio = L/D

$IFR\% = (L_2 - L_1)/(D_2 - D_1) \times 100$

Correlation → IFR% = 109 × PLR − 22 (Kyuho and Salgado, 2003).

8.9.4 End-bearing capacity of open-ended piles in sandy soils

The following equation was proposed by Kyuho and Salgado (2003) to compute the end-bearing capacity of open-ended pipe piles.

$$\frac{Q_\mathrm{b}}{\alpha\sigma'_\mathrm{h}} = 326 - 295 \times \frac{IFR\%}{100}$$

Here Q_b, ultimate end-bearing capacity (same units as effective stress); α, 1.0 for dense sands = 0.6 for medium sands, and = 0.25 for loose sands; and σ'_h, horizontal effective stress = $K_0 \times \sigma'_\mathrm{v}$ (K_0 = earth pressure coefficient at rest = $(1 - \sin\phi')$ and σ'_v = vertical effective stress).

8.9.5 Skin friction of open-ended pipe piles in sandy soils

The following equation was proposed by Kyuho and Salgado (2003) to compute the skin friction of open-ended pipe piles.

$$f\,(\text{unit skin friction}) = (7.2 - 4.8 \times PLR) \times (K_0\sigma'_v \tan\delta)\beta$$

where f = unit skin friction (units same as σ'_v), K_0 = lateral earth pressure coefficient = $(1 - \sin\phi')$, σ'_v = vertical effective stress, δ = friction angle between pile and soil, β = function of relative density (1.0 for dense sands, 0.4 for medium sands, and 0.22 for loose sands).

8.9.5.1 Prediction of plugging

It is important to predict the possibility of plugging during pile driving. Plugging of piles is dependent on density of soil. Sands with high relative density (D_r) would have a tendency to plug than sands with low relative density (Table 8.6).

Example: if the relative density (D_r) of the soil at pile tip is 60%, then find the minimum internal diameter required to avoid plugging.

Solution: 0.6 m (Table 8.6).

Table 8.6 **Relative density and plugging (Jardine and Chow, 1996)**

Relative density (D_r) at pile tip	40	50	60	70	80	90	100
Internal diameter of the pile required for zero plugging (m)	0.2	0.4	0.6	0.8	1.0	1.2	1.4

8.10 Case study 1: friction piles

Most engineers are comfortable in recommending end-bearing piles than friction piles. This case study gives details of a project that included different types of friction piles.

8.10.1 Project description

The task was to design a bridge over Maskinonge River in Canada. Two abutments and four piers were deemed necessary for the bridge (Blanchet et al., 1980) (Fig. 8.25).

8.10.2 Soil condition at the site

The top layers of soil were mostly sand, silt, and clayey silt. Below the top layers of soil, a very thick (175 ft.) layer of silty clay was encountered. Below the silty clay layer, glacial till and shale bedrock were encountered. It was clear that end-bearing piles would be very costly for this site. If one were to design end-bearing piles they would be more than 220 ft. in length (Fig. 8.26).

The trapezoid shows the distribution of undrained shear strength of the clay soil layer. Shear strength of the silty clay layer gradually increases from 1000 to 4000 psf.

The decision was made to design friction piles for this site.

8.10.3 Pile types considered

- tapered timber piles
- precast concrete piles (Herkules H-420, 2 segments)
- steel pipe piles (wall thickness 6.35 mm; Table 8.7)
- Precast concrete piles with two different lengths were tested (78 and 120 ft.).
- From load test data, it is clear that timber piles had the highest load carrying capacity per foot of pile. This is mainly due to the taper of timber piles (Table 8.7).

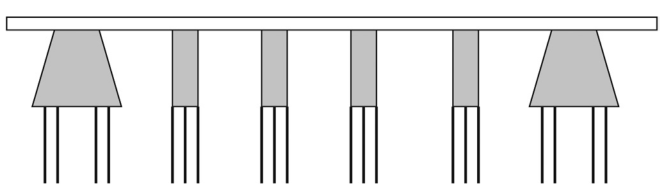


Figure 8.25 Friction piles.

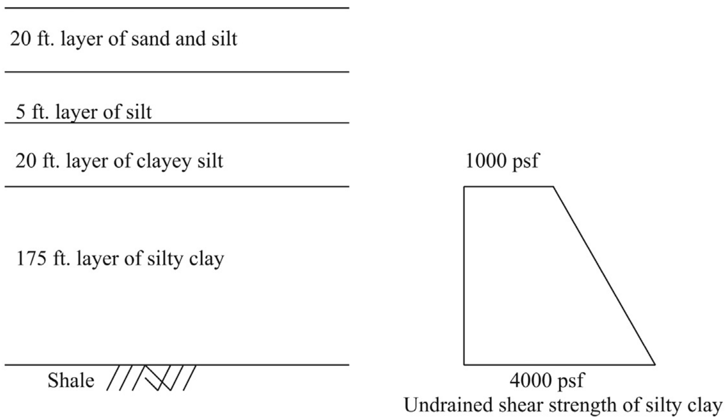


Figure 8.26 Soil conditions.

Table 8.7 **Load test data**

Pile type	Pile diameter (in.)	Pile length (ft.)	Ultimate load (tons)	Load carried by 1 ft. of pile (ton/ft.)
Tapered timber piles	Head = 14.5 Butt = 9	50	78	1.56
Precast concrete piles	9	78	65	0.83
Precast concrete piles	9	120	95	0.80
Steel pipe piles (wall thickness 6.35 mm)	9	78	50	0.64

8.10.4 Load settlement curves

- Load settlement curves obtained during pile load tests were similar in nature and one standard curve is shown in Fig. 8.27.
- On top of curve 2 is the elastic line shown for comparison purposes.
- Initial section of the pile load settlement curve is much steeper than the elastic curve.
- This means that for a given load, the pile would have a lesser settlement than the elastic compression (at a given load of "*P*" as shown, the settlement of the pile was almost half of the elastic settlement).
- This is due to skin friction between soil and pile. A portion of the pile was absorbed by soil skin friction.

8.10.5 Settlement values

- Settlement values obtained during pile load tests are given in Table 8.8.

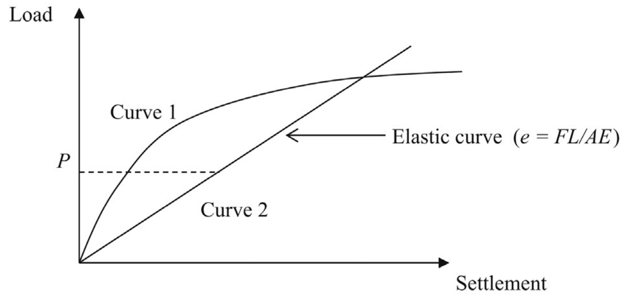


Figure 8.27 Load versus settlement.

Table 8.8 Settlement values

Pile type	Timber piles 50 ft. long (mm)	Steel pipe piles 78 ft. long (mm)	Precast concrete piles 78 ft. long (mm)	Precast concrete piles 120 ft. long (mm)
Settlement at 35 tons	3	4	2	3
Settlement at 45 tons	4	6	3	4
Settlement at 70 tons	8	Fail	4	8

Settlement of piles primarily occurs due to the following reasons:

- elastic settlement of soil
- elastic deformation of the pile
- the penetration of the pile into clay
- long-term consolidation settlement of clay soil

At 35 tons, short precast concrete piles had the lowest settlement. Long precast concrete piles settled more than the short precast concrete piles with the same diameter. This is due to larger elastic compression in long piles. In this situation, short piles may be more appropriate since, long piles would stress the soft silty clay underneath.

8.10.6 Pore pressure measurements

- Pore pressures were measured during pile driving. Pore pressure increase at a given depth was noticed to reach a peak value when the pile tip passes that depth (Fig. 8.28).
- Pore pressure starts to decrease when the pile tip moves deeper.

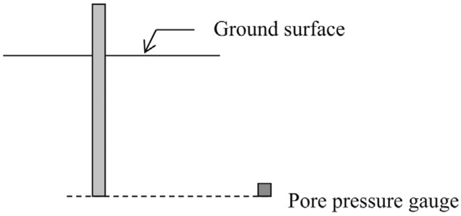
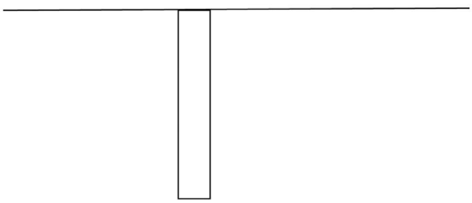


Figure 8.28 Pore pressure measurements.

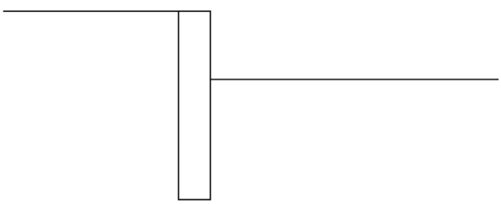
8.11 Case study 2: H–sections in retaining walls

H-Piles can be used to build retaining walls.

Step 1: drive H-piles



Step 2: excavate 3–4 ft.



Step 3: install timber lagging

Timber lagging basically comprises of timber planks. They are installed inside the H-section (Fig. 8.29).

Plan view is shown in Fig. 8.30.

Step 4: excavate more and install another set of timber lagging (Fig. 8.31, Plate 8.1).

X = 10–15 ft. (depending upon soil conditions and height of the wall, Plate 8.2).

Step 5: install wales (optional)

Wales are horizontal I-beams. Wales are needed for deep excavations (Fig. 8.32).

Step 5: install soil anchors (optional).

Soil anchors are installed into the soil and attached to wales. If soil anchors are to be installed two beams are attached together (Fig. 8.33, Plates 8.3–8.5).

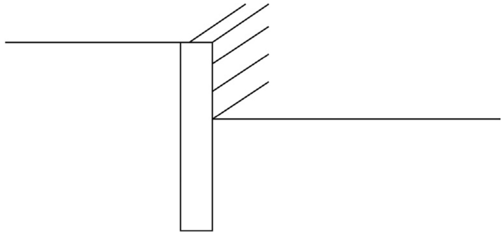


Figure 8.29 Install timber lagging and wales.

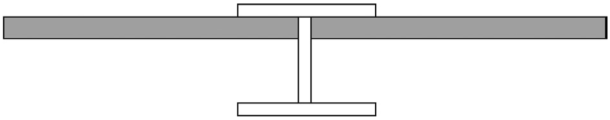


Figure 8.30 Plan view.

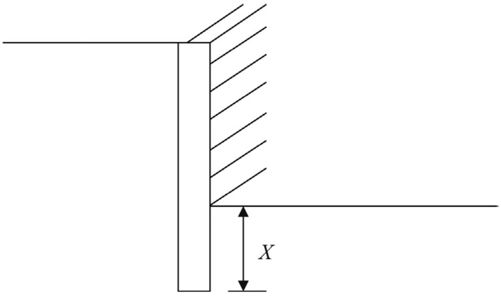


Figure 8.31 Timber lagging.

8.12 Design of pile groups

Typically piles are installed in a group and provided with a pile cap. The column will be placed on the pile cap so that the column load is equally distributed among the individual piles in the group (Fig. 8.34).

Capacity of a pile group is obtained by using an efficiency factor.

Pile group capacity = efficiency of the pile group × single pile capacity × number of piles.

If the pile group contains 16 piles, capacity of a single pile is 30 tons, and the group efficiency is found to be 0.9, the group capacity becomes 432 tons.

Pile group capacity = 0.9 × 30 × 16 = 432 tons.

Plate 8.1 Timber laggings are installed between H-sections.

Plate 8.2 Excavation after installation of first set of lagging.

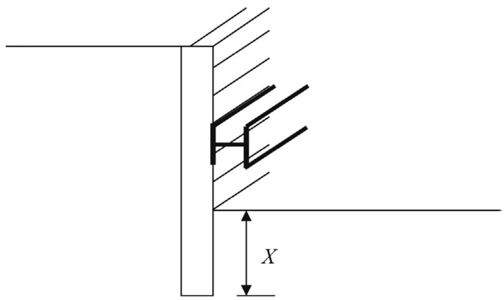


Figure 8.32 Wales.

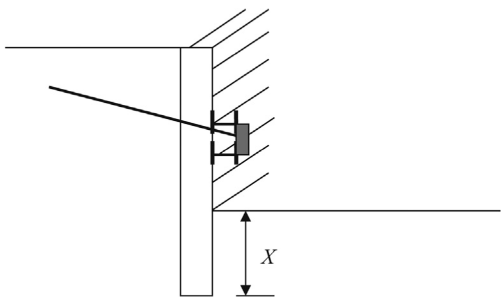


Figure 8.33 Soil anchors.

It is clear that high pile group efficiency is desirable. The question is "how to improve the group efficiency?" The pile group efficiency is dependent on the spacing between piles. When the piles in the group are closer together, the pile group efficiency decreases. When the piles are placed far apart, the efficiency increases. When the piles are placed far apart, the size of the pile cap has to be larger increasing the cost of the pile cap.

8.12.1 Soil disturbance during driving

What happens in a pile group?

When piles are driven, the soil surrounding the pile will be disturbed. Disturbed soil has a lesser strength than undisturbed soil. Some of the piles in the group are installed in partially disturbed soil causing them to have a lesser capacity than others. Typically piles in the center are driven first (Fig. 8.35).

Soil disturbance caused by one pile impacts the capacity of adjacent piles. The group efficiency can be improved by placing the piles at a larger spacing. In clay soils, shear strength will be reduced due to disturbance.

8.12.2 Soil compaction in sandy soil

When driving piles in sandy soils, surrounding soil will be compacted. Compacted soil tends to increase the skin friction of piles. A pile group placed in sandy soils may

Plate 8.3 Timber lagging, horizontal wales and soil anchors are shown. Wales are slanted so that the face plate of the soil anchor can seat on the wale properly.

Plate 8.4 Soil anchor.

Plate 8.5 Soil anchor rig.

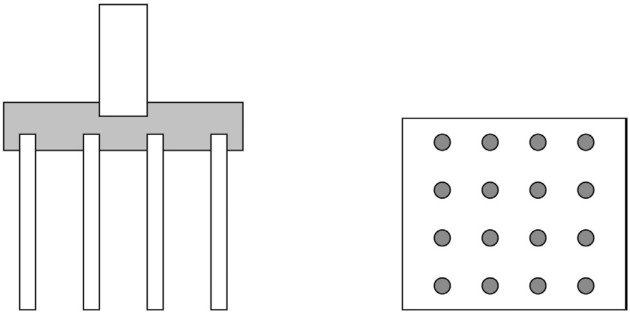


Figure 8.34 Pile groups.

have a larger than one group efficiency. Soil compaction due to pile driving will be minimal in clay soils.

8.12.3 Pile bending

When driving piles in a group some piles could be bent due to soil movement. This effect is more pronounced in clayey soils.

Assume pile A is driven first and pile B is driven next. Soil movement caused by pile "B" can bend pile "A" as shown in Fig. 8.36. This in return would create a lower group capacity.

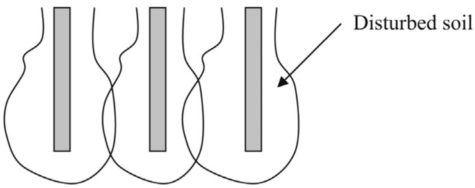


Figure 8.35 Disturbed soil due to pile driving.

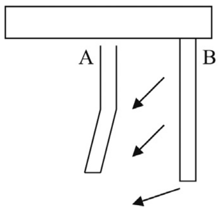


Figure 8.36 Pile bending.

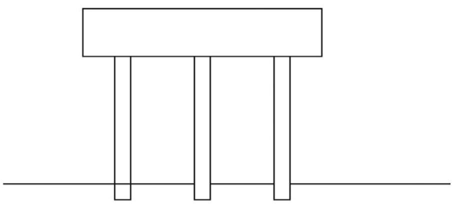


Figure 8.37 End bearing soil.

8.12.4 End bearing piles

Piles mainly relying on end bearing capacity may not be affected by other piles in the group (Fig. 8.37).

8.12.4.1 Piles ending in very strong bearing stratum or in rock

When piles are not dependent on the skin friction, group efficiency of 1.0 can be used.

Various guidelines for computing group capacity are given in the further section.

8.12.4.2 AASHTO (1992) guidelines

American Association of State Highway and Transportation (AASHTO), provides the following guidelines.

AASHTO considers three situations:

1. pile group in cohesive soils (clays and clayey silts)
2. pile group in noncohesive soils (sands and silts)
3. pile group in strong soil overlying weaker soil

Table 8.9 Pile group efficiency for clayey soils

Pile spacing (center to center, D = diameter of piles)	Group efficiency
3 D	0.67
4 D	0.78
5 D	0.89
6 D or more	1.00

Table 8.10 Pile group efficiency for sandy soils

Pile spacing (center to center, D = diameter of piles)	Group efficiency
3 D	0.67
4 D	0.74
5 D	0.80
6 D	0.87
7 D	0.93
8 D or more	1.00

The author has constructed Tables 8.9 and 8.10 using the AASHTO guidelines for pile group efficiency in cohesive soils.

8.12.5 Design example

8.12.5.1 Pile group capacity

Pile group is constructed in clayey soils as shown. Single pile capacity was computed to be 30 tons and each pile is 12 in. in diameter. Center to center distance of piles is 48 in. Find the capacity of the pile group using the AASHTO method (Fig. 8.38).

8.12.6 Solution

Pile group capacity = efficiency of the pile group × single pile capacity × number of piles
Center to center distance between piles = 48 in.
Since the diameter of piles is 12 in, center-to-center distance is 4D.
Efficiency of the pile group = 0.78 (AASHTO table).
Pile group capacity = $0.78 \times 30 \times 4 = 93.6$ tons

8.12.7 Pile group capacity when strong soil overlie weaker soil (AASHTO, 1992)

Usually piles are ended in strong soils. In some cases, there could be a weaker soil stratum underneath the strong soil strata (Fig. 8.39).

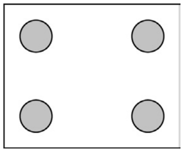


Figure 8.38 Pile group (plan view).

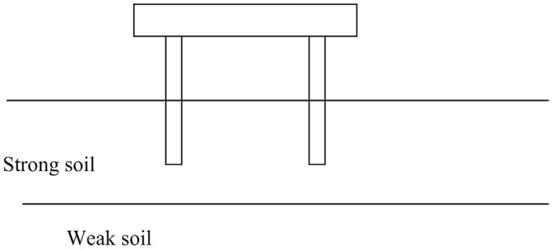


Figure 8.39 Pile group capacity.

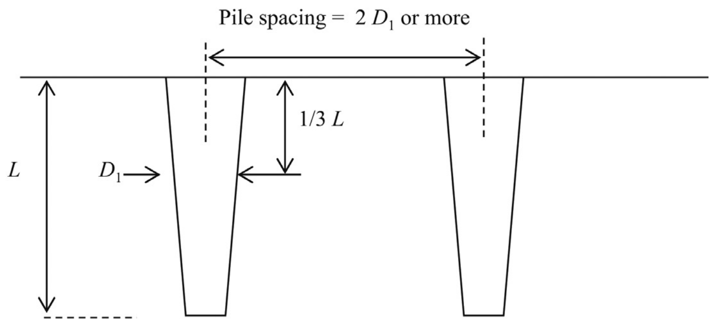


Figure 8.40 Tapered piles.

8.12.8 Group efficiency computation

A number of methods is available to find the group efficiency factor.

8.12.9 Pile spacing (center-to-center distance): International Building Code Guidelines

In no case minimum distance should be less than 24 in.

For circular piles: (minimum distance center-to-center) twice the average diameter of the butt.

Rectangular piles: (minimum center-to-center distance) 3/4 times the diagonal for rectangular piles

Tapered piles: (minimum center-to-center distance) twice the diameter at 1/3 of the distance of the pile measured from the top of pile (Fig. 8.40).

8.13 Eccentric loading on a pile group

Usually piles are arranged in a group, so that the load will be equally distributed and no additional loads will be generated due to bending moments (Fig. 8.41).

When a pile group is subjected to an eccentric load, individual piles would have different loads. The following example shows how to calculate the load on individual piles.

8.13.1 Design example 1

A 10-ton column load is acting on point "*K*". Find the loads on piles 1, 2, 3, and 4.

Eccentricity (e) = 1 ft. (Fig. 8.42).

8.13.2 Solution

- Step 1: if the column load was to act on the center of gravity, all four columns would get 2.5 tons each.
  - since the load is acting at point "*K*", load on piles 1 and 4 would be larger and load on piles 2 and 3 would be smaller.
  - assume loads on piles to be R_1, R_2, R_3, and R_4, respectively. Area of a pile is considered to be "*A*".

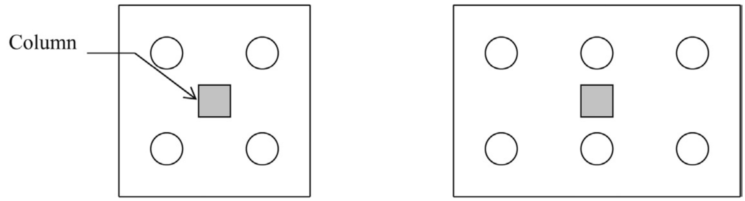


Figure 8.41 Eccentric loading on a pile group.

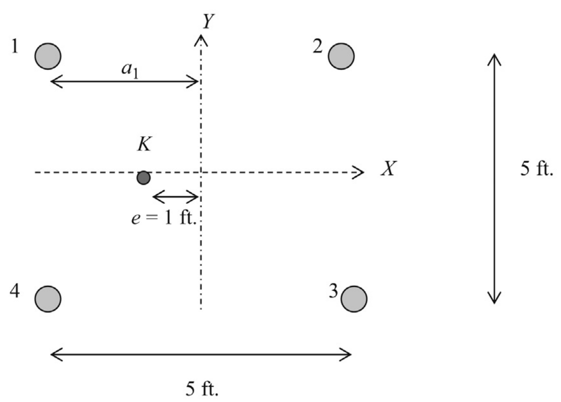


Figure 8.42 Loading.

Stress of a pile is given by Equation (8.1). The first term C/(nA) represents the load on each pile if there is no eccentricity. The second term (Cea_1)/I represents the stress due to bending moment induced by eccentricity.

The stress on some piles would increase due to eccentricity while stress on some other piles would decrease. For example, stress on piles 1 and 4 would increase due to eccentricity while stress on piles 2 and 3 would decrease.

$$\sigma_1 = \frac{R_1}{A} = \frac{C}{nA} +/- \frac{C.e.a_1}{I} \tag{8.1}$$

Stress on a pile if there is no eccentricity

Additional stress due to eccentricity

Here σ_1, stress on pile 1; A, area of the pile; C, column load; n, number of piles; e, eccentricity; a_1, the distance between the center of pile1 and the center of gravity of the pile group, measured parallel to "e" (eccentricity, Fig. 8.1); I, moment of inertia of the pile group, measured from the axis perpendicular to the eccentricity (going through the center of gravity). In this case "Y" axis is perpendicular to the eccentricity. Hence, all distances should be obtained from the "Y" axis.

- Step 2: compute "I" (moment of inertia)
  Moment of inertia = Ar^2 (r, distance to the pile from the axis. In this case axis perpendicular to the eccentricity passing through the center of gravity should be considered).

  $$I = A\times 2.5^2 + A\times 2.5^2 + A\times 2.5^2 + A\times 2.5^2 = 25\,A$$

  Note: the aforementioned distances to the piles are measured from the axis perpendicular to "e" passing through the center of gravity. The measurement should be taken parallel to "e." In this case distances to all piles were taken from "Y" axis measured *parallel* to eccentricity).
  Step 3: compute "a_1"
  a_1, the distance between the center of pile 1, and the center of gravity of the pile group, measured parallel to "e" (eccentricity).

  a_1, 2.5 ft.
- Step 4: apply Eq. (8.1)

  $$\sigma_1 = \frac{R_1}{A} = \frac{C}{nA}(+/-)\frac{Cea1}{I}$$
  $$\frac{R_1}{A} = \frac{10}{4A} + \frac{10\times 1\times 2.5}{25A} = \frac{3.5}{A}$$

  R_1 = 3.5 tons
  Note: the "+" sign was used since pile 1 was on the same side as the column load. The location of the load enhances the load on pile 1. If the 10-ton column load were applied at the center of gravity, then all four piles would be loaded equally).
  By symmetry load on pile 4 is the same as pile 1. Hence, $R_1 = R_4$ = 3.5 tons.
- Step 5: compute the load on pile 2

  $$\sigma_2 = \frac{R_2}{A} = \frac{C}{nA}(+/-)\frac{Cea2}{I}$$

  σ_2, stress in pile 2; A, area of the pile; C, column load; n, number of piles; e, eccentricity; a_2, the distance between the center of pile 2 and the center of gravity of the pile group, measured parallel to "e" (eccentricity); I, moment of inertia of the pile group, measured around the axis perpendicular to the eccentricity going through the center of gravity.

$$\frac{R_2}{A} = \frac{10}{4A} + \frac{10 \times 1 \times 2.5}{25A} = \frac{1.5}{A}$$

In this case (−) sign is used, since bending moment due to eccentricity tends to reduce the load on pile 2. $R_2 = 1.5$ tons
By symmetry, load on pile 3 is also 1.5 tons.
Hence, $R_1 = 3.5$ ton, $R_2 = 1.5$ tons, $R_3 = 1.5$ tons, and $R_4 = 3.5$ tons.
(Total10 tons).

8.14 Double eccentricity

8.14.1 Design example 2

It is possible to have a column load eccentric to X and Y axes as shown in Fig. 8.43. The column load is given to be 20 tons. Find the load on each pile.

8.14.2 Solution

- Step 1: if the column load is to act on the center of gravity of piles all four columns would get 5 tons each.
    - Assume loads on piles to be R_1, R_2, R_3, and R_4, respectively. Area of pile is considered to be "A."

$$\sigma_1 = \frac{R_1}{A} = \frac{C}{n\,A} \;{+/-}\; \underbrace{\frac{C.\,e_x.\,a_x}{I_y}}_{\text{Additional stress on pile 1, due to eccentricity along X-axis}} \;{+/-}\; \underbrace{\frac{C.\,e_y.\,a_y}{\mathbf{I}_x}}_{\text{Additional stress on pile 1, due to eccentricity along Y-axis}} \tag{8.2}$$

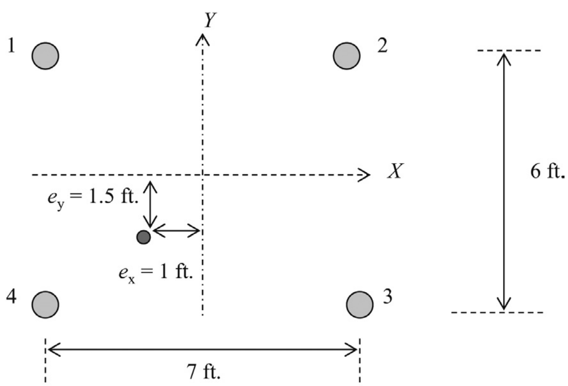


Figure 8.43 Double eccentricity.

I_y, in the second term, I_y is used instead of I_x. Moment of inertia should be considered along the axis perpendicular to the eccentricity. If eccentricity is along "X" axis then the moment of inertia should be considered along the "Y" axis.

σ_1, stress in pile 1; A, area of the pile; C, column load; n, number of piles; e_x, eccentricity parallel to X-axis; (e_x = 1 ft.); e_y, eccentricity parallel to Y-axis; (e_y = 1.5 ft.);a_x, the distance between the center of pile1 and the center of gravity of the pile group, measured parallel to "e_x" (a_x = 3.5 ft.); a_y, the distance between the center of pile 1 and the center of gravity of the pile group, measured parallel to "e_y" (a_y = 3 ft.).

- Step 2: compute "I_x" and "I_y"
  $I_y = A.\ 3.5^2 + A.\ 3.5^2 + A.\ 3.5^2 + A.\ 3.5^2 = 49\ A$ (distances are measured from Y-axis)
  $I_x = A.\ 3.0^2 + A.\ 3.0^2 + A.\ 3.0^2 + A.\ 3.0^2 = 36\ A$ (distances are measured from X-axis)
  a_x = 3.5 ft. and a_y = 3.0 ft. for pile 1 ("a_x" is measured parallel to X-axis and "a_y" is measured parallel to Y-axis.)
  e_x = 1 ft. and e_y = 1.5 ft. (these two values are given).
- Step 3: apply Equation (8.2) to pile 1

$$\sigma_1 = \frac{R_1}{A} = \frac{C}{nA} +/- \frac{Ce_x a_x}{I_y} +/- \frac{C\,e_y a_y}{I_x}$$

$$\sigma_1 = \frac{R_1}{A} = \frac{20}{4\,A} + \frac{20 \cdot (1\,\text{ft.})(3.5\,\text{ft.})}{49\,A} - \frac{20 \cdot (1.5\,\text{ft.})(3.0\,\text{ft.})}{36\,A}$$

$$\sigma_1 = \frac{3.93\ \text{ft.}^2}{A\,\text{tons}}; \qquad R_1 = 3.93\,\text{tons}$$

Eccentricity "e_x" increases the load on pile 1 while eccentricity "e_y" decreases the load on pile 1.

- Step 4: apply the preceding equation to pile 2.

$$\sigma_2 = \frac{R_2}{A} = \frac{C}{nA}(+/-)\frac{Ce_x a_x}{I_y}(+/-)\frac{Ce_y a_y}{I_x}$$

$$\sigma_2 = \frac{R_2}{A} = \frac{20}{4\,A} - \frac{20(1\ \text{ft})(3.5\ \text{ft})}{49\,A} - \frac{20 \cdot (1.5\ \text{ft})(3.0\ \text{ft})}{36\,A}$$

Note: both signs are (−) since both eccentricities are moving away from pile 2 (Fig. 8.43).

$$\sigma_2 = \frac{1.07\ \text{ft.}^2}{A\,\text{tons}}; \qquad R_2 = 1.07\,\text{tons}$$

- Step 5: apply the preceding equation to pile 3.

$$\sigma_3 = \frac{R_3}{A} = \frac{C}{nA} \pm \frac{Ce_x a_x}{I_y} \pm \frac{Ce_y\,a_y}{I_x}$$

$$\sigma_3 = \frac{R_3}{A} = \frac{20}{4\,A} - \frac{20(1\ \text{ft.})(3.5\ \text{ft.})}{49\,A} + \frac{20(1.5\ \text{ft.})(3.0\ \text{ft.})}{36\,A}$$

$$\sigma_3 = \frac{6.07\ \text{ft.}^2}{A\,\text{tons}}; \qquad R_3 = 6.07\,\text{tons}$$

For pile 3, eccentricity "e_x" reduces the load on pile 3 while eccentricity "e_y" increases the load on pile 3.

- Step 6: apply the preceding equation to pile 4.

$$\sigma_4 = \frac{R_4}{A} = \frac{C}{nA} \pm \frac{Ce_x a_x}{I_y} \pm \frac{Ce_y a_y}{I_x}$$

$$\sigma_4 = \frac{R_4}{A} = \frac{20}{4\,A} + \frac{20(1\,\text{ft.})(3.5\,\text{ft.})}{49\,A} + \frac{20(1.5\,\text{ft.})(3.0\,\text{ft.})}{36\,A}$$

$$\sigma_4 = \frac{8.93\,\text{ft.}^2}{A\,\text{tons}} \qquad R_4 = 8.93\ \text{tons}$$

Both eccentricities increase the load on pile 4.
Check whether the sum of all four piles add up to 20 tons.
$R_1 = 3.93$; $R_2 = 1.07$; $R_3 = 6.07$; and $R_4 = 8.93$

8.15 Pile groups in clay soils

Piles in a group could fail as a group. This type of failure is known as group failure.

Group failure as shown in Fig. 8.44, does not occur in sandy soils and is not a design consideration.

8.15.1 Design methodology for pile group failure

Assume a pile group with dimensions of $L \times W \times D$ as shown in Fig. 8.44.

Skin friction of the group = $2\cdot(L + W) \times D \times \alpha\cdot C_u$

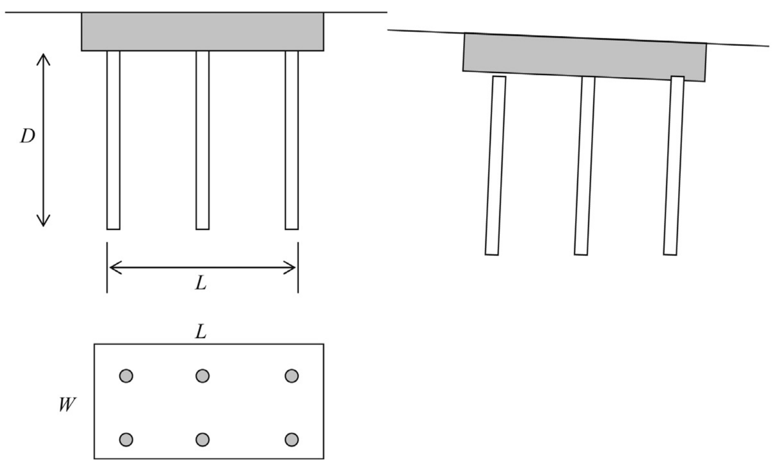


Figure 8.44 Pile groups in clay soils.

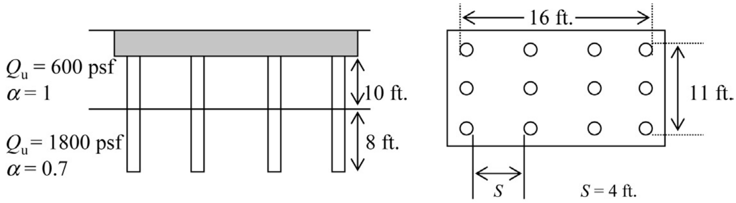


Figure 8.45 Allowable pile capacity.

Here, α, skin friction coefficient; and C_u, cohesion of the clay soil.
End bearing of the group = $N_c \cdot C_u \times (L \times W)$.
Pile group capacity = $2 \cdot (L + W) \times D \times \alpha\ C_u + N_c \cdot C_u \times (L \times W)$.
N_c = bearing capacity factor = usually taken as 9.

8.15.2 Pile group capacity based on individual pile capacity

Pile group capacity based on capacity of individual piles = efficiency × N × single pile capacity, where N, number of piles in the group.

8.15.3 Design example

Find the allowable pile capacity of the pile group shown. Individual piles are of 1 ft. diameter. Pile spacing = 4 ft.

Note: pile spacing (S) is normally in the range of $3d$ to $5d$. (d = pile diameter) (Fig. 8.45).

Step 1: find the ultimate pile capacity of an individual pile.
Top layer, Q_u = 600 psf; $Q_u/2 = S_u = C$ = 300 psf = 0.15 tsf;
α = 1.0 (α value can be obtained from the Kolk and van der Valde table).
Skin friction due to top layer = $\alpha \cdot C$ × perimeter = 1.0 × 0.15 × π × 1 × 10 = 4.7 tons.
Bottom layer, Q_u = 1800 psf; $Q_u/2 = S_u = C$ = 900 psf; C = 0.45 tsf
α = 0.7
Skin friction due to bottom layer = $\alpha \cdot C$ × perimeter = 0.7 × 0.45 × π × 1 × 8 = 7.9 tons.
Total skin friction (for an individual pile) = 7.9 + 4.7 = 12.6 tons.
Tip bearing resistance = 9.C. area of the pile at the tip = 9 × 0.45 × (π × 1^2)/4 = 3.2 tons.
Tip bearing resistance = 3.2 tons.
Ultimate pile capacity of an individual pile = 3.2 + 12.6 = 15.8 tons.
Assume an efficiency factor of 0.8 (efficiency factor is used since the piles in the group are closely spaced. Hence, piles could stress the soil more than individual piles located far apart.).
Ultimate capacity of the group = 0.8 × 12 × 15.8 tons = 151.7 tons.

Note: AASHTO (Section 4.5.6.4) recommends an efficiency factor of 0.7 for pile groups in clay soils with a center-to-center spacing of $3D$ or more.

Step 2: find the ultimate pile capacity of the group as a whole.

- The pile group can fail as a group. This type of failure is known as "group failure."
  The ultimate capacity of the group = group skin friction + group tip bearing resistance
- Pile group is considered as one big rectangular pile with dimensions of 11 × 16 ft.
  Group skin friction = group skin friction (top layer) + group skin friction (bottom layer).
  Group skin friction (top layer) = $\alpha \cdot C \cdot$perimeter area of the group within the top layer
  = 1.0 × 0.15 × 2x (16 + 11) × 10 = 81 tons
  The perimeter area of the pile group is 2 × (16 + 11) × 10 (depth = 10 ft.).
  Group skin friction (bottom layer) = α·C·perimeter area of the group within the bottom layer
  = 1.0 × 0.45 × 2 × (16 + 11) × 8 = 194 tons.
  For group failure, "α" value is taken to be 1.0 since the failure occurs between soil against soil. For soil/pile failure "α" coefficient may or may not be equal to 1.0.
  Group *tip* bearing resistance = 9*C* × (area of the group) = 9 × 0.45 × (16 × 11) = 713 tons.
  Ultimate group capacity = 81 + 194 + 713 = 988 tons.
  Select the lesser of 988 tons and 151.7 tons.
  Hence, the ultimate capacity of the group = 151.7 tons.
  Allowable pile capacity of the group = 151.7/3.0 = 50.6 tons (FOS = 3.0).

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Design of bored piles

9

Bored piles are constructed by drilling a hole, inserting a rebar cage, and grouting the hole. When the hole is augered, we call these piles augercast piles. See section: Augercast Piles in chapter: Pile Types for a complete description of the construction of augercast piles (Fig. 9.1).

9.1 Augercast pile design (empirical method)

Step 1: auger a hole
Step 2: start pumping grout or concrete through the auger while lifting the auger. Concrete should be pumped inside the grout. The auger should not be raised above the grout level. The auger should be lifted slowly so that the grout level is always above the tip of the auger.
Step 3: concrete or grout the hole completely and place an "H" section or a steel rod to anchor the pile to the pile cap.
Step 4: construct the pile cap

9.2 Design concepts

- Bearing capacity of augercast piles are relatively low compared to similar-size driven piles.
- Volume of grout depends on the applied pressure. If grout is pumped at a higher pressure more volume of grout will be pumped. In most cases the volume of grout pumped is more than the volume of the hole.
- Grout factor is defined as the ratio of grout volume pumped to volume of the hole.

$$\text{Grout factor} = \frac{\text{Grout volume pumped}}{\text{Volume of the hole}}$$

- Augercast piles with high grout factors would perform better than augercast piles with low grout factors.
- It is obvious that piles with low grout factors would have a lesser skin friction since not much grout would go into the surrounding soil. On the other hand, when a higher grout pressure is applied, grout would go into the voids in the surrounding soil and develop a higher skin friction (Fig. 9.2).

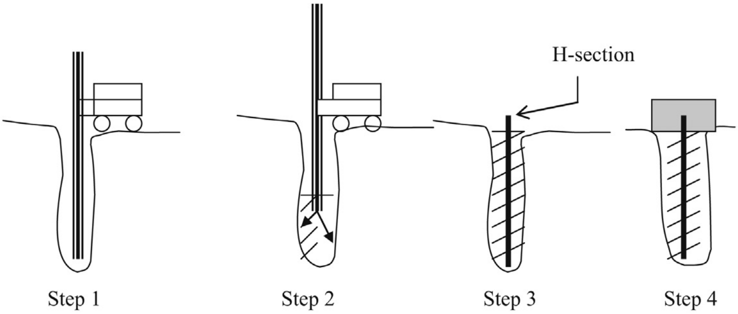


Figure 9.1 Augercast pile construction.

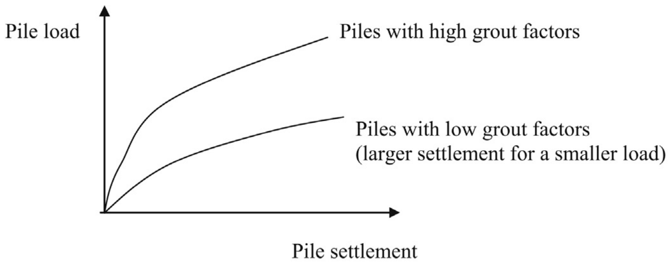


Figure 9.2 Pile load versus settlement.

9.3 Augercast Pile design in sandy soils

The ultimate strength of a pile depends on three factors:

- skin friction
- end bearing
- structural capacity of the pile

9.3.1 Computation of skin friction

The skin friction of augercast piles, is given by the following equation:

$$f_s = P_0' \times K_s \tan \delta$$

where, f_s, unit skin friction; P_0', effective stress at the depth considered; K_s, lateral earth pressure coefficient; and δ, friction angle between grout and soil. Assume that $P_0' \times K_s = \beta$; hence, $f_s = \beta \cdot \tan \delta$ tsf (Table 9.1).

- Intermediate values can be interpolated.
- Experiments indicate that β for concrete and grout did not change significantly (Montgomery, 1980).
- Research has not shown any variation of the β value based on the N value of sand. Hence, β factors are independent of N value of sand.

Table 9.1 **Shaft length versus β (Neely, 1991)**

Shaft length (ft.)	80	60	40	30	20	10
β	0.2	0.25	0.55	1.0	1.7	2.5

9.3.2 Computation of end bearing

After analyzing vast numbers of empirical data, Neely (1991) has provided the following equation:

$q = 1.9\,N < 75$ tsf (Neely, 1991)

where q, end bearing resistance (tsf); and N = SPT value at the tip of the pile; q value should not be larger than 75 tsf.

9.4 Failure mechanisms of augercast piles

The failure mechanisms of augercast piles are shown in Fig. 9.3.

9.4.1 Structural capacity of augercast piles

The structural capacity of grouted base piles are given by:

$Q = \sigma_c + 2\,[\sigma_c/(\sigma_c - \sigma_t)]^{1/2} \cdot P_a$ (Kusakabe et al., 1994).

Here, Q, pressure exerted by pile on the grouted base; σ_c, compressive strength of grout; σ_t, tensile strength of grout; and P_a, ultimate inner pressure of hollow thick cylinder (the following equation for "P_a" is obtained from structural mechanics);

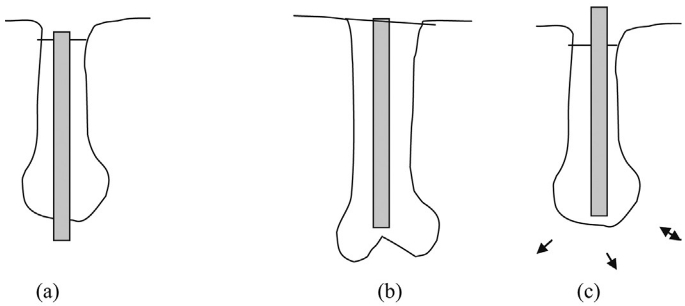


Figure 9.3 Failure mechanisms of augercast piles. (a) Pile penetrates the grouted base and fails (punching failure). This is rare when a rebar cage is inserted. (b) Grouted base fails (splitting failure). (c) Bearing failure of soil.

$$P_a = \frac{[1-(a/b)^2]\sigma_t + [1+(a/b)^2]K_0 \cdot \gamma \cdot D}{2}.$$

Here, K_0 = earth pressure coefficient at rest = $(1 - \sin \phi')$ and γ = unit weight of soil.

9.5 Case study: comparison between bored piles and driven piles

- A case study was done to compare the capacities of bored piles and driven piles in clay soils. (Meyerhof and Murdock, 1953).
- Bored and driven piles were installed to a depth of 40 ft. and load tested.
- Pile load test values were compared with theoretical computations (Fig. 9.4).

9.5.1 Clay properties

- Plastic limit and liquid limit of the clay was constant throughout the total depth (Fig. 9.5).
- Shelby tube samples were taken and tested for shear strength under different conditions:
  - Shelby tube samples were tested immediately after taking them (curve 1)
  - Shelby tube samples were tested after remolding the clay (curve 2)
  - Shelby tube samples were tested after 1 week (curve 3)
  - Shelby tube samples were tested after softening them by adding water (curve 4, Fig. 9.6)
- Samples tested immediately after taking them had the highest shear strength.
- Samples tested after adding water had the lowest shear strength.
- Shear strength of remolded samples were less than the ones, which were tested immediately.
- Shear strength decreased when tested after 1 week. This was due to creation of cracks and fissures during that time due to loss of moisture (Fig. 9.4).

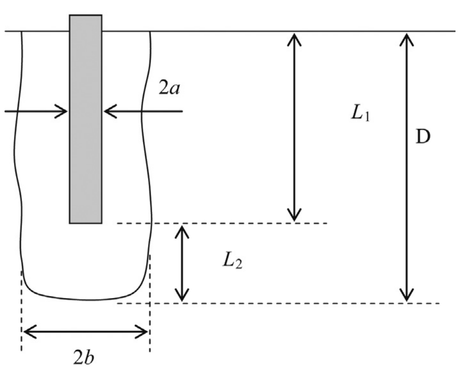


Figure 9.4 Bored pile.

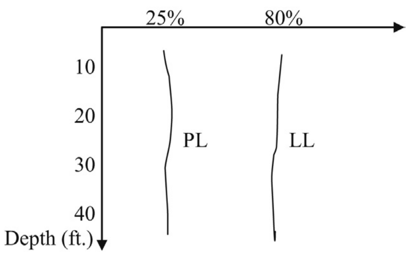


Figure 9.5 Depth versus LL and PL.

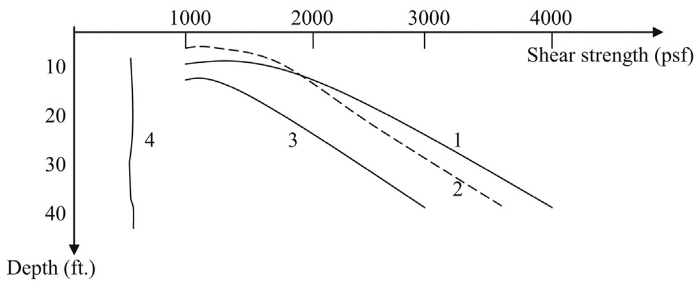


Figure 9.6 Depth versus shear strength.

9.5.2 Results

- Driven piles had a higher capacity than bored piles.
- Load test values for bored piles were compatible with the shear strength values given by curve 4. The reason for this was assumed to be migration of water from wet concrete to surrounding clay. Increase of water content in clay decreased the shear strength value drastically.
- The solution to this problem is to use a dry concrete mix. In that case, compaction of concrete could be a problem.
- Pile load test values of driven piles were compatible with shear strength values given by curve 2. This was expected. When a pile is driven, the soil surrounding the pile would be remolded.

9.6 Design of pin piles: semiempirical approach

9.6.1 Theory

- A few decades ago, no engineer would have recommended any pile less than 9 in. in diameter. Today, some piles could be as small as 4 in. in diameter.
- These small-diameter piles are known as *pin piles*. Other names such as *mini piles, micro piles, GEWI piles, pali radice, root piles,* and *needle piles,* are also used to describe small-diameter piles (Fig. 9.7).

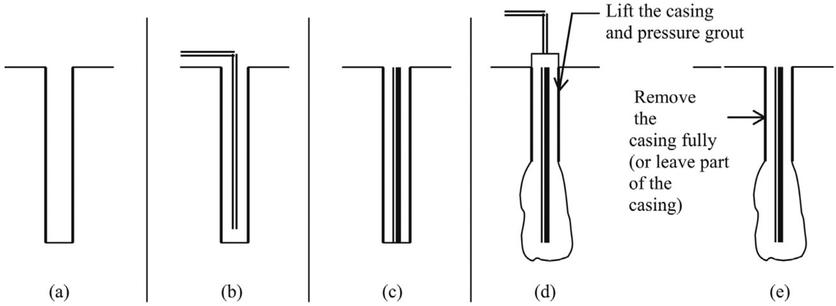


Figure 9.7 Construction of a pin pile. (a) Drill a hole using a roller bit or an auger. A casing is installed to stop soil from dropping into the hole. If the hole is steady, a casing may not be necessary. (b) Tremie grout the hole. (c) Insert the reinforcement bar or bars. In some cases more than one bar may be necessary. (d) Lift the casing and pressure grout the hole. (e) Fully remove the casing. Some engineers prefer to leave part of the casing. This would increase the strength and the cost.

9.6.2 Construction of a pin pile

The method to construct a pin pile has been discussed in this section.

9.6.3 Concepts to consider

9.6.3.1 Drilling the hole

- Augering could be a bad idea for soft clays since augers tend to disturb the soil more than roller bits. This would decrease the bond between soil and grout and would lower the skin friction. Drilling should be conducted with water. Drilling mud should not be used since casing is utilized to stop soil from falling in. Hence, drilling mud is not necessary. Drilling mud would travel into the pores of the surrounding soil. When drilling mud occupies the pores, grout may not spread into the soil pores. This would reduce the bond between grout and soil.

9.6.3.2 Tremie grouting

- After the hole is drilled, tremie grout is placed. Tremie grout is a cement/water mix (typically with a water content ratio of 0.45:0.5 by weight).

9.6.3.3 Placement of reinforcement bars

- High-strength reinforcement bar (or number of bars wrapped together) or steel pipes can be used at the core. High-strength reinforcement bars specially designed for pin piles are available from Dywidag Systems International and William Anchors (leaders in the industry). These bars can be spliced easily, so that any length can be accommodated.

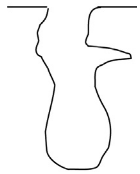


Figure 9.8 Grout distribution in the hole.

9.6.3.4 Lift the casing and pressure grout

- During the succeeding step, the casing is lifted and pressure grouted. The pressure should be adequate to force the grout into the surrounding soil to provide a good soil/grout bond. At the same time, pressure should not be large enough to fracture the surrounding soil. Failure of the surrounding soil would drastically reduce the bond between grout and soil. This would decrease the skin friction. Typically grout pressure ranges from 0.5 MPa (10 ksf) to 1.0 MPa (1 MPa = 20.9 ksf).
- *Ground heave:* There are many instances where ground heave occur during pressure grouting. This aspect needs to be considered during the design phase. If there are nearby buildings, action should be taken to avoid any grout flow into these buildings.

9.6.3.5 Remove the casing

At the end of pressure grouting, the casing is completely removed. Some engineers prefer to leave part of the casing intact. Obviously this would increase the cost. The casing would increase the rigidity and the strength of the pin pile. The casing would increase the *lateral resistance* of the pile significantly. Furthermore, it is guaranteed that there is a pin pile with a diameter not less than the diameter of the casing. The diameter of a pin pile could reduce due to soil encroachment into the hole. In some occasions it is possible for the grout to spread into the surrounding soil and create an irregular pile.

The grout could spread in an irregular manner as shown in Fig. 9.8. This could happen when the surrounding soil is loose. The main disadvantage of leaving the casing in the hole is the additional cost.

9.6.4 Design of pin piles in sandy soils

9.6.4.1 Design example

Compute the design capacity of the pin pile shown (diameter 6 in.). The surrounding soil has an average SPT (N) value of 15 (Fig. 9.9).

Step 1: ultimate unit skin friction in gravity grouted pin piles is given by

$\tau = 21 \times (0.007\,N + 0.12)$ ksf (Suzuki et al., 1972)

Note: This equation has been developed based on empirical data.

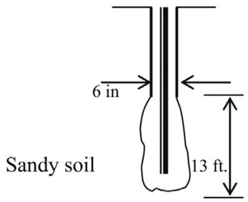


Figure 9.9 Pin pile.

N, SPT value; and τ, unit skin friction in ksf.

For N value of 15:

$\tau = 21 \times (0.007 \times 15 + 0.12)$ ksf

$\tau = 4.725$ ksf

The diameter of the pile is 6 in. and the length of the grouted section is 13 ft.

Step 2: Skin friction below the casing = $\pi \times (6/12) \times 13 \times 4.725$ kip = 96.5 kip = 48.2 tons.

Note: most engineers ignore the skin friction along the casing. End bearing capacity of pin piles is not significant in most cases due to low cross-sectional area. If the pin piles are seated on a hard soil or rock stratum, end bearing needs to be accounted for.

Assume a factor of safety of 2.5.

Allowable capacity of the pin pile = 48.2/(2.5) = 19 tons.

Note: Suzuki's equation was developed for pin piles grouted using gravity. For pressure-grouted pin piles the ultimate skin friction can be increased by 20–40%.

Note: Littlejohn (1970) proposed another equation:

$\tau = 0.21$N ksf (where "N" is the SPT value)

For $N = 15$, $\tau = 0.21 \times 15 = 3.15$ ksf.

Suzuki's equation yielded $\tau = 4.725$ ksf.

For this case Littlejohn's equation provides a conservative value for skin friction. If Littlejohn's equation was used for the pin pile previously discussed:

Ultimate skin friction = $\pi \times (6/12) \times 13 \times 3.15$ = 64.3 kip.

Allowable skin friction = 64.3/2.5 = 25 kip = 12.5 tons.

Suzuki's equation yielded 19 tons for the pin pile.

9.6.4.2 Pin pile example

- The building shown in Fig. 9.10 is built on a shallow foundation resting on a compressible clay layer. The building has been subjected to settlement. Pin piles are driven through the *foundation* to provide additional support.
- The rig is taken inside the building to install pin piles.
- Pin piles extend to the deep bearing ground. If the building tends to settle further, pin piles would stop it.

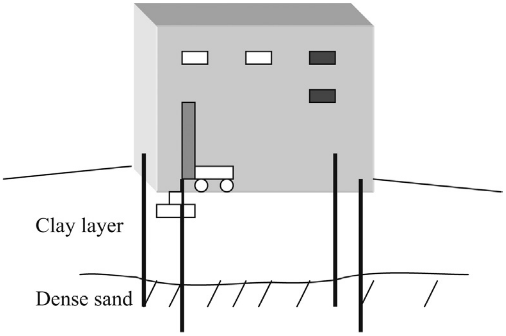
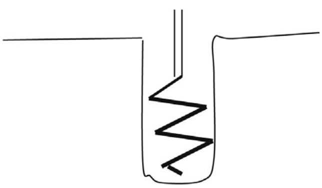


Figure 9.10 Piles on dense sand.

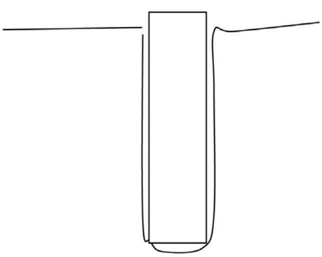
9.7 Bored piles in retaining walls

Bored piles or augercast piles are commonly used in retaining walls. Basically piles are first installed by augering a hole. These augercast piles are used as soldier beams. The following construction procedure is generally followed (Fig. 9.11).

Step 1: drill a hole with an auger



Step 2: insert a casing (if the soil is not caving in casing is not necessary)



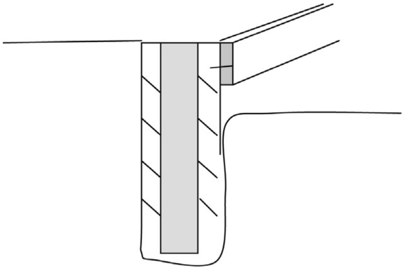
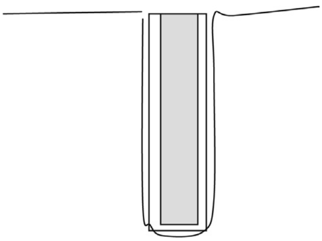
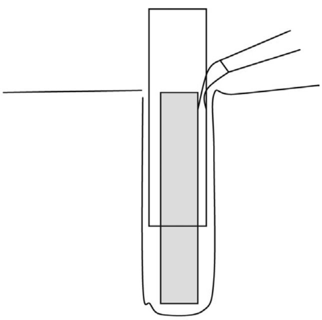


Figure 9.11 Bored piles in retaining walls.

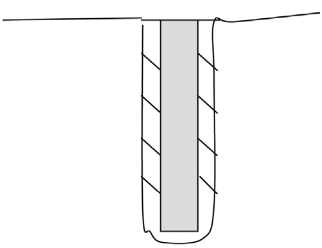
Step 3: insert a rebar cage or an I-section



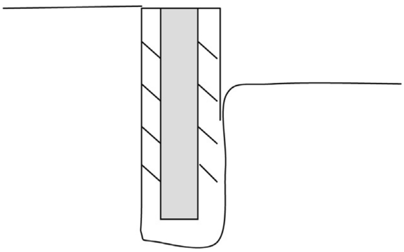
Step 4: concrete the hole, while pulling out the casing (typically high-strength grout is used)



Step 5: complete the pile and wait for concrete or grout to cure



Step 6: excavate 3–4 ft of soil



Step 7: install lagging (Fig. 9.11)

There are many different ways to attach lagging to concrete. Concrete bolts, or welding a steel plate to the I-beam, and so on, can be used (Plate 9.1).

Step 8: Install wales and soil anchors (see the previous chapters for wales and soil anchors).

Plate 9.1 In this case, casing is left in place. I-beam is welded to the casing and lagging is installed.

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Caisson design

10

Caisson is another name for bored piles. Caissons generally tend to be large-diameter bored piles. First let us discuss caissons in sandy soils.

10.1 Caissons in sandy soils

10.1.1 AASHTO method

AASHTO adopts the method proposed by Reese et al. (1976). Ultimate end bearing capacity in caisson placed in sandy soils is given by

$$Q_u = q_t A$$
$$q_t = 1.20\ N \quad \text{ksf}\ (0 < N < 75)$$

Metric units, $q_t = 57.5\ N$ kPa $(0 < N < 75)$.
A = Cross-sectional area at the bottom of the shaft.
For all "N" values above 75, $q_t = 90$ ksf (in psf units) $q_t = 4310$ kPa (in metric units).
N, standard penetration-test value (blows/ft.).

10.2 Belled caisson design

Belled caissons are used to increase the end bearing capacity. Unfortunately, one loses the skin friction in the bell area since experiments have shown that the skin friction in the bell to be negligible (Reese et al. (1976)). In addition AASHTO suggests excluding the skin friction in a length equal to the shaft diameter above the bell (Fig. 10.1).

In addition, one may lose approximately 5 ft. (1.5 m) of shaft skin friction at the top of the shaft due to desiccation of clay.

10.2.1 Belled caissons placed in clay

Majority of belled caissons are placed in clay. It is not easy to construct a bell in sandy soil.

Ultimate capacity of belled caissons is given by the equation:

ultimate capacity of the belled caisson = ultimate end bearing capacity + ultimate skin friction

$$P_u = Q_u + S_u$$

$$Q_u = 9\ c\ \text{area of the bell}$$

Pile Design and Construction Rules of Thumb

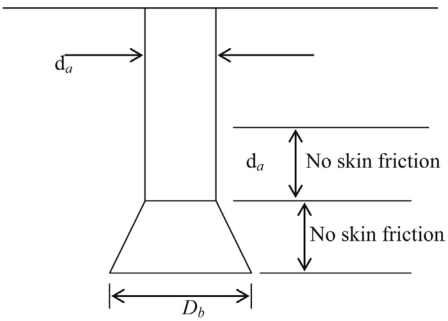


Figure 10.1 Belled caisson (Reese et al., 1976).

Here, c, cohesion; area of the bell $= \pi \times d_b^2/4$ (d_b, bottom diameter of the bell); $S_u = \alpha \times c \times$ perimeter surface area of the shaft (ignore the bell); $S_u = \alpha \times c \times (\pi \times d \times L)$; d, diameter of the shaft; and L, length of the shaft portion.

Design example 1

Find the allowable capacity of the belled caisson shown. The diameter of the bottom of the bell is 4 m and the height of the bell is 2 m. Diameter of the shaft is 1.8 m and the height of the shaft is 10 m. Cohesion of the clay layer is 100 kN/m^2. Adhesion factor (α) was found to be 0.5.

Solution

Step 1: ultimate caisson capacity (P_u) = $Q_u + S_u - W$.

$$Q_u = \text{ultimate end bearing capacity}$$
$$= 9 \times c \times (\text{area of the bottom of the bell})$$
$$= 9 \times 100 \times \left(\pi \times \frac{4^2}{4}\right) = 11{,}310\,\text{kN}$$

S_u, ultimate skin friction; and W, weight of the caisson.

Step 2: find the ultimate skin friction.

$$\text{ultimate skin friction}(S_u) = \alpha \times c \times (\pi \times d \times L)$$
$$\text{ultimate skin friction}(S_u) = 0.5 \times 100 \times (\pi \times 1.8 \times 10) = 2827\,\text{kN}$$

Step 3: find the weight of the caisson.
Assume the density of concrete to be 23 kN/m^3.

$$\text{Weight of the shaft} = \left(\pi \times \frac{d^2}{4}\right) \times 10 \times 23$$
$$= \left(\pi \times \frac{1.8^2}{4}\right) \times 10 \times 23\,\text{kN} = 585.3\,\text{kN}$$

Find the weight of the bell.

Average diameter of the bell (d_a) = (1.8 + 4)/2 = 2.9 m.
Use the average diameter of the bell (d_a) to find the volume of the bell.
The volume of the bell = $\pi \times d_a^2/4 \times h$, where h, height of the bell.
$\pi \times 2.9^2/4 \times 2 = 13.21\ m^3$.
Weight of the bell = volume × density of concrete = 13.21 × 23 = 303.8 kN

Step 4: ultimate caisson capacity (P_u) = $Q_u + S_u - W$.

Allowable caisson capacity = 11,310/FOS + 2827/FOS − 585.3 − 303.8

Assume a factor of safety of 2.0 for end bearing and 3.0 for skin friction. Since the weight of the caisson is known fairly accurately no safety factor is needed.

Allowable caisson capacity = 11,310/2.0 + 2827/3.0 − 585.3 − 303.8

Allowable caisson capacity = 5710 kN.

Design example 2

Find the allowable capacity of the shaft in example 1, assuming a bell was not constructed.

Solution

Step 1: ultimate end bearing capacity (Q_u) = $9 \times c \times (\pi \times 1.8^2/4)$.

Since there is no bell new diameter at the bottom is 1.8 m.
Ultimate end bearing capacity (Q_u) = $9 \times 100 \times (\pi \times 1.8^2/4)$ = 2290 kN.

Step 2: ultimate skin friction (S_u) = $\alpha \times c \times (\pi \times 1.8) \times 12$.

New height of the shaft is 12 m since a bell is not constructed.
Ultimate skin friction (S_u) = $0.5 \times 100 \times (\pi \times 1.8) \times 12$ = 3393 kN.

Step 3: find the weight of the shaft.

W, weight of the shaft = $(\pi \times d^2/4) \times L \times$ density of concrete
W, weight of the shaft = $(\pi \times 1.8^2/4) \times 12 \times 23$ = 702.3 kN.

Step 4: find the allowable caisson capacity.

Allowable caisson capacity = Q_u/FOS + S_u/FOS − W.

Assume a factor of safety of 2.0 for end bearing and 3.0 for skin friction. Since the weight of the caisson is known fairly accurately no safety factor is needed.

Allowable caisson capacity = 2290/2.0 + 3393/3.0 − 702.3 kN = 1573.7 kN.

In the aforementioned example, we found the allowable capacity with the bell to be 5710 kN, significantly higher than the straight shaft.

Reference

Reese, et al., 1976. Behaviour of drilled piers under axial loading. Journal of Geotechnical Engineering Division, ASCE 102 (5), 493–510.

Piles in rock

11

First let us spend some time understanding rock engineering. Rocks are not monolithic bodies. They have joints or cracks, different bedding planes, different minerals, and also different constituents.

11.1 Rock joints

A rock joint is basically a fracture in the rock mass. Most rocks consist of joints. Joints could occur in the rock mass due to many reasons:

- Earthquakes: major earthquakes could shatter the bedrock and create joints.
- Plate tectonic movements: continents move relative to each other. When they collide, the bedrocks fold and joints get created.
- Volcanic eruptions.
- Generation of excessive heat in the rock.

Depending on the location of the bedrock, the number of joints in a core run could vary. Some core runs contain a few joints while some other core runs may contain dozens of joints (Fig. 11.1).

11.1.1 Joint set

When a group of joints are parallel to each other, that group of joints is called a *joint set*.

11.1.2 Joint filler materials

Some joints are filled with matter. Typical joint filler materials are:

- Sands: sands occur in joints where there is high-energy flow (or high velocity).
- Silts: silts indicate the flow to be less energetic.
- Clays: clay inside joints indicates stagnant water in joints.

Joint filler material information would be very useful in interpreting Packer test data. It is known that filler material could clog up joints and reduce the flow rate with time.

11.1.3 Rock joint types

Most joints can be divided into two types:

- extensional joints (joints due to tensile failure)
- shear joints

Pile Design and Construction Rules of Thumb

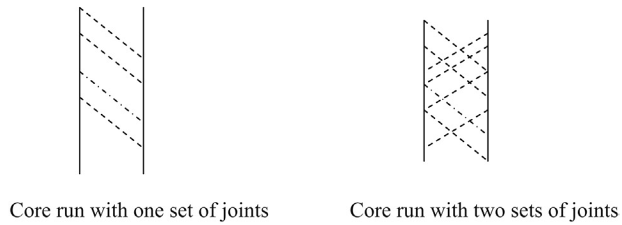


Figure 11.1 Core runs.

Slickensided (very smooth) joint surfaces indicate shearing. Smooth planar joints also most probably could be shear joints.

11.1.4 Core loss information

It has been said by experts that core loss information is more important than the rock core information. Core loss location may not be obvious in most cases. Coring rate, color of return water, and arrangement of core in the box can be used to identify the location of the core loss (Fig. 11.2).

Core loss occurs in weak rock or in highly weathered rock.

11.1.5 Fractured zones

A fracture log would be able to provide fractured zones. Fracture logs of each boring can be compared to check for joints.

11.1.6 Drill water return information

The engineer should be able to assess the quantity of returning drill water. Drill water return could vary from 90% to 0%. This information could be very valuable in determining weak rock strata. Typically drill water return is high in sound rock.

11.1.7 Water color

Color of returning drill water can be used to identify the rock type.

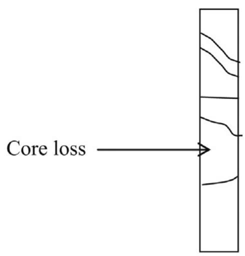


Figure 11.2 Core loss.

Figure 11.3 Rock joint.

11.1.8 Rock joint parameters

11.1.8.1 Joint roughness

Joint surface could be rough or smooth. Smooth joints could be less stable than rough joints

11.1.8.2 Joint alteration

It includes alterations to the joint, such as color, filling of materials, and so on.

11.1.8.3 Joint filler material

Some joints could be filled with sand while some other joints could get filled with clay. Smooth joints filled with sand may provide additional friction. However, rough joints filled with clay may reduce the friction in the joint.

11.1.8.4 Joint stains

Joint stains should be noted. Stains could be due to groundwater and various other chemicals (Fig. 11.3).

11.1.9 Joint types

11.1.9.1 Extensional joints

Joints that formed due to tension or pulling apart.

11.1.9.2 Shear joints

Joints that occurred due to shearing (Fig. 11.4).

11.2 Dip angle and strike

Each set of joints would have a dip angle and a strike (Fig. 11.5).

11.2.1 Joint plane

EBCF is the joint plane in Fig. 11.5.

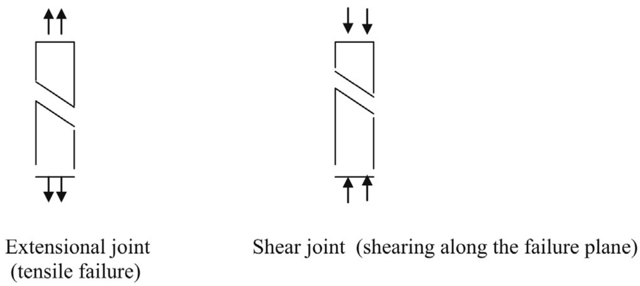


Figure 11.4 Joint types.

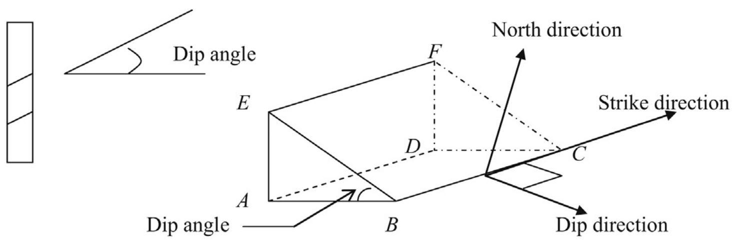


Figure 11.5 Dip and strike.

11.2.2 Dip angle

The angle between the joint plane (*EBCF*) and the horizontal plane (*ABCD*). Dip angle is easily measured in the field.

11.2.3 Strike

Strike line is the horizontal line *BC* as shown in the figure.

11.2.4 Strike direction

Right-hand rule is used to obtain the strike direction.

- Open your right hand and face the palm down. Mentally lay the palm on the joint plane.
- Point four fingers (except the thumb) along the downward direction of the slope.
- The direction of the thumb indicates the strike direction.
- The clockwise angle between the strike direction and the North direction is called the strike angle. *ABCD* plane and North direction are in the same horizontal plane.

11.2.5 Dip direction

Dip direction is the direction of the downward slope. Dip direction is different than the dip angle. Strike and dip directions are perpendicular to each other.

11.2.6 Notation

Dip angle and strike angle are written as 35/100.

The first value indicates the dip angle (not the dip direction). The second value indicates the strike direction measured clockwise from the North. Dip angle is always written with two digits, while strike direction is always written with three digits. A joint plane with a dip angle of 40 degrees and a 70-degree strike angle is written as (40/70) degrees.

11.3 Oriented rock coring

11.3.1 Oriented coring procedure

Step 1: a knife is installed in the core barrel to create a mark in the rock core. This mark is known as the scribe mark.
Step 2: the knife is installed in such a manner that a line drawn through the scribe mark and the center of the core would point toward the North direction (line *AB*). The driller would use a magnetic compass, prior to coring and locate the North direction. Then he or she would be able to locate the knife (Fig. 11.6).
Step 3: draw a horizontal line going through point C at the joint (line *CD*). Both lines (line *AN* and line *CD*) are in horizontal planes.
Step 4: measure the clockwise angle between line *AN* (which is the North direction) and line *CD*. This angle is the strike angle of the joint.

11.3.2 Oriented coring procedure (summary)

- The driller finds the North direction using a compass. Then the driller places the knife along the North direction. The line connecting the knife-point (point *A*) and the center of the core will be the North–South line (line *AN*).

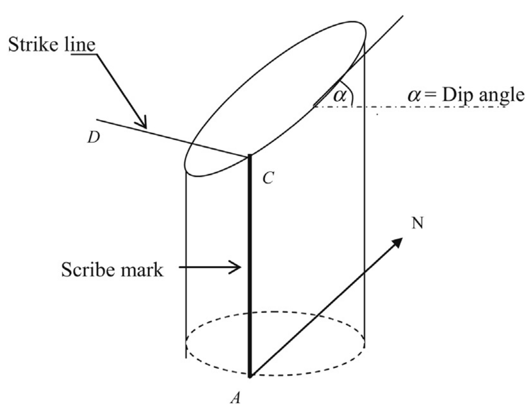


Figure 11.6 Oriented cores.

- Rock coring is conducted. A scribe mark will be created along the rock core (the rock core does not rotate during coring).
- After the rock core is removed from the hole, a horizontal line is drawn along the plane of the joint (line *CD*). The clockwise angle between the line *AN* and the line *CD,* is the strike angle.

11.4 Oriented core data

- Oriented coring would produce a *dip angle* and a *strike angle* for each joint.
- A “joint” can be represented by one point known as the pole (Fig. 11.7).

11.4.1 Concept of pole

Step 1: the joint plane is drawn across the sphere A.
Step 2: a perpendicular line is drawn (line *B*) to the joint plane.
Step 3: the point where line *B* intersects the sphere is known as the pole.
Step 4: a vertical line is dropped from the pole to obtain the pole point.

Theoretically there are two poles for any given joint (one in the lower hemisphere and the other one in the upper hemisphere). The pole in the lower hemisphere is always selected.

As you can see, it is not easy to obtain the pole point for a given joint. For that purpose, one needs a sphere and has to go through a lot of trouble. There are charts available to obtain the pole point for any given joint.

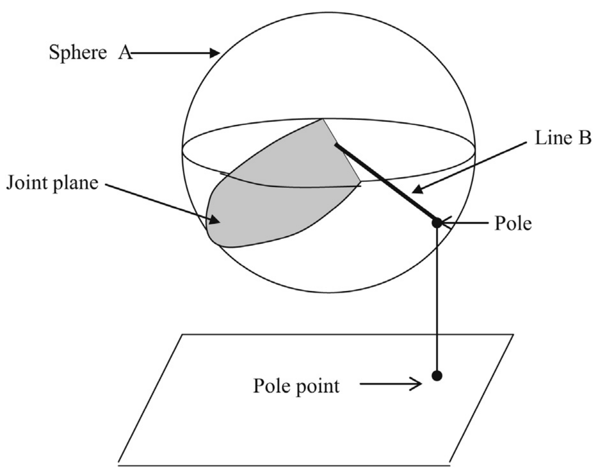


Figure 11.7 Oriented core data.

11.5 Rock mass classification

Who is a better athlete?

Athlete A: long jump: 23 ft., runs 100 m in 11 s, high jump 6.5 ft.
Athlete B: long jump: 26 ft., runs 100 m in 13 s, high jump 6.4 ft.

Athlete A is very weak in long jump but very strong in 100 m. Athlete B is very good in long jump, but not very good in 100 m. Athlete A has a slight edge in high jump.

It is not easy to determine which athlete is better. Due to this reason, the Olympic committee came up with a marking system for the decathlon. The best athlete is selected based on the marking system.

A similar situation exists in rock types. Let us look at an example.

Example: a geotechnical engineer has the choice to construct a tunnel in either rock type A or rock type B.

Rock type A: average rock quality designation (*RQD*) = 60%, joints are smooth, joints are filled with clay.
Rock type B: average *RQD* = 50%, joints are rough, joints are filled with sand.

Rock type A has a higher *RQD* value. On the other hand, Rock type B has rougher joints. Smooth joints in rock type A are not favorable for geotechnical engineering work. Joints in rock type A are filled with clay, while joints in rock type B are filled with sands. Based on the aforementioned information, it is not easy to select a candidate, since both rock types have good properties and bad properties. Early engineers recognized the need of a classification system to determine the better rock type. Unfortunately, more than one classification system exists. These systems are named *Rock Mass Classification Systems*.

The popular Rock Mass Classification Systems are

- Terzaghi Rock Mass Classification System (rarely used)
- Rock Structure Rating (RSR) method.
- Rock Mass Rating system (RMR).
- Rock Tunneling Quality Index (better known as the "*Q*" system)

Recently, the "*Q*" system has gained popularity over other systems.

11.6 Q system

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

where Q, rock quality index; J_n, joint set number; J_r, joint roughness number; J_a, joint alteration number; J_w, joint water reduction factor; and SRF, stress reduction factor.

11.6.1 RQD

To obtain *RQD*, select all the rock pieces longer than 4 in. in the rock core. Measure the total length of all the individual rock pieces greater than 4 in. This length is given as a percentage of the total length of the core.

Total length of the core = 60 in.

Total length of all the pieces longer than 4 in. = 20 in.

$RQD = 20/60 = 0.333 = 33.3\%$
RQD (0–25%) = very poor
RQD (25–50%) = poor
RQD (50–75%) = fair
RQD (75–90%) = good
RQD (90–100%) = excellent

11.6.1.1 Joint set number (J_n)

Find the number of joint sets. When a group of joints have the same dip angle and a strike angle, that group is known as a joint set. In some cases many joint sets exit. Assume there are eight joints in the rock core with the following dip angles: 32, 67, 35, 65, 28, 64, 62, 30, and 31. It is clear that there are at least two joint sets. One joint set has a dip angle approximately at 30 degrees while the other joint set has a dip angle of approximately at 65 degrees.

The *Q* system allocates the following numbers:
Zero joints: $J_n = 1.0$
One joint set: $J_n = 2$
Two joint sets: $J_n = 4$
Three joint sets: $J_n = 9$
Four joint sets: $J_n = 15$
Higher J_n number indicates a weaker rock for construction.

11.6.1.2 Joint roughness number (J_r)

When subjected to stress, smoother joints may slip and failure could occur before rougher joints. Due to this reason, joint roughness plays a part in rock stability (Fig. 11.8).

Note: Slippage occurs along a smoother joint at a lower load (*P*).

How to obtain the joint roughness number?

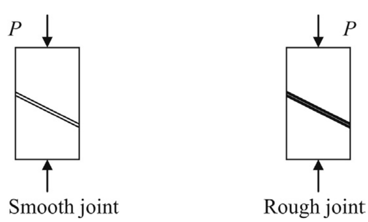


Figure 11.8 Smooth and rough joints.

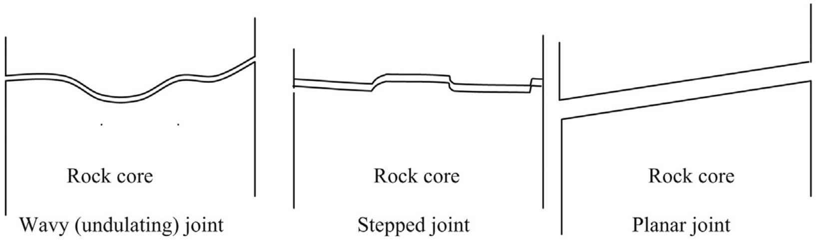


Figure 11.9 Wavy, stepped, and planar joints.

Step 1: there are three types of joint surface profiles:

- Wavy (undulating) joint surface profiles.
- Stepped joint surface profiles.
- Planar joint surface profiles.

No joint surface is either 100% planar, stepped, or wavy. Select the type that best describes the joint surface (Fig. 11.9).

Step 2: feel the joint surface and categorize into one of the following types:

- Rough: if the surface feels rough.
- Smooth: if the surface feels smooth.
- Slickensided: slickensided surfaces are very smooth and slick. Slickensided surfaces occur when there was a shear movement along the joint. In some cases the surface could be polished. One would notice polished (shining) patches. These polished patches indicate shear movement along the joint surface. The name slickensided was given to indicate slick surfaces.

Step 3: use Table 11.1 to obtain J_r.

- Rock joints with stepped profiles provide the best resistance against shearing. From Table 11.1 a smooth joint with a stepped profile would be better than a rough joint with a planar profile.

Table 11.1 Joint roughness coefficient (J_r) (Hoek et al. (1995))

Joint profile	Joint roughness	J_r
Stepped	Rough	4
	Smooth	3
	Slickensided	2
Undulating	Rough	3
	Smooth	2
	Slickensided	1.5
Planar	Rough	1.5
	Smooth	1.0
	Slickensided	0.5

Table 11.2 Joint alteration number (J_a) (Hoek et al. (1995))

Description of filler material	J_a
A. Tightly healed with a nonsoftening impermeable filling seen in joints (*quartz or epidote*)	0.75
B. Unaltered joint walls, no filler material seen (surface stains only)	1.0
C. Slightly altered joint walls, nonsoftening mineral coatings are formed, sandy particles, clay, or disintegrated rock seen in the joint	2.0
D. Silty or sandy clay coatings, small fraction of clay in the joint	3.0
E. Low-friction clay in the joint (kaolinite, talc, and chlorite are low-friction clays)	4.0

11.6.1.3 Joint alteration number (J_a)

Joints get altered with time. Joints could get altered due to material filling inside them. In some cases filler material could cement the joint tightly. In other cases, filler material could introduce a slippery surface creating a much more unstable joint surface (Table 11.2).

It is easy to notice a tightly healed joint. In this case use $J_a = 0.75$. If the joint has not undergone any alteration other than surface stains use $J_a = 1.0$. If there are sandy particles in the joint, then use $J_a = 2.0$. If there is clay in the joint then use $J_a = 3.0$. If clay in the joint can be considered low friction, then use $J_a = 4.0$. For this purpose, clay type existing in the joint needs to be identified.

11.6.1.4 Joint water reduction factor (J_w)

J_w cannot be obtained from boring data. A tunnel in the rock needs to be constructed to obtain J_w. Usually data from previous tunnels constructed in the same formation is used to obtain J_w. Another option is to construct a pilot tunnel ahead of the real tunnel (Table 11.3).

11.6.1.5 Stress reduction factor

SRF cannot be obtained from boring data. A tunnel in the rock needs to be constructed to obtain *SRF* as in the case of J_w. Usually data from previous tunnels constructed in the same formation is used to obtain *SRF*. Another option is to construct a pilot tunnel ahead of the real tunnel.

All rock formations have weak zones. A weak zone is a region in the rock formation, which has a low *RQD* value. Weak zones may have weathered rock or different rock types (Table 11.4).

Table 11.3 **Joint water reduction factor (J_w) (Hoek et al. (1995))**

Description	Approximate water pressure (kg/cm^2)	J_w
A. Excavation (or the tunnel) is dry, no or slight water flow into the tunnel.	Less than 1.0	1.0
B. Water flows into the tunnel at a medium rate (water pressure 1.0–2.5 kgf/cm^2), joint fillings get washed out occasionally due to water flow.	1.0–2.5	0.66
C. Large inflow of water into the tunnel or excavation, the rock is competent and joints are unfilled (water pressure 1.0–2.5 kgf/cm^2)	2.5–10.0	0.5
D. Large inflow of water into the tunnel or excavation, the joint filler material gets washed away (water pressure 2.5–10 kgf/cm^2).	Greater than 10	0.33
E. Exceptionally high inflow of water into the tunnel or excavation (water pressure > 10 kgf/cm^2)	Greater than 10	0.1 to 0.2

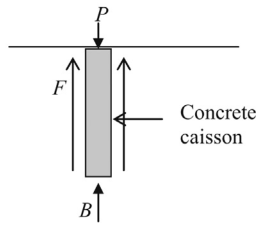
Table 11.4 **Stress reduction factor**

A. More than one weak zone occurs in the tunnel, in this case use $SRF = 10.0$
B. A single weak zone of rock with clay or chemically disintegrated rock (excavation depth < 150 ft.). Use $SRF = 5.0$
C. A single weak zone of rock with clay or chemically disintegrated rock (excavation depth > 150 ft.). Use $SRF = 2.5$
D. More than one weak zone of rock without clay or chemically disintegrated rock. Use $SRF = 7.5$
E. A single weak zone of rock without clay or chemically disintegrated rock (excavation depth < 150 ft.). Use $SRF = 5.0$.
F. A single weak zone of rock without clay or chemically disintegrated rock (excavation depth > 150 ft.). Use $SRF = 2.5$.
G. Loose open joints observed. Use $SRF = 5.0$

Design example 1

The average *RQD* of a rock formation was found to be 60%. Two sets of joints have been identified. Most joint surfaces are undulated (wavy) and rough. Most joints are filled with silts and sands. It has been reported that medium inflow of water has occurred during the construction of past tunnels. During earlier constructions a single weak zone containing clay was observed at a depth of 100 ft. Find the *Q* value.

Step 1: $Q = \dfrac{RQD}{J_n} \times \dfrac{J_r}{J_a} \times \dfrac{J_w}{SRF}$



F = skin friction B = end bearing P = applied load

Figure 11.10 Caisson in rock.

$RQD = 60\%$
Since there are two sets of joints, $J_n = 4$
$J_r = 3$ (from Table 11.1, for undulating, rough joints)
Since most joints are filled with silts and sands, $J_a = 2$ (from Table 11.2)
$J_w = 0.66$ (from Table 11.3)
$SRF = 5$ (from Table 11.4)
Hence, $Q = (60/4) \times (3/2) \times (0.66/5) = 2.97$

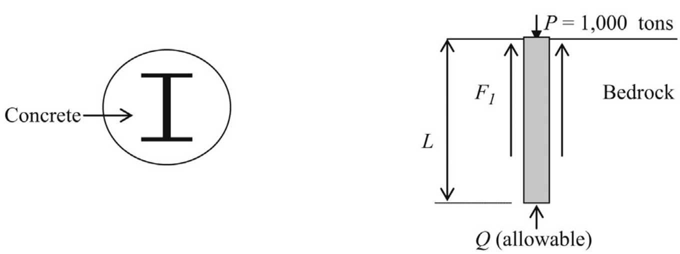
11.7 Caisson design in rock

11.7.1 Caissons under compression

Note: most of the load is taken by skin friction in rock. In most cases, end bearing is less than 5% of the total load. In other words 95% or more load will be carried by skin friction (Fig. 11.10).

Design example 2

Design a concrete caisson with a *W*-section (steel) at the core to carry a load of 1000 tons. Assume the skin friction to be 150 psi and end bearing to be 200 psi (Fig. 11.11).



Following parameters are given: Ultimate steel compressive strength = 36,000 psi
Ultimate concrete compressive strength = 3,000 psi

Figure 11.11 Concrete caisson.

The following parameters are given:
ultimate steel compressive strength = 36,000 psi
ultimate concrete compressive strength = 3,000 psi

11.7.2 Simplified design procedure

- A simplified design procedure is first explained. In this procedure, composite nature of the section is ignored.

Step 1: structural design of the caisson.

- Assume a diameter of 30 in. for the concrete caisson. Since E (elastic modulus) of steel is much higher than concrete, a major portion of the load is taken by steel. Assume 90% of the load is carried by steel.

Load carried by steel = 0.9 × 1000 tons = 900 tons
Allowable steel compressive strength = 0.5 × 36,000 = 18,000 psi
(NYC Building Code, Table 11.3 recommends a factor of safety of 0.5. Check your local building code for the factor of safety value).

$$\text{Steel area required} = \frac{(900 \times 2{,}000)}{18{,}000} = 100 \text{ in.}^2$$

Check the manual of steel construction for an appropriate W-section.
Use W14 × 342. This section has an area of 101 in.^{2} Dimensions of this section are given in Fig. 11.12.
Step 2: check whether this section fits inside a 30 in. hole.
Distance along a diagonal (Pythagoras theorem) = $(16.36^2 + 17.54^2)^{1/2} = 23.98$ in.
This value is smaller than 30 in. Hence, the section can easily fit inside a 30-in. hole.
Step 3: compute the load carried by concrete.
Concrete area = area of the hole − area of steel = 706.8 − 101 = 605.8

- Allowable concrete compressive strength = 0.25 × ultimate compressive strength = 0.25 × 3000 = 750 psi.

(NYC Building Code, Table 11.3 recommends a factor of safety of 0.25. Engineers should refer to local building codes for the relevant factor of safety values).

- Load carried by concrete = concrete area × 750 psi = 605.8 × 750 lbs = 227.6 tons
- Load carried by steel = 900 tons (computed earlier)

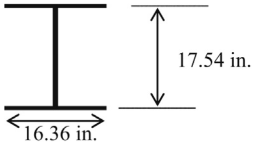


Figure 11.12 H-Section.

- Total capacity of the caisson = 900 + 227.6 = 1127.6 tons > 1000 tons

Note: the designer can start with a smaller steel section to optimize the given value.
Step 4: compute the required length (L) of the caisson.

- The skin friction is developed along the perimeter of the caisson.
- Total perimeter of the caisson = $\pi \times$ (diameter) $\times$ (length) = $\pi \times D \times L$.
- Total skin friction = $\pi \times 30 \times L \times$ unit skin friction of rock (in this case 150 psi).
- Total skin friction = $\pi \times 30 \times L \times 150$ lbs ("L" should be in inches).
  Design the caisson so that 95% of the load is carried by skin friction. 0.95 × 1000 tons = 950 tons
- Hence, the total load carried through skin friction = 950 tons.
  Total skin friction = $\pi \times 30 \times L \times 150$ = 950 tons = 950 × 2000 lbs.
  Length (L) of the caisson required = 134 in. = 11.1 ft.

Design example 3

The following parameters are given (Fig. 11.13):

Caisson diameter = 4 ft.
Compressive strength of steel = 36,000 psi
$E_r/E_c = 0.5$ (E_r; elastic modulus of rock; E_c; elastic modulus of concrete)
Cohesion of the bedrock = 24,000 psf
Adhesion coefficient (α) for rock = 0.5
Adhesion coefficient (α) for clay = 1.0

Solution

Step 1: compute the ultimate end bearing capacity.

q_u; ultimate end bearing strength of the bedrock = $N_c \times$ cohesion; $N_c = 9$

Hence, $q_u = 9 \times$ cohesion = 216,000 psf

Ultimate end bearing capacity (Q_u) = area $\times q_u$

$Q_u = (\pi \times 4^2/4) \times 216{,}000 = 1{,}357$ tons

Allowable end bearing capacity ($Q_{\text{allowable}}$) = 1,357/3 = 452 tons.

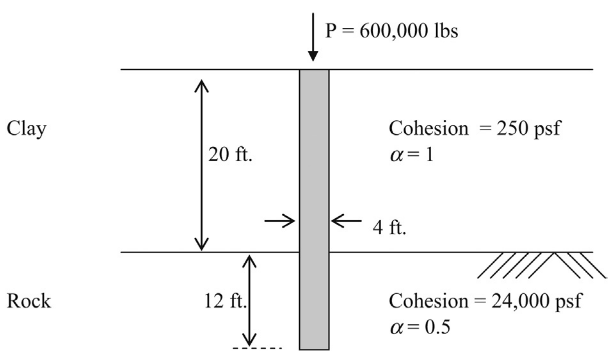


Figure 11.13 Pile and soil strata.

Table 11.5 End bearing ratio (n) (Osterberg and Gill (1973))

$E_r/E_c = 0.5$		$E_r/E_c = 1.0$		$E_r/E_c = 2.0$		$E_r/E_c = 4.0$	
L/a	n	L/a	n	L/a	n	L/a	n
1	0.5	1	0.48	1	0.45	1	0.44
2	0.28	2	0.23	2	0.20	2	0.16
3	0.17	3	0.14	3	0.12	3	0.08
4	0.12	4	0.08	4	0.06	4	0.03

Step 2: compute the ultimate skin friction.

Ultimate unit skin friction per unit area within the rock mass (f) = $\alpha.C = \alpha.24{,}000$.

Adhesion coefficient (α) = 0.5

Hence, ultimate unit skin friction (f) = 0.5 $\times$ 24,000 = 12,000 psf.

Assuming a FOS of 3.0; allowable unit skin friction per unit area within the rock mass ($f_{allowable}$) = 12,000/3.0 = 4,000 psf = 2 tsf.

Step 3: find the skin friction within the soil layer.

The skin friction generated within the soil layer can be calculated as in a pile.

Soil skin friction (f_{soil}) = $\alpha.C$; α = adhesion factor

(f_{soil}) = 1.0 $\times$ 250 = 250 psf.

Skin friction mobilized along the pile shaft within the clay layer = (f_{soil}) $\times$ perimeter = 250 $\times$ ($\pi \times d$) $\times$ 20 = 62,800 lbs.

Allowable skin friction = 62,800/3.0 = 20,900 lbs = 10 tons (factor of safety of 3.0 is assumed).

Load transferred to the rock (F) = (P − 20,900) lbs = 600,000 − 20,900 = 579,100 lbs.

Note: it is assumed that allowable skin friction within soil is fully mobilized.

Step 4: load transferred to the rock.

The load transferred to the rock is divided between the total skin friction (F_{skin}) and the end bearing at the bottom of the caisson (Q_{base}).

Here, f_{skin}, unit skin friction mobilized within the rock mass; F_{skin}, total skin friction = f_{skin} $\times$ perimeter area within the rock mass; q_{base}, end bearing stress mobilized at the base of the caisson; $Q_{base} = q_{base} \times$ area of the caisson at the base.

Step 5: find the end bearing (Q_{base}) and skin friction (F_{skin}) within the rock mass.

End bearing ratio (n) is defined as the ratio between the end bearing load of the rock mass (Q_{base}) and the total resistive force mobilized ($Q_{base} + F_{skin}$) within the rock mass.

End bearing ratio (n) = $Q_{base}/(Q_{base} + F_{skin})$; "$n$" is obtained from Table 11.5.

Q_{base} = end bearing load generated at the base; F_{skin} = total skin friction generated within the rock mass.

Here, L, length of the caisson within the rock mass; and a = radius of the caisson = 2 ft.

$$\frac{E_r}{E_c} = \frac{\text{Elastic modulus of rock}}{\text{Elastic modulus of concrete}} = 0.5\,(\text{given})$$

Total load transferred to the rock mass = 579,100 lbs = $Q_{base} + F_{skin}$ (see Step 3).
Assume a length (L) of 8 ft (since a = 2 ft.; L/a = 4).
From Table 11.5, for L/a of 4 and E_r/E_c of 0.5, end bearing ratio (n) = 0.12.

$n = 0.12 = Q_{base}/(Q_{base} + F_{skin})$
$0.12 = Q_{base}/579{,}100$

Hence, $Q_{base} = 579{,}100 \times 0.12 = 69{,}492$ lbs = 35 tons
$Q_{allowable} = 452$ tons (see Step 1)
$Q_{allowable}$ is greater than the end-bearing load (Q_{base}) generated at the base
F_{skin} = load transferred to the rock − end bearing load
$F_{skin} = 579{,}100 - 69{,}492 = 509{,}608$ lbs = 255 tons
F_{skin} should be less than $F_{allowable}$
$F_{allowable} = f_{allowable} \times$ perimeter of the caisson within the rock mass
$f_{allowable} = 2$ tsf (see Step 2)
Since a length (L) of 8 ft. was assumed within the rock mass
$F_{allowable} = 2 \text{ tsf} \times (\pi \times 4) \times 8 = 201$ tons
Skin friction generated = 255 tons (see F_{skin})
$F_{allowable}$ is less than the skin friction generated. Hence, increase the pile diameter or length of the pile.

References

Hoek, E., Kaiser, P.K., Bawden, W.F., 1995. Support of Underground Excavations in Hard Rock. A.A Balkeema Publishers, Rotterdam.

Osterberg, J.O., Gill, S.A., 1973. Load Transfer Mechanism for Piers Socketed in Hard Soils or Rock, Proceedings of the Canadian Rock Mechanics Symposium, Montreal 235–261.

Underpinning

12

Underpinning is conducted mainly for two reasons: (1) to stop settlement of structures and (2) to transfer the load to a lower hard stratum.

12.1 Underpinning to stop settlement

Design example 1

Underpin the strip footing shown here to stop further settlement (Fig. 12.1).

Procedure

Step 1: excavate an area as shown in Fig. 12.1 (excavation area is marked *ABCD*). Expose the area under the footing to erect underpinning columns. Structural integrity of the strip footing needs to be assessed prior to excavation of the footing. Normally 10–15% of a strip footing can be excavated in this manner.

Step 2: erect column footings *X* and *Y* as shown. Usually columns are made of reinforced concrete.

Conduct the same procedure for the other end of the strip footing. Usually contractors buttress the excavated footing with timber, during construction of the concrete underpinning columns.

Step 3: the following aspects need to be taken into consideration during the design process.

- Eventually, the total load will be transferred to underpinning piles.
- The central portion of the strip footing may crack due to sagging. If the strip footing is too long, underpinning columns may be necessary at the center of the strip footing as well.

12.2 Pier underpinning

12.2.1 Scenario

A wall supported by a strip footing has developed cracks. These cracks have been attributed to uneven settlement in the strip footing. By conducting boreholes, a previously unnoticed pocket of soft clay was located (Fig. 12.2).

12.2.2 Solution

It is proposed to provide a concrete pier underneath the footing as shown in Fig. 12.3. The pier would extend below the clay layer to the hard soil layers below.

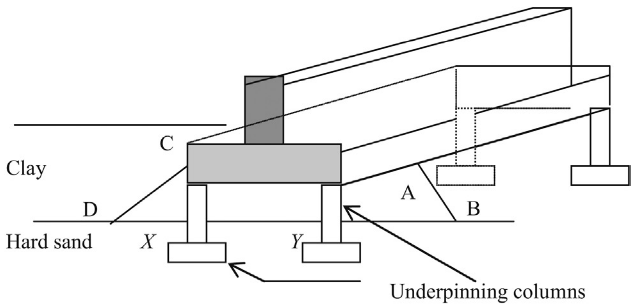


Figure 12.1 Underpinning.

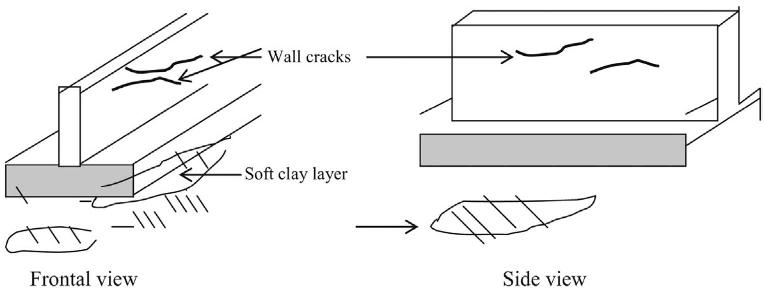


Figure 12.2 Soft soil under a footing.

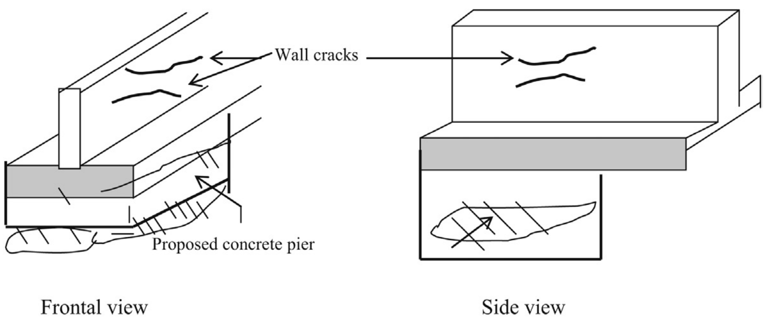


Figure 12.3 Concrete pier.

12.3 Pier underpinning: construction procedure

Basically a pit needs to be dug underneath the footing. In a strip footing, a pit can be dug underneath a small section.

Step 1: construct an approach pit (Fig. 12.4).
Construct an approach pit outside the footing and provide shoring as shown. The pit should extend only for a short length of the footing (in most cases less than 4 ft. is recommended).

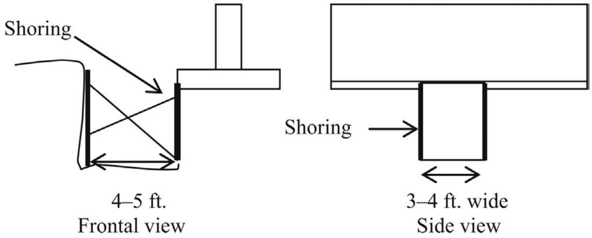


Figure 12.4 Construct an approach pit.

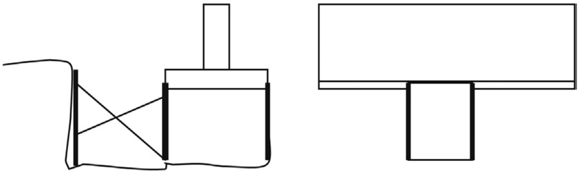


Figure 12.5 Excavate underneath the footing and provide shoring.

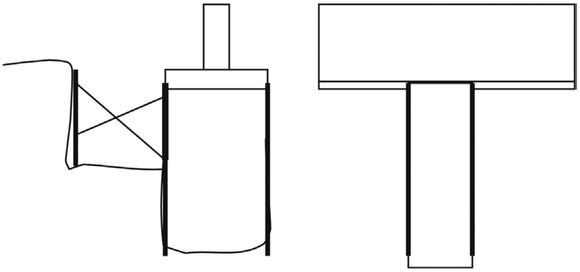


Figure 12.6 Excavate to the stable ground.

Step 2: generally these approach pits are not more than 3–4 ft. wide. In most cases depth can be kept under 4 ft. The purpose of the approach pit is to provide space for the workers to dig underneath the footing and later concrete underneath the footing. All precautions should be taken to avoid any soil failure. Proper shoring should be provided to hold the soil from failing. Excavate underneath the footing and provide shoring (Fig. 12.5).
Step 3: excavate to the stable ground (Fig. 12.6).
The excavation is deepened underneath the footing to extend to the capable soil below. Shoring should be provided to avoid any soil failure.
Step 4: concrete within 2–3 in. from the footing (Fig. 12.7).
Concrete the pit as shown. A 2–3 in. gap is left when concreting. After the concrete is set, this gap is filled with *drypack* (drypack is rammed into the gap). The dry pack would provide a good contact between the footing and the concrete pier. Steel plates also can be used for this purpose. Steel plates can be driven into the gap to obtain a good seating.

- After one section is concreted and underpinned, an approach pit for the next section is constructed and the procedure is repeated. It is important that each section should not be more than 3–4 ft. in length. Removal of soil under a large section of footing should be avoided.

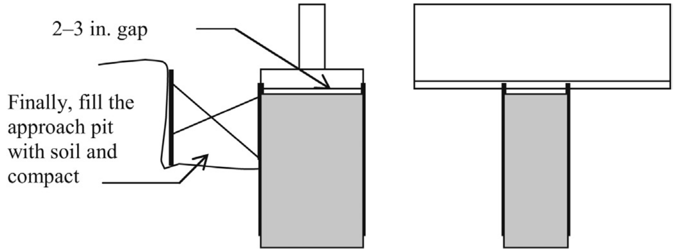


Figure 12.7 Concrete within 2–3 in. from the footing.

12.4 Jack underpinning

Pier underpinning is not feasible when the bearing strata are too deep. Manual excavation and construction of a pier will be too cumbersome. In such cases jack underpinning can be a practical solution. A pit is constructed and a pipe pile is jacked underneath the footing. The load is transferred from the soil to the pipe pile.

12.4.1 Jack underpinning of a strip footing

Step 1: construct an approach pit (Fig. 12.8).
Construct an approach pit outside the footing and provide shoring as shown as similar to pier underpinning.
Step 2: excavate underneath the footing and provide shoring (Fig. 12.9).
The excavation is extended below the footing as shown. It is important to make sure that soil would *not fail*.
Step 3: setup the pipe pile and the jack (Fig. 12.10).

- Place the pipe pile first. Then place the steel plate on the pipe pile.
- Now place the jacks.

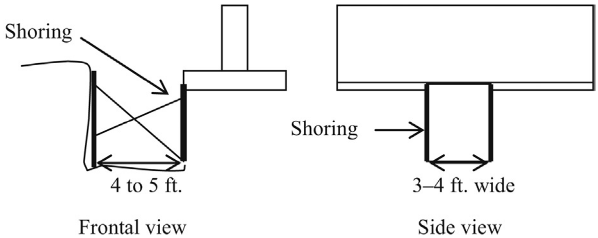


Figure 12.8 Construct an approach pit.

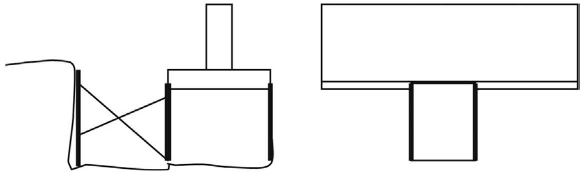


Figure 12.9 Excavate underneath the footing and provide shoring.

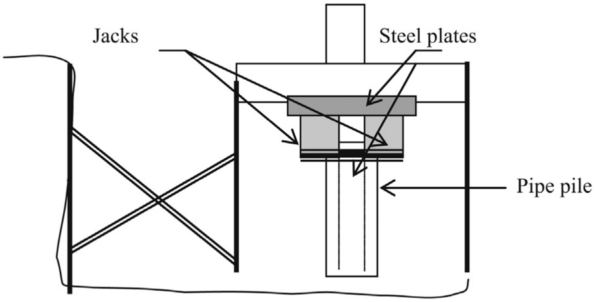


Figure 12.10 Setup the pipe pile and the jack.

- Place steel plates between the footing and the jacks.
- Now extend the jacks. The jacks would push the pipe pile into the ground.

Step 4: push the pipe pile into the ground (Fig. 12.11).

- Extend the jack and push the pipe pile into the ground. The pipe pile would get filled with soil.

Step 5: remove the jacking assembly and clean the soil inside (Fig. 12.12).

- Remove the jacking assembly.
- Remove soil inside the pipe pile.

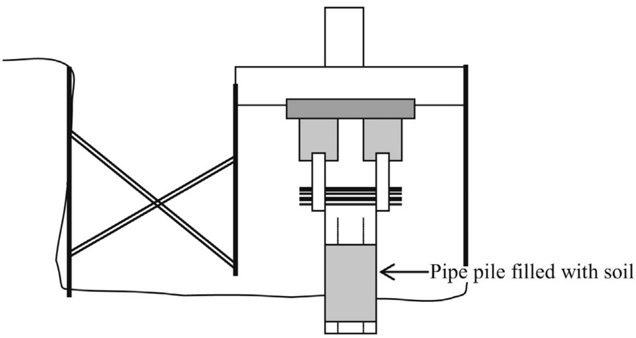


Figure 12.11 Push the pipe pile into the ground.

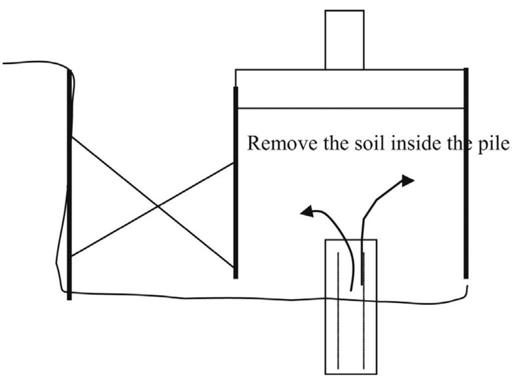


Figure 12.12 Remove the jacking assembly and clean the soil inside.

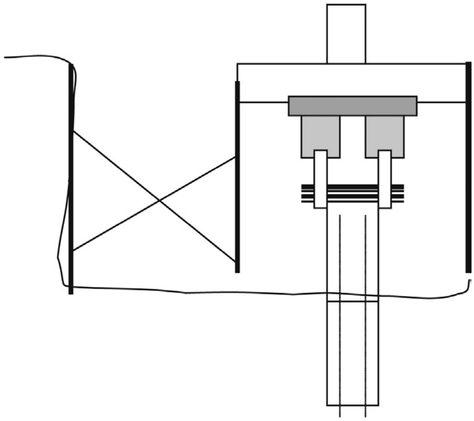


Figure 12.13 Attach another section of pipe and assemble the jacks.

- Removal of soil can be done by hand augers, suction pumps, peel buckets, and water jets.
- Attach another piece of pipe and assemble the jacks.

Step 6: attach another section of pipe and assemble the jacks (Fig. 12.13).

- Extend the jack and push the second pipe pile into the ground.
- Remove the soil inside the pile.
- Repeat the process until the desired depth is reached.
- After the desired depth is reached concrete the pipe pile.
- Wait till the concrete is set and use drypack and steel plates to transfer the footing load to the pile.

Note: instead of open-end pipe piles, "H" sections and "close-end pipe piles" also can be used. Closed end pipe piles may be suitable in soft soil conditions. It may not be feasible to jack a closed end pipe pile into a relatively hard soil layer. "H" sections may be easier to jack than a closed end pipe pile. Extreme caution should be taken not to disturb the existing footing.

12.4.2 Monitoring upward movement of the footing

When the pile has been jacked into the ground, the jacks will be exerting an upward force on the footing. The upward movement of the footing should be carefully monitored. In most cases jacking the piles is conducted while keeping the upward movement of the footing at "zero." If the pile cannot be jacked into the ground without causing damage to the existing footing, a different type of a pile should be used. A smaller-diameter pipe pile or smaller-size "H" pile should be selected in such situations.

12.5 Underpinning with driven piles

Strip footings can be underpinned with driven piles. An approach pit is constructed as before. A pile is driven outside the footing. The footing load is transferred to the pile through a bracket (Fig. 12.14).

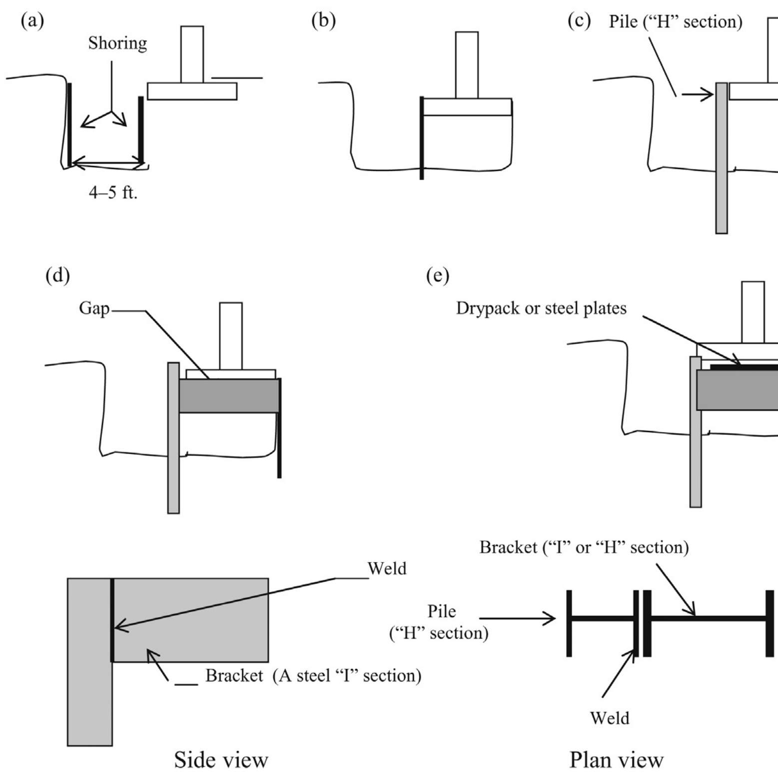


Figure 12.14 Underpinning with driven piles. (a) Construct an approach pit. (b) Dig underneath the footing. (c) Drive a pile outside the footing to the desired depth. (d) Provide a bracket with a gap (Bracket detail shown below). (e) Place drypack or ram steel plates into the gap.

12.6 Mudjacking (underpinning concrete slabs)

Settling concrete slabs can be underpinned using a process known as "mudjacking." In this technique, small holes are drilled and a sand, cement, limestone mixture is pumped underneath the slab to provide support (Fig. 12.15).

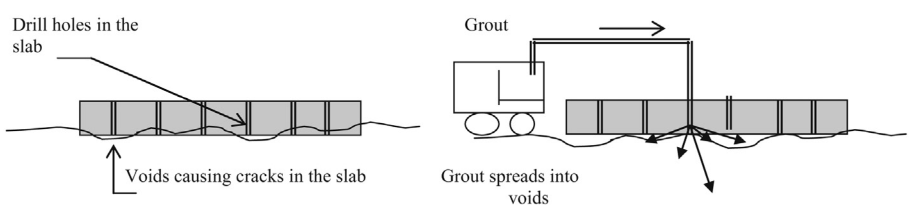


Figure 12.15 Mudjacking (underpinning concrete slabs).

Step 1: drill holes in the concrete slab.
Step 2: pump grout into the holes.

The procedure is simple to understand. The engineer has to determine the following parameters.

12.6.1 Grout pressure

Higher grout pressure could carry grout to greater distances. Voids under the slab could be filled at a faster rate with a high grout pressure.

On the other hand grout pressure has to be low enough, so that grout would not damage nearby existing structures.

12.6.2 Spacing between holes

Greater spacing between holes would require high grout pressure. If the grout pressure has to be maintained at a lower level, spacing between the holes should be reduced. In general, holes are drilled at 3–4 ft. spacing.

12.6.3 Composition of grout

Grout for slab underpinning is mainly created using cement, sand, limestone, and water. Thicker grout can be used for coarser material while a thinner grout needs to be used for fine sands. Silty soils and clays are difficult or impossible to grout.

12.7 Underpinning: case study

- A 26-story building was settling on one side. The building was built on Frankie piles. Frankie piles were constructed by hammering out a concrete base and concreting the top portion of the pile (see chapter: Pile Types).
- The settlement took place rapidly.
- The building was 300 ft. high and the width was relatively small.
- Soil profile of the site is shown. During the boring program, a very soft clay layer occurring on right-hand side of Fig. 12.16 was missed. Due to this reason, pile foundations on that side started to settle.

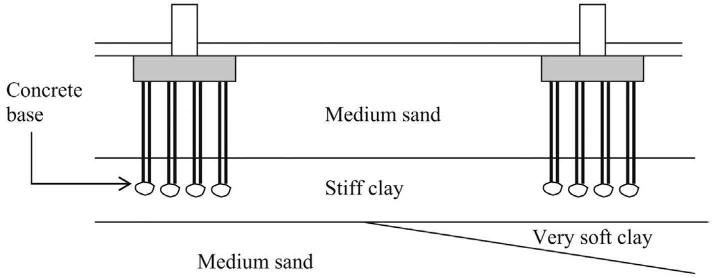


Figure 12.16 Case study.

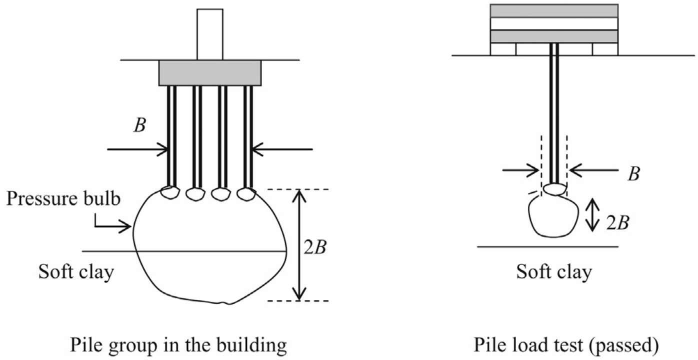


Figure 12.17 Pile group versus single piles.

12.7.1 Pile load tests

- Pile load tests were conducted on the side that had the very soft clay layer. Pile load tests of single piles passed. This could be explained using a pressure bulb (Fig. 12.17).
- Typically, the pressure bulb of a footing extends 2*B* below the bottom of the footing (*B* = width of the footing). The width of the pile group is larger than the width of the concrete bulb of a single pile.
- In the case of the single pile (Fig. 12.3), the pressure bulb does not extend to the soft clay layer below. This is not the situation with the pile group. The stress bulb due to the pile group extends to the soft clay layer. The stressed soft clay consolidated and settled. Success of pile load tests done on single piles could be explained using pressure bulbs.

The engineers were called upon to provide solutions to two problems:

1. Stop further settlement of the building.
2. Bring back the building to its original status.

Stopping further settlement was achieved by ground freezing (Fig. 12.18).

- Ground freezing was conducted by circulating a Brine solution in pipes.
- Ground freezing stopped further settlement.

12.7.2 Bringing the building back to the original position

After further settlement was stopped by freezing the ground, an underpinning program was developed to bring the building back to the original level (Figs. 12.19 and 12.20).

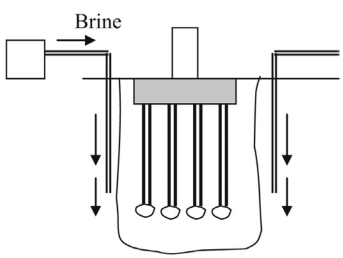


Figure 12.18 Ground freezing.

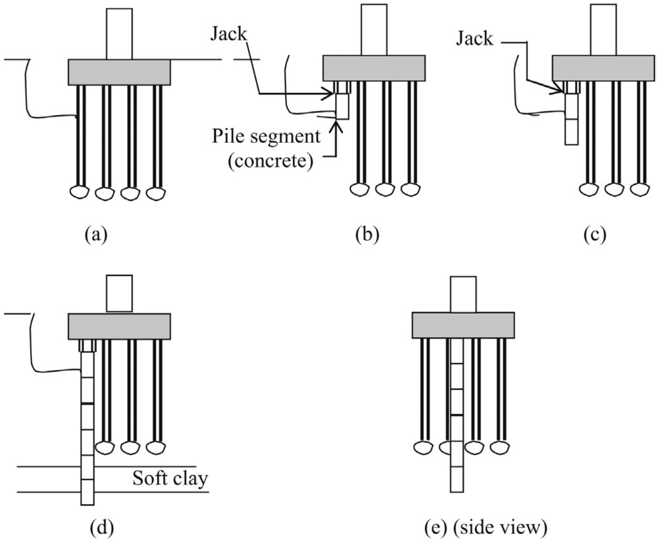


Figure 12.19 Bringing the building back to the original position. (a) An approach pit was constructed as shown. (b) Concrete pile segment was placed and jack was assembled on top of the pile. (c) Concrete pile segment was jacked down using the pile cap above. The engineer had to make sure that the pile cap would be able to resist the load due to jacking. Usually, piles are jacked down one at a time. At any given time only one pile was jacked. (d) The pile segments are jacked down below the soft soil layer. (e) The new pile segments were jacked down between the existing piles without damaging the existing piles. Side view of a jacked down pile is shown in Fig. 12.10.

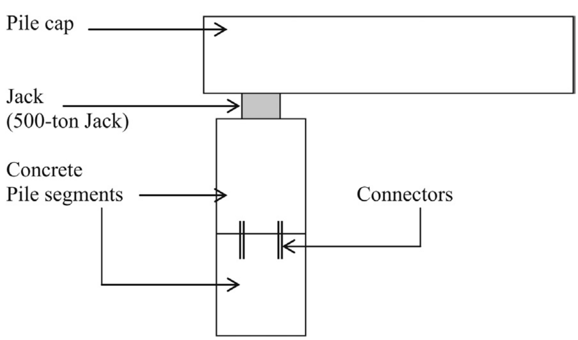


Figure 12.20 Schematic diagram.

Pile settlement

13

13.1 Pile settlement measurement

Measurement of settlement in piles is not straightforward as one would think. When a pile is loaded two things would happen:

- The pile would settle into the soil.
- The pile material would compress due to the load (Fig. 13.1).

In Fig. 13.1, a pile is loaded with a load "*P*" and the settlement at the top (y) was measured. Assume the compression of the pile due to the load to be "c." Besides, "y" is not equal to "s" due to compression of the pile.

Settlement $y = s + c$ (only "y" can be measured).

13.1.1 Why pile compression is difficult to compute?

Unfortunately, the Hook's equation $(FL/Ae) = E$ cannot be used to compute the compression of the pile. This is due to skin friction acting on the pile walls (Fig. 13.2).

- Figs. 13.1 and 13.2 show a column and a pile. At midpoint, a strain gauge is attached to the pile and the column (The strain gauge would measure the strain at the point of contact. The stress at that point is calculated using the measured strain value.).
- The stress at the midpoint of the column = F/A (A = cross-sectional area)
- The stress at the midpoint of the pile = $(F - S)/A$
  (A, cross-sectional area; S, total skin friction above midpoint)
  S = unit skin friction (f) $\times$ pile perimeter $\times$ depth to the midpoint.
- The stress along a column is a constant. The stress along a pile varies with the depth since total skin friction above a given point depends upon the depth to that point measured from the top of the pile.
- Due to skin friction, Hook's equation $FL/Ae = E$ cannot be used directly for piles.

13.2 Method to compute the settlement and pile compression

13.2.1 Preparation of pile for settlement measurements

Step 1: elevation view of an H-pile with a welded pipe is shown in Figs. 13.3–13.5.
Step 2: q rod is inserted into the welded pipe (Fig. 13.5b).
Step 3: the pile is loaded. When the pile is loaded, the pile and the welded pipe would compress due to the load. The rod is not loaded. The rod is freely sitting inside the welded pipe. The length of the rod will not change.

Pile Design and Construction Rules of Thumb

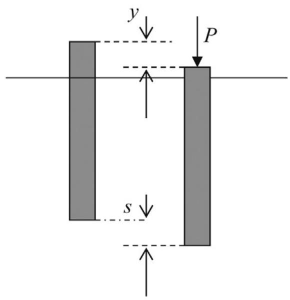


Figure 13.1 Pile settlement measurement.

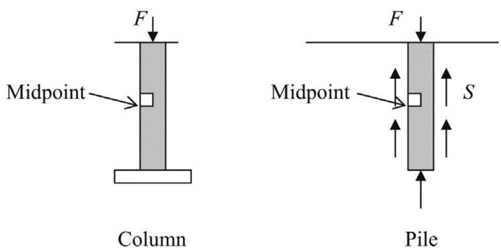


Figure 13.2 Pile compression.

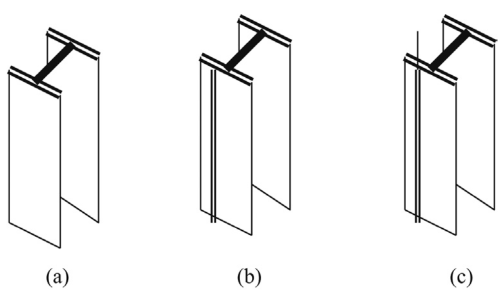


Figure 13.3 Preparation of pile for settlement measurements. (a) Regular H-pile is shown. (b) A pipe is welded to the H-pile. (c) A rod is inserted into the pipe (the procedure can be done for steel pipe piles, concrete piles, or timber piles).

Step 4: "*y*" and "*x*" can be measured. "*c*" and "*s*" need to be computed.
Step 5: $y = s$. The rod is not compressed. Hence, if the rod goes down by "*s*" at the bottom, that same amount will go down at the top of the rod.
Step 6: Total settlement of the pile at the top (x) = settlement of the pile into the soil (s) + elastic compression of the pile (c):

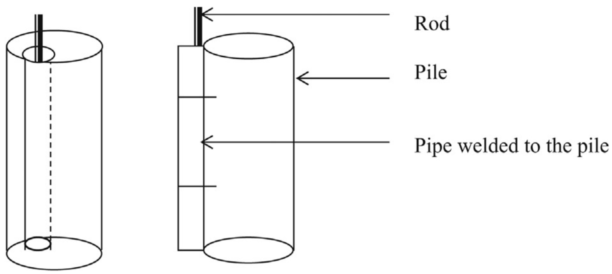


Figure 13.4 Final configuration.

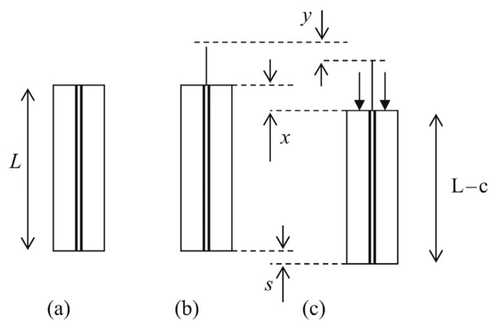


Figure 13.5 Pile compression measurement procedure.

$x = s + c$ (from the given relationship)
$y = s$ (from Step 5)
Hence, $x = y + c$.
x and y can be measured. Hence, "c" can be computed.

13.3 Stiffness of single piles

13.3.1 Stiffness of a spring

Stiffness of single piles (Fig. 13.6).

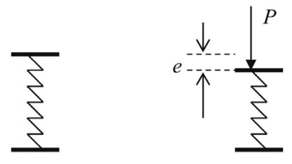


Figure 13.6 Stiffness of single piles.

13.3.2 Stiffness of soil: pile system

Soil settles by a distance of "s" (Fig. 13.7). This settlement occurs due to two reasons:

1. Shortening of the pile due to the load.
2. Settlement of surrounding soil.

In this analysis, settlement due to shortening of the pile due to the load is ignored. Settlement of the pile due to settlement of soil will be analyzed (Fig. 13.8).

Two hypothetical soil layers are selected. When the pile is loaded, the pile would stress the soil immediately next to the pile wall. Hence, the soil immediately next to the wall would settle. Soil particles further away from the pile would feel lesser stress. Hence, these soil particles settle less. Experiments have shown in most cases that soil particles lying at a distance that equals to the length of the pile can be considered to be unstressed.

Assume the shear stress between soil particles and pile wall to be "f" ("f" is the skin friction of the pile). Skin friction is dependent upon the depth in sandy soils.

Shear force acting on a unit length of the pile = $f \times$ pile perimeter = $f \times 2\pi \times r_0$ (r_0 is the radius of the pile, Fig. 13.9).

f, shear stress at the pile wall; f_s, shear stress of soil at a distance "r"

$f \times 2\pi \times r_0 = f_s \times 2\pi \times r$; $f_s = (f \times r_0)/r$

Shear modulus (G) is defined as G = shear stress/shear strain.

(Note: shear modulus (G) = $E/2\,(1 + \nu)$ (E, Young's modulus; ν, Poisson's ratio)

Assume the shear strain at "r" = γ

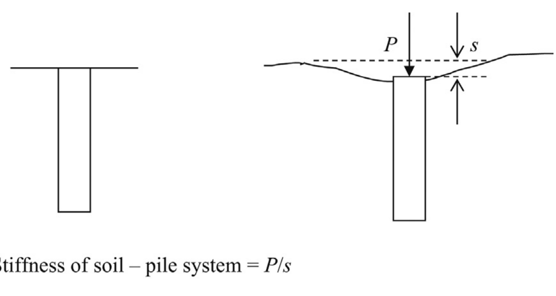


Figure 13.7 Stiffness of soil–pile system.

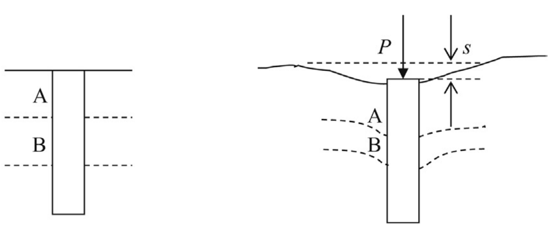


Figure 13.8 Settlement of single piles.

Hence,

$$\frac{\text{shear stress}}{\text{shear strain}} = \frac{fs}{\gamma} = G = \frac{(f \times r_0)}{r \times \gamma}$$

$$\gamma = \frac{(f \times r_0)}{Gr}$$

Assume the settlement at a distance "r" to be "s."

Hence, shear strain = $\gamma = \mathrm{d}s/\mathrm{d}r$

$$\mathrm{d}s = \gamma \mathrm{d}r$$

$$s = \int_{r_0}^{r_1} \frac{(f \times r_0)}{G} \frac{\mathrm{d}r}{r}$$

r_1, influence zone (r_1 varies with the type of soil and pile diameter. Usually, r_1 is taken to be the length of the pile. Shear stress beyond that distance would be negligible in most cases.)

$$s = \int_{r_0}^{r_1} \frac{(f \times r_0)}{G} \frac{\mathrm{d}r}{r} = \frac{(f \times r_0)}{G} \ln\left(\frac{r_1}{r_0}\right)$$

13.4 Settlement of single piles (semiempirical approach)

The following semiempirical procedure can be used to compute the settlement of single piles.

Settlement of single piles can be broken down into three distinct parts:

1. settlement due to axial deformation
2. settlement at pile point
3. settlement due to skin friction

Let us look at each component.

13.4.1 Settlement due to axial deformation

As described in a previous chapter (see chapter: Load Distribution of Piles) it is not an easy task to accurately compute the load distribution along the length of the pile. Skin friction acting on the side walls of the pile absorbs a certain percentage of the load. In some cases, skin friction is so paramount, very little load develops at the tip of the pile. Axial compression of the pile is directly linked to the load distribution of the pile.

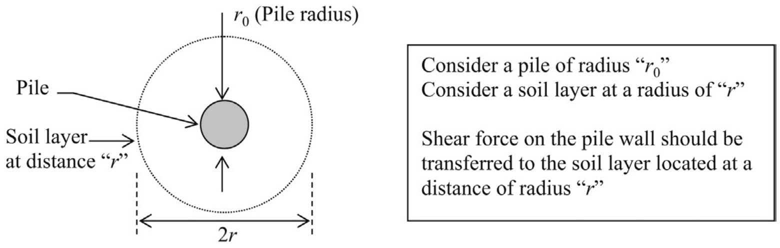


Figure 13.9 Pile and soil.

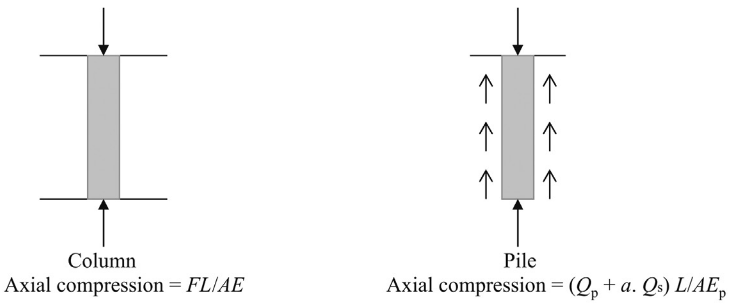


Figure 13.10 Axial compression of columns and piles.

The following semiempirical equation is provided to compute the axial compression of piles:

$S_a = (Q_p + aQ_s)\ L/AE_p$ (Vesic, 1977)

Here, S_a, settlement due to axial compression of the pile; Q_p, load transferred to the soil at tip level; a, 0.5 for clay soils; a, 0.67 for sandy soils; Q_s, total skin friction load; L, length of the pile; A, cross-sectional area of the pile; and E_p, Young's modulus of the pile.

Compare the preceding equation with axial compression in a free-standing column.

Axial compression of a column (s) = FL/AE (elasticity equation, Fig. 13.10).

Skin friction acting along pile walls is the main difference between piles and columns.

13.4.2 Settlement at pile point

Due to the load transmitted at the pile tip, soil just under the pile tip could settle (Fig. 13.11).

Settlement at pile tip (S_t) = $(C_p Q_p)/(Bq_0)$ (Vesic, 1977)

Here, C_p, empirical coefficient; B, pile diameter; q_0, ultimate end bearing capacity; and Q_p, point load transmitted to the soil at pile tip (Table 13.1).

- Bored piles have higher C_p values compared to driven piles. Higher C_p values would result in higher settlement at the pile point.

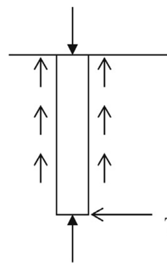


Figure 13.11 Tip settlement.

Table 13.1 Typical values of C_p

Soil type	Driven piles	Bored piles
Dense sand	0.02	0.09
Loose sand	0.04	0.18
Stiff clay	0.02	0.03
Soft clay	0.03	0.06
Dense silt	0.03	0.09
Loose silt	0.05	0.12

13.4.3 Settlement due to skin friction

Skin friction acting along the shaft would stress the surrounding soil. Skin friction acts in an upward direction along the pile. Equal and opposite forces act on the surrounding soil. The force due to pile on surrounding soil would be in downward direction (Fig. 13.12).

When the pile is loaded, the pile would slightly move down. The pile would drag the surrounding soil with it. Hence, pile settlement occurs due to skin friction.

Settlement due to skin friction $(S_{sf}) = (C_s Q_s)/(D q_0)$

Here, C_s, empirical coefficient = $(0.93 + 0.16 \times D/B) \cdot C_p$ (C_p values are obtained from Table 13.1); Q_s, total skin friction load; D, length of the pile; and B, diameter of the pile.

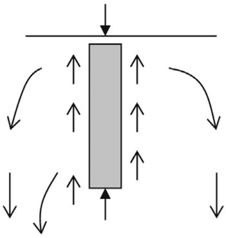


Figure 13.12 Settlement due to skin friction.

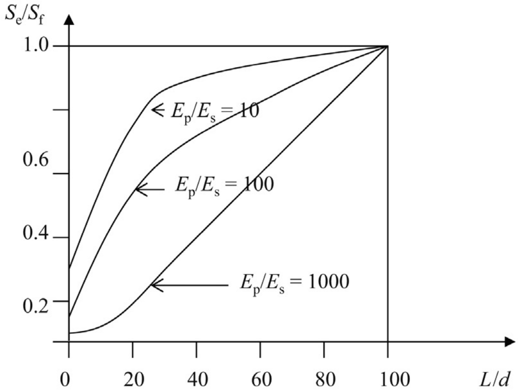
13.5 Pile settlement comparison (end bearing versus floating)

- Most engineers are reluctant to recommend floating piles due to high settlement compared to end bearing piles.
- In this chapter, comparison of settlement of floating piles and end bearing piles are conducted.

13.5.1 Factors that affect settlement

The following parameters affect the settlement of piles (Figs. 13.13 and 13.14):

- L/D ratio (L, length of the pile; and D, diameter of the pile)
- E_p/E_s ratio (E_p and E_s, Young's modulus of pile and soil, respectively)
- Fig. 13.13 shows the relationship between S_e/S_f ratio versus L/d ratio for a given load.



S_e = Settlement of an end bearing pile; S_f = Settlement of a floating pile

Figure 13.13 Settlement graph (Poulos, 1989).

End bearing pile (settlement = S_e) Floating pile (settlement = S_f)

Figure 13.14 End bearing versus floating piles.

Design example

A 30-ft.-long pile with a diameter of 1 ft. is placed on bedrock. E_p/E_s ratio was computed to be 100. If the pile is designed as a floating pile, find the increase of settlement.

Solution

$L/d = 30$; $E_p/E_s = 100$

From Fig. 13.13, $S_e/S_f = 0.62$; $S_f = S_e/0.62 = 1.61\ S_e$.

The floating pile would have a 60% higher settlement than the end bearing pile. The following observations could be made from Fig. 13.13:

- When L/d ratio increases, the settlement difference between two pile types becomes negligible. For instance, when L/d is 100, S_e/S_f ratio reaches 1. This means that it is not profitable to extend long piles all the way to the bed rock.
- When the E_p/E_s ratio increases, S_e/S_f ratio decreases.
  Assume a pile with L/d ratio of 20 and E_p/E_s ratio of 100. This pile would have an S_e/S_f ratio of 0.5. Assume the pile material is changed and new E_p/E_s ratio is 1000. Then S_e/S_f ratio would change to 0.2. This means that it is not much profitable to extend a pile with low E_p/E_s ratio to bedrock.

$L/d = 20$ and $E_p/E_s = 100$; then $S_e/S_f = 0.5$ or $S_f = 2 \times S_e$
$L/d = 20$ and $E_p/E_s = 1000$; then $S_e/S_f = 0.2$ or $S_f = 5 \times S_e$

13.6 Critical depth for settlement

- Pile settlement can be reduced by increasing the length of the pile. It has been reported that increasing the length beyond the critical depth will not cause a reduction of settlement.
  Critical depth for settlement is given by the following equation:
  $L_c/d = [(\pi E_p A_p)/(E_s d^2)]^{1/2}$ (Hull, 1987)
  Here, L_c, critical depth for settlement; E_p, Young's modulus of pile material; E_s, Young's modulus of soil; A_p, area of the pile; and d, pile diameter.
- It should be noted here that critical depth for settlement has no relationship to critical depth for end bearing and skin friction.

13.7 Pile group settlement in sandy soils

The following equation could be adopted to compute pile group settlement in sandy soils:

$S_g = S\,(B/D)^{½}$ (Vesic, 1977)

Here, S_g, settlement of the pile group; S, settlement of a single pile; B, smallest dimension of the pile group; and D, diameter of a single pile.

Design example

A (3 × 3) pile group (with 1 ft. diameter piles) is loaded with 270 tons. It is assumed that the load is uniformly distributed among all piles. Settlement of a single pile due to a load of 30 tons is calculated to be 1 in. Estimate the pile group settlement (Fig. 13.15).

$S_g = S\,(B/D)^{1/2}$

$B = 9$ ft.; $D = 1$ ft.; $S = 1$ in.

$S_g = 1 \times (9/1)^{1/2} = 3$ in.

When a single pile is loaded to 30 tons, the settlement is 1 in. When the same pile is loaded to 30 tons inside a group, the settlement becomes 3 in. (Total load on the group is 270 tons. There are nine piles in the group. Hence, each pile is loaded to 30 tons.).

Larger settlement in a pile group can be attributed to the shadowing effect (Fig. 13.16).

13.8 Long-term pile group settlement in clay soils

Pile groups could undergo consolidation settlement in clay soils. Consolidation settlement of pile groups is computed using the following simplified assumptions:

1. Assume the pile group to be a solid foundation with a depth of 2/3 the length of piles
2. Effective stress at midpoint of the clay layer is used to compute settlement
3. Consolidation equation

$$\text{Consolidation settlement (S)} = C_c/(1+e_0)\cdot H\cdot \log_{10}\cdot(p_0' + \delta_p)/p_0'$$

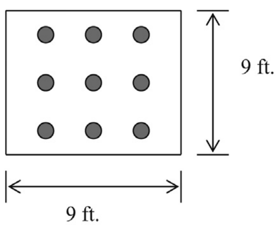


Figure 13.15 Pile group.

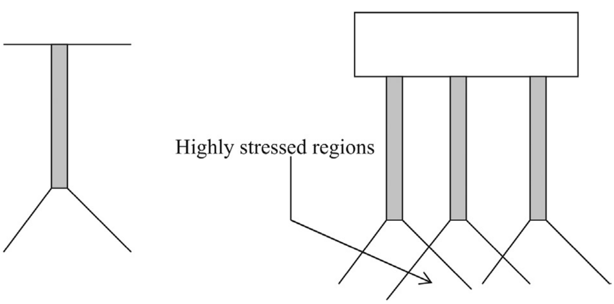


Figure 13.16 Stresses under piles.

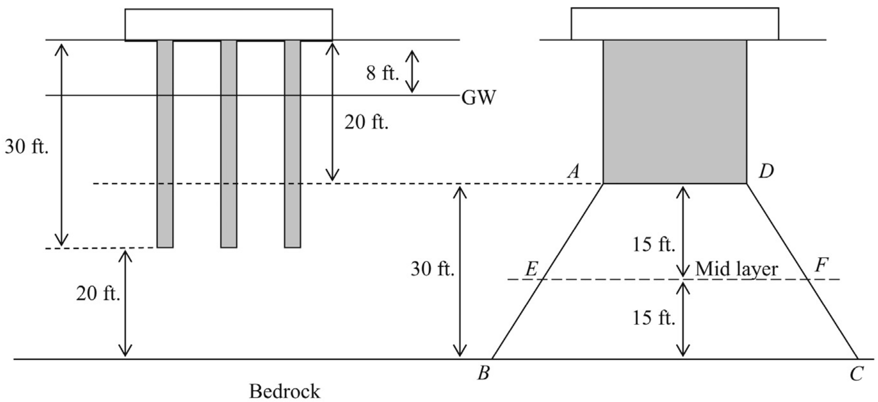


Figure 13.17 Stress distribution.

Here, C_c, compression index; e_0, initial void ratio of the clay layer; H, thickness of the clay layer; p_0', initial effective stress at midpoint of the clay layer; δ_p increase of effective stress due to pile load.

Design example

A pile group is consisted of nine piles each with a length of 30 ft. and a diameter of 1 ft. The pile group is 10 × 10 ft. The pile group is loaded with 160 tons. Density of soil is 120 pcf.

The following clay parameters are given:

C_c = Compression index = 0.30
e_0 = Initial void ratio of the clay layer = 1.10
Depth to bedrock from the bottom of the piles = 20 ft.
Groundwater is at a depth of 8 ft. below the surface

Find the long-term settlement due to consolidation (Fig. 13.17).

Solution

Step 1: simplify the pile group to a solid footing with a depth of 20 ft. (2/3 of the length of piles).

Step 2: find the effective stress at midlayer of the clay stratum. Midlayer occurs 15 ft. below the end of the assumed solid footing.

$$p_0' = (8 \times 120) + 27 \times (120 - 62.4) = 2515.2 \text{ psf}$$

Here, (8 × 120) = effective stress component above the groundwater level and 27 × (120 − 62.4) = effective stress component from groundwater level to the midlayer.

Step 3: find the effective stress increase due to the pile group.

Assume a 2V to 1H stress distribution.
Pile group dimensions = 10 × 10 ft.
Length EF = 10 + 2 × (15/2) = 25 ft. (Fig. 13.17)
Pile group load = 160 tons
Stress at the bottom of assumed solid footing = 160/(10 × 10) = 1.6 tsf

Stress at midlayer = 160/(25 × 25) = 0.256 tsf = 512 psf

δ_p = 512 psf

Step 4: consolidation settlement of the pile group (S) = $C_c/(1 + e_0)\cdot H\cdot \log_{10}\cdot(p_0' + \delta p)/p_0'$

Here, C_c = 0.30, e_0 = 1.10, H, thickness of the compressible portion of the clay layer = 30 ft.

S = 0.30/(1 + 1.10)·30·$\log_{10}$·(2515 + 512)/2515 = 0.345 ft. = 4.13 in.

Note: the consolidation settlement can be reduced by conducting any of the following:

1. Increase the length of piles (when the length of piles are increased, "H" value in the preceding equation would reduce. "H" is the thickness of the compressible portion of the clay layer).

Increase the length and width of the pile group (when the length and width of the pile group is increased, stress increment (δ_p) due to pile group load would reduce).

13.9 Long-term pile group settlement in clay soils Janbu method

The method explained in the previous chapter needs C_c value and e_0 value. Laboratory tests are needed to obtain these parameters. In this chapter a simplified procedure to deduce long-term settlement in pile groups will be presented. Parameters in the Janbu method are easily obtained compared to the compression index method described in the previous chapter.

- Janbu method can be used to compute the long-term elastic settlement in sandy soils and clayey soils.
- Janbu assumes that the soil settlement is related to two nondimensional parameters:
  - Stress exponent: j (nondimensional parameter)
  - Modulus number: m (nondimensional parameter)
- Stress exponent (j) and modulus number (m) are unique to individual soils.

13.9.1 Janbu equation for clay soils

$$\text{Settlement} = L \times \frac{1}{m} \ln(\sigma_1' / \sigma_0')$$

Since stress exponent, j = 0 for pure clay soils, it does not appear in the preceding equation.

Modulus number is obtained from Table 13.2. Note that "j" is not equal to zero for silty clays and clayey silts (Table 13.2).

L, thickness of the clay layer, ln = natural log; m, the modulus number (dimensionless); σ_1', new effective stress after the pile load; and σ_0', effective stress prior to the pile load (original effective stress).

Table 13.2 Janbu soil settlement parameters

Soil type	Modulus number (m)	Stress exponent (j)
Till (very dense to dense)	1000–300	1
Gravel	400–40	0.5
Sand (dense)	400–250	0.5
Sand (medium dense)	250–150	0.5
Sand (loose)	150–100	0.5
Silt (dense)	200–80	0.5
Silt (medium dense)	80–60	0.5
Silt (loose)	60–40	0.5
Silty clay (or clayey silt)		
Stiff silty clay	60–20	0.5
Medium silty clay	20–10	0.5
Soft silty clay	10–5	0.5
Soft marine clay	20–5	0
Soft organic clay	20–5	0
Peat	5–1	0

13.9.2 Settlement calculation methodology

- The pile group is represented with an equivalent footing located at the neutral plane. Neutral plane is considered to be located at 2/3 H from the pile cap (H = pile length).
- The settlement of the pile group is computed using the Janbu equation.

Design example

Estimate the settlement of the pile group shown (H = 30 ft. and D = 10 ft.).
(Density of organic clay = 110 pcf.; column load = 100 tons; pile cap size = 5 × 5 ft) (Fig. 13.18).

Step 1: consider a footing located at neutral plane.
- The stress at the neutral plane = 100/(5 × 5) tsf = 4 tsf
- The thickness of the compressible clay layer = H/3 + D = (30/3 + 10) ft. = 20 ft.

Step 2: apply the Janbu equation

Janbu equation Settlement = $L \times 1/m \ln(\sigma_1'/\sigma_0')$

Here L, thickness of the clay layer = 20 ft. m, in Table 13.2 gives a range of 20–5 for soft organic clay. Use m = 10. σ_0', initial effective stress at midpoint of the compressible clay layer.

The midpoint of the compressible clay layer occurs at plane "C," at a depth of 30 ft. below the ground surface.

$$\begin{aligned}\sigma_0' &= 30 \times 110 = 330\,\text{psf}\\ \sigma_1' &= 330\,\text{psf} + \text{stress increase due to columnload at the midpoint}\end{aligned} \tag{13.1}$$

Column load at neutral plane = 100 tons (width and length of the equivalent footing at the neutral plane is 5 × 5 ft.).

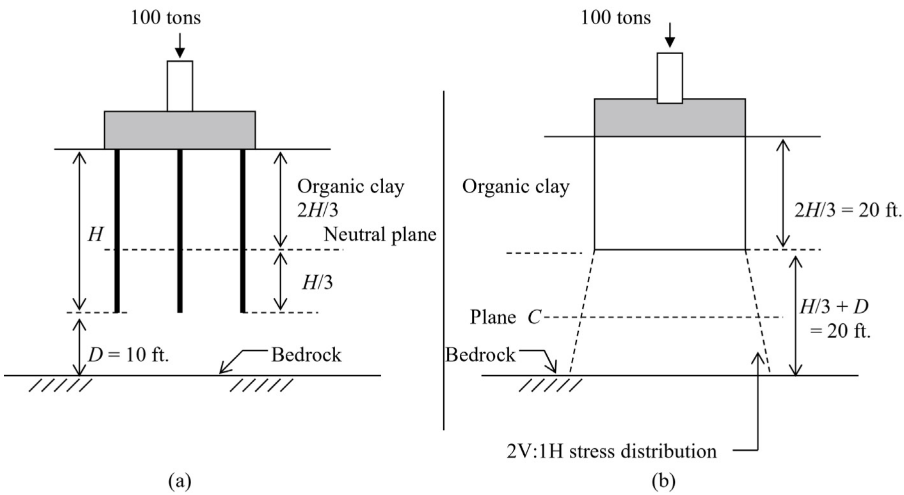


Figure 13.18 Figure for the example.

Length at plane "C" = 5 + 10/2 + 10/2 ft. = 15 ft. (assuming a 2 vertical to 1 horizontal stress distribution).

Hence, area at plane "C" = 15 × 15 ft = 225 ft.^{2}.

Stress increase due to column load at point "C" = 100/225 = 0.444 tsf = 889 psf

σ_1' = 330 psf + stress increase due to column load at the midpoint (from Equation (13.1))

σ_1' = 330 + 889 = 1219 psf

Janbu equation settlement = $L \times 1/m \ln(\sigma_1'/\sigma_0')$

Settlement = 20 × 1/10 ln(1219/330) = 2.61 ft.

This is a very high settlement. Hence, increase the area of the pile cap or embedment depth of piles.

13.10 Pile group settlement in sandy soils: Janbu method

- Janbu proposed the following semiempirical equation for sandy soils. Sandy soils reach the maximum settlement within a short period of time.

$$\text{Settlement} = L \times \frac{2}{m} \times [(\sigma_1'/\sigma_r)^{1/2} - (\sigma_0'/\sigma_r)^{1/2}]\,\text{tsf}$$

Note: the stress exponent (j) for sandy soils is 0.5 and it is already integrated into the equation. The equation is unit sensitive. All parameters should be in tons and feet for English units or meters and kPa for metric units.

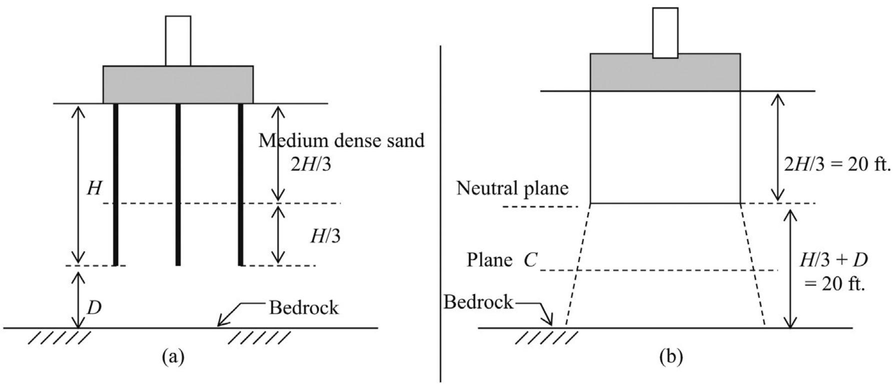


Figure 13.19 Pile group.

L, thickness of the compressible sand layer; *m*,the modulus number (dimensionless); $\sigma_1{}'$, new effective stress after the pile load; $\sigma_0{}'$, effective stress prior to the pile load (original effective stress); and σ_r, reference stress (100 kPa for metric units and or 1 tsf for English units).

13.10.1 Janbu procedure for sandy soils

- Assume the neutral axis to be at a depth 2/3 of the length of the piles.
- The pile group is simplified to a solid footing ending at a depth of 2/3 of the length of the piles.
- Compute the original stress (stress due to overburden prior to the construction of piles)
- Compute the stress increase due to pile group.
- Use the Janbu equation to find the settlement.

Design example

Estimate the settlement of the pile group shown ($H = 30$ ft. and $D = 10$ ft.).

(Column load = 100 tons; pile cap size = 5 × 5 ft., density of medium dense sand = 100 pcf, Fig. 13.19).

Step 1: simplify the pile group to a solid footing ending at 2/3 of the depth of the pile group.

The stress at the neutral plane = 100/(5 × 5) tsf = 4 tsf.

- The thickness of the compressible sand layer = $H/3 + D = 30/3 + 10$ ft. = 20 ft.

Step 2: apply the Janbu equation.

Janbu equation: settlement = $L \times (2/m) \times [(\sigma_1{}'/\sigma_r)^{1/2} - (\sigma_0{}'/\sigma_r)^{1/2}]$ tsf

Here, L = thickness of the sand layer = 20 ft. m = Table 13.2 gives a range of 250–150 for medium dense sand. Use $m = 200$. $\sigma_0{}'$ = initial effective stress at midpoint of the compressible sand layer.

The midpoint of the compressible sand layer occurs at plane "*C*," at a depth of 30 ft. below the ground surface.

$$\begin{aligned}\sigma_0' &= 30\times 100 = 300\text{ psf} = 0.15\text{ tsf (all units should be converted to tsf)}\\ \sigma_1' &= 0.15\text{ tsf } + \text{stress increase due to column load at the midpoint}\end{aligned} \tag{13.2}$$

Column load at neutral plane = 100 tons (width and length of the equivalent footing at the neutral plane is 5 × 5 ft.).

Length of footing at plane "*C*" = 5 + 10/2 + 10/2 ft. = 15 ft.

Hence, area at plane "*C*" = 15 × 15 ft. = 225 ft.^2

Stress increase due to column load at point "*C*" = 100/225 = 0.444 tsf.

σ_1' = 0.15 tsf + stress increase due to column load at the midpoint [from Equation (13.2)]

σ_1' = 0.15 + 0.444 = 0.594 tsf

- When computing the stress increase due to the column load at midpoint, stress distribution of 2V:1H is used.

$$\text{Settlement} = L\times\frac{2}{m}\times\left(\frac{\sigma_0'}{\sigma_r}\right)^{1/2} - \left(\frac{\sigma_0'}{\sigma_r}\right)^{1/2}\ \text{tsf}$$

σ_r, reference stress = 1 tsf.

Settlement = 20 × 2/200 × $[(0.594)^{1/2} - (0.15)^{1/2}]$ tsf = 0.077 ft. = 0.92 in.

13.11 Pile group settlement versus single pile settlement

Researchers in the past did not see any relationship between settlement of single piles and settlement of pile groups. The following comment was made by Karl Terzaghi regarding this issue:

> *Both theoretical considerations and experience leave no doubt that there is no relation whatever between the settlement of an individual pile at a given load and that of a large group of piles having the same load per pile.*
>
> *James Forrest Lecture, Terzaghi (1939)*

Thanks to numerous analytical methods and high-powered computers, researchers have been able to develop relationships between single pile settlement and pile group settlement.

13.11.1 Factors that affect pile group settlement

- L/d ratio (L, pile length; d, pile diameter)
- s/d ratio (s, spacing between piles)
- E_p/E_s ratio (E_p, Young's modulus of pile; E_s, Young's modulus of soil)

Interestingly, the geometry of the group does not have much of an influence on the settlement. A (2 × 8) group and a (4 × 4) group would act almost the same.

13.11.2 Group settlement ratio

Group settlement ratio (R_s) = (settlement of group/settlement of single pile)

R_s can be approximated as follows:

$R_s = n^{0.5}$ (n, number of piles in the group)

Design example

A single pile would have a settlement of 1.5 in. when it is loaded to 100 tons. A (4 × 4) pile group (of similar piles) was loaded to 1600 tons. Find the settlement of the group.

Solution

Settlement of the group = R_s × settlement of a single pile

$R_s = n^{0.5} = 16^{0.5} = 4$

Settlement of the group = 4 × 1.5 in. = 6 in.

Pile group settles more than a single pile when the load per pile is the same as the single pile. This happens due to increase of soil stress within the group.

13.11.3 Consolidation settlement

- For pile groups, settlement due to consolidation is more important than for single piles. Hence, it is recommended to compute the settlement due to consolidation as well as the elastic settlement.

13.12 Pile group design (capacity and settlement): example

Design example

Find the allowable pile capacity and the settlement of the pile group shown. Individual piles are of 1 ft. diameter. Pile group efficiency = 0.75 (Fig. 13.20).

Table 13.3 provides soil parameters for the given soil strata.

Step 1: compute the skin friction of an individual pile.

Skin friction in sands: S (skin friction) = $K \times \sigma \times \tan\delta$ × perimeter of the pile

Skin friction in clays: S (skin friction) = $\alpha \times C$ × perimeter of the pile

- Skin friction in the first layer (medium sand) = $K \times \sigma \times \tan\delta \times [(\pi \times 1) \times 12]$

  σ at midpoint of the pile in the given layer is used.

  $\sigma = 120 \times 2 + (120 - 62.4) \times 4 = 470.4$ psf (groundwater is at 2 ft.)
- Skin friction in the first layer (medium sand) = $1 \times 470.4 \times \tan 25 \times [(\pi \times 1) \times 12] = 7345$ lbs
- Skin friction in the second layer (organic clay) = $\alpha \times C \times [(\pi \times 1) \times 10]$

  $= 1 \times 500 \times [(\pi \times 1) \times 10]$

  $= 15{,}708$ lbs
- Skin friction in the third layer (silty sand) = $K \times \sigma \times \tan\delta \times [(\pi \times 1) \times 15]$

  σ at midpoint of the pile in the given layer is used

  $\sigma = 120 \times 2 + (120 - 62.4) \times 10 + (115 - 62.4) \times 10 + (118 - 62.4) \times 7.5 = 1759$ psf

  Skin friction in third layer (silty sand) = $1.2 \times 1759 \times \tan 30 \times [(\pi \times 1) \times 15] = 50{,}682$ lbs
- Skin friction in fourth layer (soft clay) = $\alpha \times C \times [(\pi \times 1) \times 13]$

  $= 0.65 \times 120 \times [(\pi \times 1) \times 13] = 3185$ lbs

Total skin friction = 7,345 + 15,708 + 50,682 + 3,185 = 76,920 lbs.

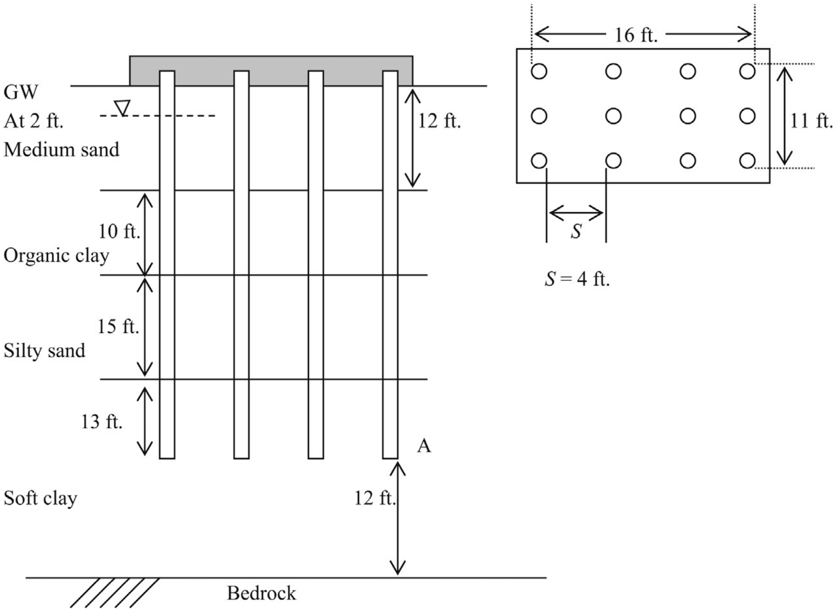


Figure 13.20 Allowable pile capacity.

Table 13.3 Soil parameters

Parameters	Medium sand	Organic clay	Silty sand	Soft clay
Thickness (ft.)	12	10	15	25
Density (pcf)	120	115	118	110
K (Earth pressure coefficient)	1		1.2	
δ (friction angle between pile and soil)	25		30	
ϕ (soil friction angle)	30		35	
α		1		0.65
(C) cohesion (psf)		500 psf		120 psf
N_c				9
M (Janbu parameter)	200	10	80	15
J (Janbu parameter)	0.5	0	0.5	0.5

Step 2: compute the ultimate end bearing capacity (acting as individual piles).

Ultimate end bearing capacity in clay = $N_c \times C \times$ pile tip area ($N_c = 9$).

Ultimate end bearing capacity in soft clay = $9 \times 120 \times \pi \times$ diameter^2/4 = 848.2 lbs/per pile.

Total ultimate bearing capacity per pile = 76,920 + 848.2 = 77,768 lbs.

Total ultimate bearing capacity of the group (assume a group efficiency of 0.8) =
= $12 \times 77{,}768 \times 0.8$ = 746,573 lbs = 373 tons.

Step 3: compute the pile group capacity (failure as a group).

- Skin friction of a pile group in clay = $C \times$ perimeter of the pile group
  Since the failure is between soil and soil, "α" coefficient is essentially "1".
- Skin friction of a pile group in sand = $K \times \sigma \times \tan \phi \times$ Perimeter of the pile group
  Notice in the case of a pile group "ϕ" is used instead of δ, since failure is between soil and soil.
- Skin friction in the first layer (medium sand) = $K \times \sigma \times \tan \phi \times [16 + 16 + 11 + 11] \times 12$
  σ at midpoint of the pile in the given layer is used = $120 \times 2 + (120 - 62.4) \times 4 = 470.4$ psf
- Skin friction in the first layer (medium sand) = $1 \times 470.4 \times \tan 30 \times 54 \times 12 = 155{,}313$ lbs
- Skin friction in the second layer (organic clay) = $C \times [16 + 16 + 11 + 11] \times 10$
  $= 500 \times 54 \times 10 = 270{,}000$ lbs
- Skin friction in the third layer (silty sand) = $K \times \sigma \times \tan \phi \times [16 + 16 + 11 + 11] \times 15$
  σ at midpoint of the pile in the given layer is used.
  $\sigma = 120 \times 2 + (120 - 62.4) \times 10 + (115 - 62.4) \times 10 + (118 - 62.4) \times 7.5 = 1759$ psf
  Skin friction in the third layer (silty sand) = $1.2 \times 1759 \times \tan 30 \times 54 \times 15 = 871{,}160$ lbs.
- Skin friction in the fourth layer (medium stiff clay) = $C \times [16 + 16 + 11 + 11] \times 13$
  $= 120 \times 54 \times 13 = 84{,}240$ lbs.
  Total skin friction = 155,313 + 270,000 + 871,160 + 84,240 = 1,380,713 lbs = 690 tons
- End bearing capacity of the pile group in clay = $N_c \times C \times$ area ($N_c = 9$)
  $= 9 \times 120 \times (16 \times 11) = 190{,}080$ lbs
  = 95 tons.
- Total ultimate capacity = 690 + 95 = 785 tons.
  In this case, pile group capacity acting as a group is greater than the collective capacity of individual piles. Hence, use the lower value of 373 tons with a factor of safety of 2.5.
  Allowable load on the pile group = 150 tons.

Step 4: settlement computation.

- Janbu tangent modulus procedure is used to compute the pile group settlement. Consider an equivalent shallow foundation placed at neutral plane (at a depth of $2H/3$ from the surface, H = length of piles, Fig. 13.21).
- Compute the settlement of the 4-ft. thick silty sand layer.

13.12.1 Janbu equation for sandy soils

$$\text{Settlement} = L \times \frac{2}{m}[(\sigma_1')^{1/2} - (\sigma_0')^{1/2}] \text{ tsf}$$

Here, L, thickness of the compressible sand layer; m, the modulus number (dimensionless); σ_1', new effective stress after the pile load; σ_0', effective stress prior to the pile load (original effective stress); L, thickness of the sand layer = 4 ft.; m, Table 13.2 gives a range of 250–150 for medium dense sand. Use $m = 200$;

σ_0', initial effective stress at midpoint of the compressible sand layer.

The midpoint of the compressible sand layer occurs 2 ft. below the hypothetical footing.

$\sigma_0' = 2 \times 118 = 236$ psf = 0.118 tsf (all units should be converted to tsf).

σ_1' = 0.118 tsf + stress increase due to column load at the midpoint [Equation (13.2)]

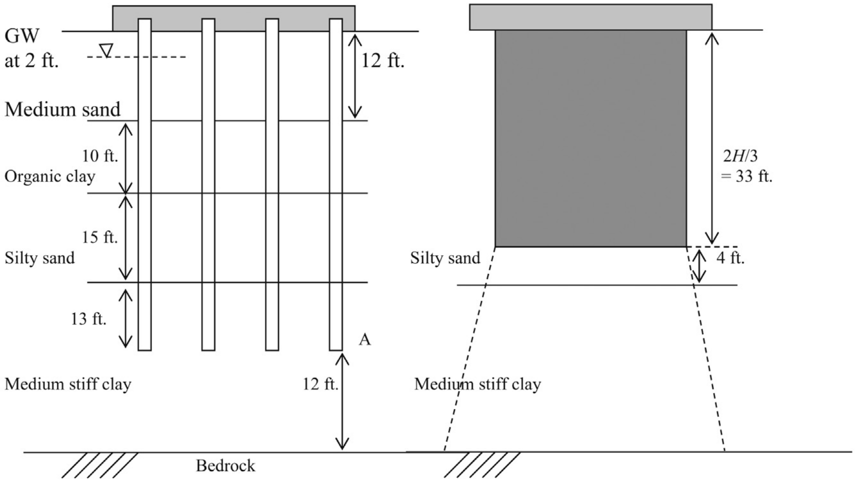


Figure 13.21 Pile group settlement.

Column load at neutral plane = 150 tons (width and length of the equivalent footing at the neutral plane is 11 × 16 ft.).

Length of base at the midpoint of the silty sand layer = 16 + 2/2 + 2/2 ft. = 18 ft.

Width of base at midpoint of the silty sand layer = 11 + 2/2 + 2/2 ft. = 13 ft.

Hence, the area at midplane of the silty sand layer = 18 × 13 ft. = 234 ft.^{2}.

Stress increase due to column load at midpoint = 150/234 = 0.64 tsf.

σ_1' = 0.118 tsf + Stress increase due to column load at midpoint [Equation (13.2)]

σ_1' = 0.118 + 0.64 = 0.758 tsf

- When computing the stress increase due to the column load at midpoint, stress distribution of $2V{:}1H$ is used.

Janbu equation → settlement = $L \times 2/m\,[(\sigma_1')^{1/2} - (\sigma_0')^{1/2}]$ tsf

Settlement = 2 × 2/200 [(0.758)$^{1/2}$ – (0.118)$^{1/2}$] tsf = 0.011 ft. = 0.92 in.

- Settlement in medium stiff clay

13.12.2 Janbu equation for clay soils

$$\text{Settlement} = L \times \frac{1}{m} \ln(\sigma_1'/\sigma_0')$$

Here, L, thickness of the clay layer; ln = natural log; m, the modulus number (dimensionless);

σ_1' = new effective stress after the pile load, and σ_0' = effective stress prior to the pile load (original effective stress)

Consider a footing located at neutral plane.

- Thickness of the compressible clay layer = 25 ft.
- m = Table 13.2 gives a range of 20–10 for medium silty clay. Use m = 15.

σ_0', initial effective stress at midpoint of the compressible clay layer. Midpoint of the medium silty clay layer occurs at a depth of 49.5 ft. from the surface.

$\sigma_0' = 2 \times 120 + 10 \times (120 - 62.4) + 10 \times (115 - 62.4) + 15 \times (118 - 62.4) + 12.5 \times (110 - 62.4)$

$= 2771$ psf $= 1.385$ tsf

$\sigma_1' = \sigma_0'$ + stress increase due to column load at the midpoint.

Area at the midpoint of the medium stiff clay layer = (16 + 16.5/2 + 16.5/2) × (11 + 16.5/2 + 16.5/2)

= 893.75 ft.^{2}

$\sigma_1' = \sigma_0' + 150/893.75 = 1.385 + 0.1678 = 1.552$ tsf

Settlement in medium stiff clay = 25 × 1/15 [ln(1.552/1.385)] = 0.189 ft. = 2.28 in.

Settlement due to medium stiff clay and silty sand = 2.28 + 0.92 = 3.2 in.

This settlement is excessive. Piles need to be driven deeper or the number of piles needs to be increased.

References

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Terzaghi, K., 1939. Soil mechanics – a new chapter in engineering science. J. Inst. Civil Eng. 12, 106–141.

Vesic, A.S., 1977. Design of Pile Foundations, National Cooperative Highway Research Program 42, Transportation Research Board.

Wave equation basics

14

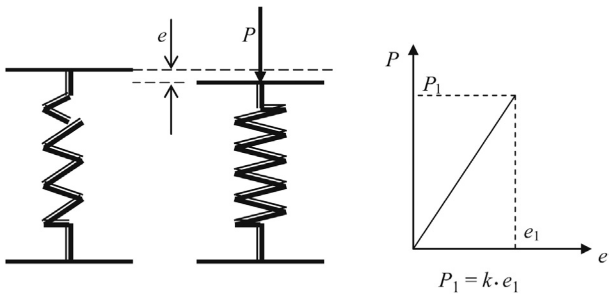
- D.V. Isaacs in 1931, for the first time, proposed the wave equation (Smith, 1960). In 1938, E.N. Fox published a solution to the wave equation. During that time, computer usage was not popular and Fox used many simplifying assumptions. Smith (1960) solved the wave equation without simplifying assumptions using a computer. According to Smith, his numerical solution was within 5% of the analytical solution. Smith's solution was based on finite difference technique.
- Prior to discussing the wave equation, it is important to look at dynamic equations.
- The fundamental driving formula $Qs = Wh$.
- The hammer is dropped on the pile and the set (s) is measured (Fig. 14.1).
- The set is the distance that the pile had moved into the ground.
- "h" is the drop of the hammer.
- Energy imparted to the pile by the hammer = Wh.
  Energy used by the pile = Qs
- Energy imparted to the pile is equal to the energy used by the pile (energy losses are ignored).

Hence, $Wh = Qs$

14.1 Assumptions

1. It is assumed that "Q" is constant force acting at the bottom of the pile. This assumption is not correct. The resistance of the pile is due to two forces. One is the end bearing force and the other is the skin friction. Both these forces do not remain constant during pile movement (the pile moves by "s" inches in downward direction).
2. Dynamic equations do not consider stress distribution in the pile, pile diameter, or type of pile. For instance, both piles shown in Fig. 14.2 would give the same bearing capacity.
   - The two piles shown are located in completely different soil conditions. Dynamic equations will not differentiate between the two cases.
   - Dynamic equations consider the pile to be rigid during driving. In reality, the pile would recoil and rebound during driving.

14.1.1 Springs



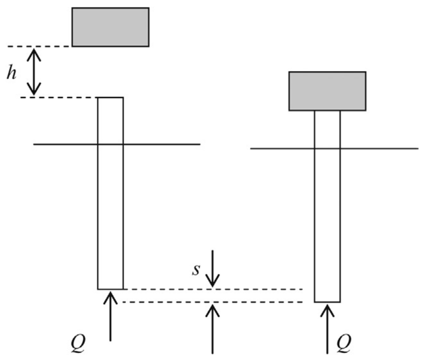


Figure 14.1 Pile and hammer.

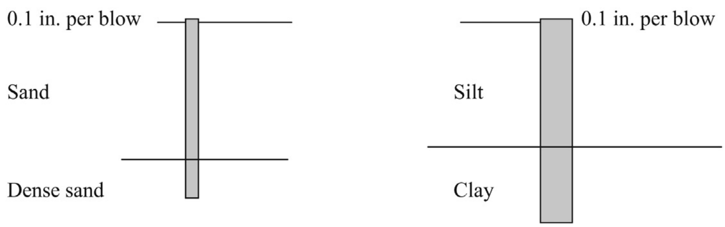


Figure 14.2 Pile and soil.

Force "P" is applied to the spring. The spring settles by a distance of "e."
$P = ke$
k = spring constant.

14.1.2 Dashpots

- In the case of dashpots, the force is proportional to the *velocity* of the dashpot. "C" is known as the dashpot constant (Fig. 14.3).

  Springs (force is proportional to distance): $P = ke$
  Dashpots (force is proportional to velocity): $P = CV$

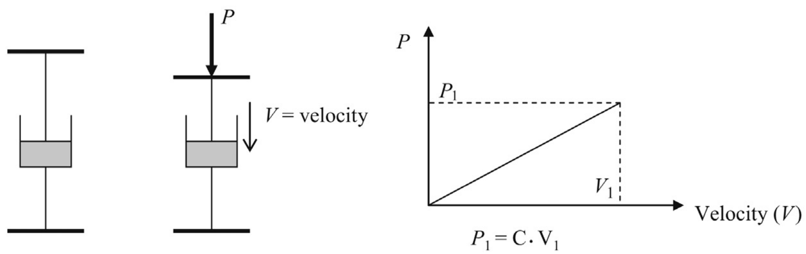


Figure 14.3 Springs and dashpots.

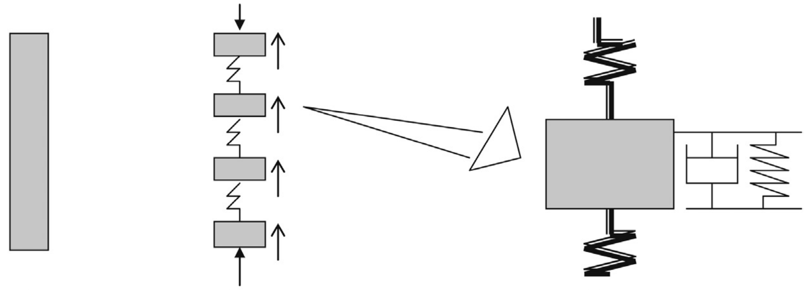


Figure 14.4 Representation of piles in wave equation analysis. (a) A pile is shown. (b) The pile is divided into segments. If the pile is divided into more segments, accuracy of the analysis can be improved. On the other hand it would take more computer time for the analysis. (c) One segment is shown here. The skin friction is represented with a dashpot and a spring.

14.2 Representation of piles in wave equation analysis

- The pile is broken down to small segments.
- Skin friction is represented by springs and dashpots acting on each segment. Refer to Fig. 14.4 for further understanding.

14.3 Wave equation

$$\frac{\partial^2 u}{\partial t^2} = c^2 \frac{\partial^2 u}{\partial x^2}$$

Here, u, displacement; c, wave speed; t, time; and x, length.

- Finite difference method is used to solve the equation. Computer programs, such as WEAP and GRLWEAP, are available in the market to perform the wave equation analysis.

14.3.1 Soil strength under rapid loading

This chapter is designed to provide information regarding the strength properties of soil under rapid loading situations. During pile driving, the pile is subjected to rapid loading.

Experiments have shown the failure stress to be higher under rapid loading conditions than the failure stress during gradual loading (Fig. 14.5).

- During pile driving, the pile is subjected to rapid loading.

R_u, ultimate tip resistance during *static* loading; R_f, ultimate tip resistance during *rapid* loading; S_u, ultimate skin friction during *static* loading; and S_f, ultimate skin friction during *rapid* loading.

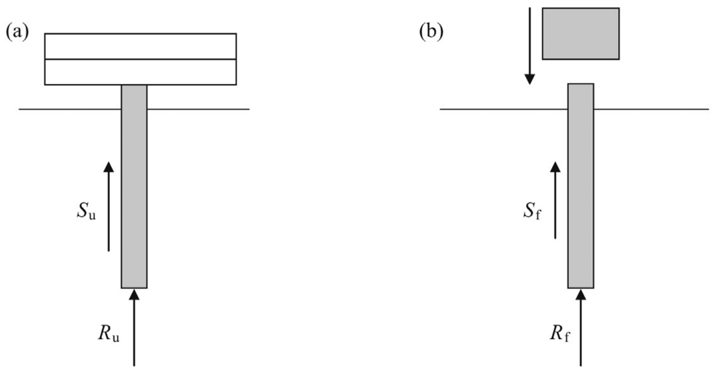


Figure 14.5 Soil strength under rapid loading. (a) Gradual loading (pile load test). (b) Rapid loading (pile driving).

14.4 Equation for tip resistance for rapid loading condition

The following relationship has been developed for tip resistance under rapid loading condition.

$$R_f = R_u\,(1 + J_u \times V^{N_u})$$

Here, S_u, ultimate skin friction during *static* loading; S_f, ultimate skin friction during *rapid* loading; J_u, rate effect parameter for tip resistance; V, velocity of the pile; and N_u, rate effect exponent for tip resistance.

$J_u = 1.2 - 0.007\ C_u$ (for clay soils, Lee et al., 1988)

Here C_u, cohesion measured in kN/m^2.

Units of J_u would be (s/m)^N.

$J_u = 2.2 - 0.08\ (\phi - 20)$ for sandy soils (Lee et al., 1988).

Here, ϕ, friction angle of sand.

Units of R_f and R_u = kN/m^2

Note: the aforementioned equations were developed by the author based on experimental data provided by Lee et al. (1988).

N_u: experiments show that N_u to lie between 0.17 and 0.37 for sandy and clayey soils. Hence, N_u can be taken as 0.2.

14.5 Equations for skin friction for rapid loading condition

The following relationship has been developed for skin friction under rapid loading condition.

$$S_f = S_u\,(1 + J_s \times V^{N_s})$$

Here, S_u, ultimate skin friction during *static* loading; S_f, ultimate skin friction during *rapid* loading; J_s, rate effect parameter for skin friction; V, velocity of the pile; and N_s, rate effect exponent for skin friction.

$J_s = 1.6 - 0.008\ C_u$ for clay soils (Lee et al., 1988).

Here, C_u, cohesion measured in kN/m^2.

Units of J_s would be (s/m)^N.

$J_s = 0$ for sandy soils (Heerema, 1979)

N_s: experiments show N_s to lie between 0.17 and 0.37 for sandy and clayey soils. Hence, N_s can be taken as 0.2 (Heerema, 1979). It should be noted here that N_s (skin friction exponent) and N_u (tip resistance exponent) lie in the same range.

Units of S_f and S_u = kN/m^2.

14.5.1 Data required for wave equation analysis

- The solution of the wave equation will not be discussed here. Geotechnical engineers are required to provide information to wave equation computer programs for analysis.

14.6 Example of input data for wave equation software

14.6.1 Pile hammer data

- Hammer type: Delmag D 12-32 Diesel Hammer (single acting)
- Hammer energy: 31,320 ft. lbs
- Hammer efficiency: 80%
- Blows per minute: 36
- Hammer weight (striking part only): 2,820 lbs
- Hammer stroke: 11′ 1″

(The engineer should obtain the equivalent stroke for double acting hammers from the manufacturer).

14.6.2 Capblock data

- Type: Micarta sheets
- Modulus of elasticity: 70,000 psi
- Coefficient of restitution: 0.6
- Diameter: 12 in.
- Thickness: 6 in.

14.6.3 Pile cushion data

Usually pile cushions are used only for concrete piles.

If a pile cushion is used, material, modulus of elasticity, and dimensions of the cushion should be provided.

14.6.4 Pile properties

- Pile material: steel
- Density: 475 lbs/ft.^{3}

- Outer diameter of pile: 24 in.
- Wall thickness: ¼ in.
- Modulus of elasticity: 30,000 psi
- Pile is driven closed end
- Pile embedment: 30 ft.

Note: for concrete piles, area of steel reinforcements and prestress force also should be provided.

14.6.5 Soil information

Depth 0–1: top soil, SPT (*N*) values: (10, 13)
Depth 1–5: silty sand SPT (*N*) values: (15, 13), (12,18), (13,15), (19,10)
Depth 5–9: soft clay SPT (*N*) values: (2,2), (3,1), (2,5), (4,7)

- Cohesion of clay (C_u) (cohesion value should be provided for clay soils).
- Rapid loading parameters for soil layers (J_u, J_s, N_u, and N_s) (Note: see the previous chapter for an explanation of these parameters).
- Density of soil.
- G_s (shear modulus of soil). G_s can be obtained experimentally. If not, the following approximate equations can be used. See chapter: Shear Modulus.
  $G_s = 150\ C_u$ (for clays); G_s would have the same units as C_u.
  $G_s = 200\ \sigma_v'$ (for sands); σ_v' = vertical effective stress (Lee et al., 1988)

14.6.6 Information provided by wave equation programs

Wave equation programs are capable of providing the following information:

- Blows required to penetrate a certain soil stratum (or rate of penetration per blow).
- Pile capacity.
- Ability of the given pile hammer to complete the project in a timely basis.

References

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Lee, S.L., et al.,1988. Rational wave equation model for pile driving analysis. ASCE Geotech. Eng. J. 114, 306–325.

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Negative skin friction (downdrag) 15

15.1 Introduction

Negative skin friction (also known as downdrag) could be a major problem in some sites. Negative skin friction occurs when there is consolidation of clay due to fill. Figs. 15.1–15.5 show the general construction procedure of piles, pile caps, and the building.

As shown in Fig. 15.1, existing ground profiles are typically uneven. After piles are installed, the fill layer has to be placed so that pile caps can be constructed. This fill layer would create a settlement in the clay layer below. When clay settles, it would drag the piles down. This process is known as downdrag or negative skin friction (Figs. 15.6 and 15.7).

When negative skin friction occurs, a neutral plane is created. Skin friction is negative above the neutral plane. Negative skin friction acts downward. Skin friction is positive below the neutral plane. At neutral plane, skin friction is zero.

15.2 Bitumen-coated pile installation

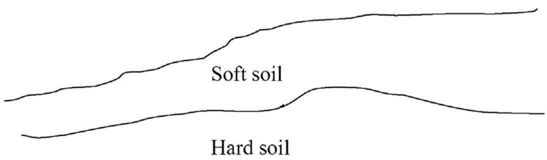
Negative skin friction can be effectively reduced by providing a bitumen coat around the pile. The bitumen coating would drag along with the soil without dragging the pile (Fig. 15.8).

Step 1: a hole is bored.
Step 2: the bitumen coated pipe pile is placed.
Step 3: the pipe pile is concreted.
Step 4: the annular space is grouted.

15.3 How bitumen coating would work against downdrag

- The consolidating clay layer would pull the outer grout layer down.
- The grout layer would pull down the bitumen-coated pile with it.
- The bitumen coating would be pulled down but the downdrag force on the pile is reduced due to the extension of the bitumen coating (Fig. 15.9).

15.3.1 Uneven negative skin friction



Pile Design and Construction Rules of Thumb

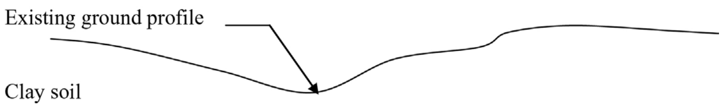


Figure 15.1 Existing ground profile.

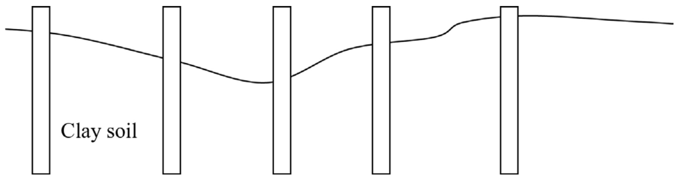


Figure 15.2 Piles installed for the building.

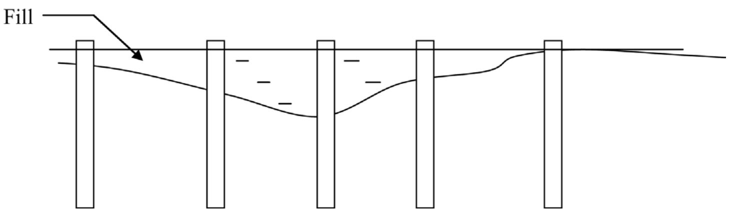


Figure 15.3 Fill layer placed.

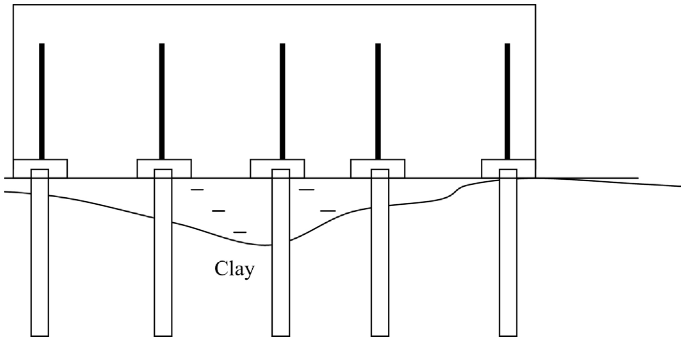


Figure 15.4 Construction of piles caps, columns, and the building.

15.4 Original site soil profile

Due to larger fill load acting on the left half of the building, negative skin friction acting on pile A would be greater than pile B. Hence, pile A, would undergo larger settlement than pile B. Tension cracks could form as shown due to differential settlement (Fig. 15.10).

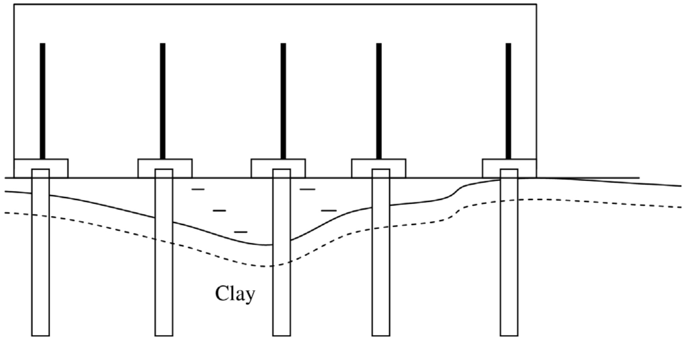


Figure 15.5 Settlement of the clay layer due to weight of the fill.

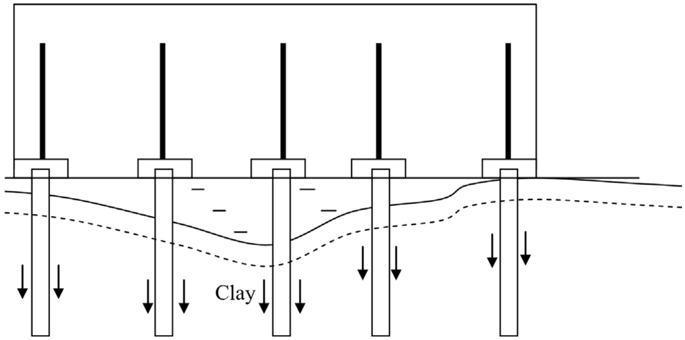


Figure 15.6 Negative skin friction in piles due to settlement of the clay layer.

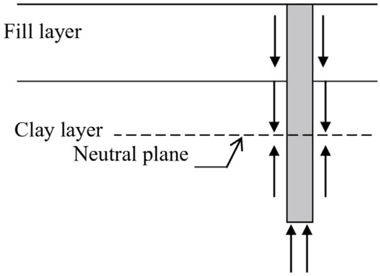


Figure 15.7 Negative skin friction or downdrag.

15.5 Load distribution inside piles

- Prior to analyzing the load distribution inside a pile it is important to look at the load distribution inside a building column.
- Strain gauges can be attached to points "*A*" and "*B*" and stress inside the column could be deduced. Multiplication of the stress by the cross-sectional area would provide the internal load in the column (Figs. 15.11 and 15.12).

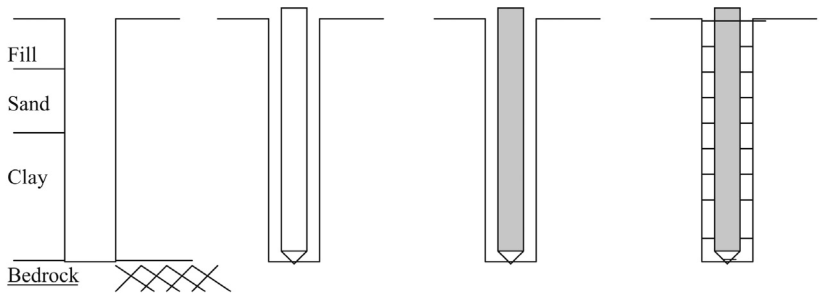


Figure 15.8 Bitumen-coated pile installation.

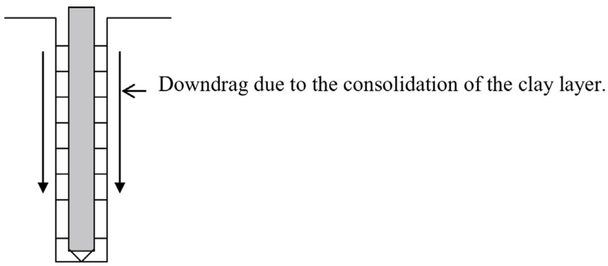


Figure 15.9 How bitumen coating would work.

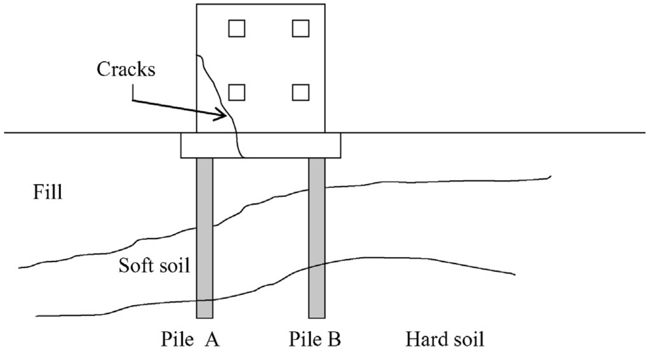


Figure 15.10 Uneven negative skin friction. Original site soil profile.

- Consider point "*A*" immediately below load "*P*" At this location, the column will be stressed to compensate the load from above. Hence, the load inside the column will be equal to "*P*"
- This load will be transferred all the way down to the bottom. Load at point "*B*" will be equal to "*P*" as well. Upward reaction at the footing level also will be equal to "*P*."

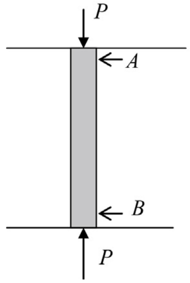


Figure 15.11 Load distributions inside building column.

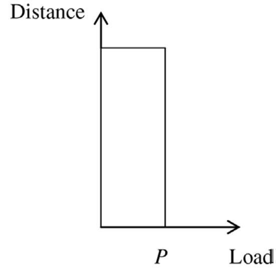


Figure 15.12 Internal load in column.

15.5.1 Load distribution inside a pile prior to loading (sandy soils)

First we need to look at the load distribution inside a pile immediately after driving the pile, prior to applying the load.

- Fig. 15.13 shows a pile immediately after driving, prior to applying the load. Interestingly, there is an end bearing load (Q_1) even before any load has been applied. This is due to the weight of the pile and downward skin friction. In this example, the weight of the pile is ignored for simplicity (Figs. 15.14 and 15.15).

15.5.2 Elasticity in soil

- When a pile is driven, the soil around the pile is stressed. Due to elastic forces, surrounding soil would try to push the pile upward (Figs. 15.16 and 15.17).
- After pile driving is completed, bottom soil layers would try to push the pile out of the ground due to elastic forces. This action of bottom soil layers is countered by the top soil layers.
- Think of a weight placed on a rubber mattress as shown in Fig. 15.17. The rubber mattress is exerting an upward force on the weight due to elasticity. Similarly, the bottom soil layers are exerting an upward force on the pile.
- Elastic forces acting upward are resisted by the skin friction of the top layers. If elastic forces are too large to be countered by skin friction, then the pile would pop out of the ground. Usually this does not happen.

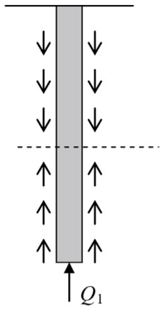


Figure 15.13 Load distribution inside a pile immediately after driving.

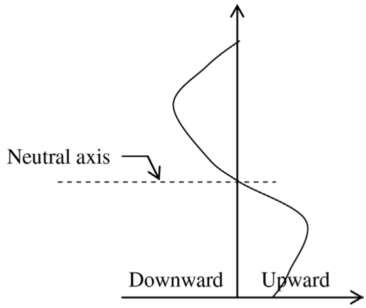


Figure 15.14 Unit skin friction.

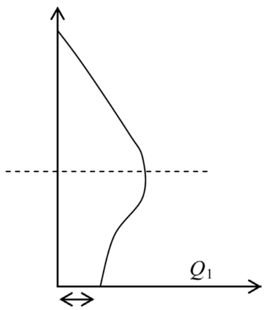


Figure 15.15 Load inside the pile.

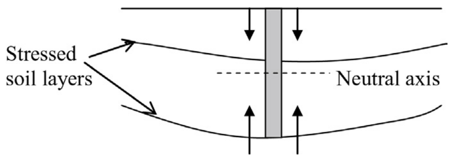


Figure 15.16 Elasticity in soil.

15.5.3 Neutral axis

- Neutral axis is defined as the layer of soil that stays neutral or that does not exert a force on the pile. Reversal of force direction occurs at the neutral axis.

15.5.4 Residual stresses

- Due to the elasticity of soil, the pile would be stressed just after it has been driven prior to loading. These stresses existing in the pile just after driving, prior to loading, is known as *residual stresses*. In the past, researchers ignored residual stresses inside piles prior to loading. Today it is well accepted that residual stresses could be significant in some soils.
- The difference between the upward and downward load will be transferred to the soil below the pile in the form of end bearing load (in this case it is taken to be "Q_1").
- Fig. 15.15 shows the loads generated inside the pile. Internal pile load at the very top of the pile is zero, since there is no external load sitting on the pile.
- When one moves downward along the pile, the pile starts to get stressed due to downward skin frictional load. Hence, the internal pile load increases as shown.
- Internal pile load keeps increasing until the neutral axis. At the neutral axis, internal pile load would reach its maximum. Below the neutral axis, the direction of the skin friction changes. Hence, the internal pile load would start to decrease.

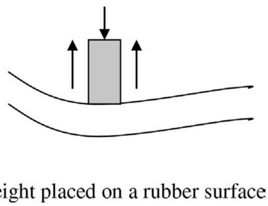


Figure 15.17 Weight placed on a rubber surface.

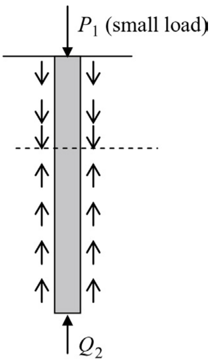


Figure 15.18 Load distribution inside a pile (small load).

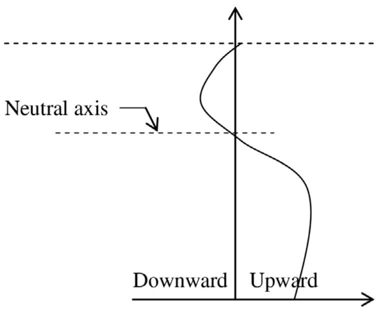


Figure 15.19 Unit skin friction.

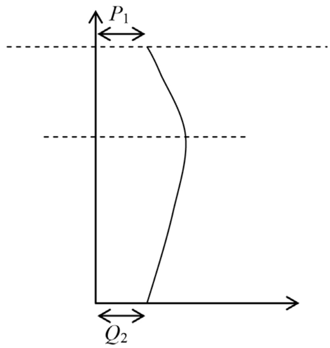


Figure 15.20 Load inside the pile.

- In sandy soils there would be an end bearing load at the bottom of the pile.
- Relationship between applied load (in this case zero) and the end bearing load is End bearing load = Applied load + Downward skin friction + Weight of the pile
  $Q_1 = 0$ + Downward skin friction − Upward force due to elasticity of soil (note: weight of the pile is ignored).

15.5.5 Load distribution inside a pile (small load applied) (sandy soils)

Assume a small load has been applied to the pile. Load distribution diagrams are as shown in Figs. 15.18–15.20.

- When a small load is applied to the pile, the neutral axis would move up. This occurs since the pile has a higher tendency to move downward due to the applied load (P_1). End bearing load (Q_2) will be higher than the previous case (Q_1) in Fig. 15.15 ($Q_2 > Q_1$).
- End bearing load = Applied load + Downward skin friction + Weight of the pile − Upward skin friction.
  $Q_2 = P_1$ + Downward skin friction − Upward forces due to elasticity (note: weight of the pile is ignored).
- When a load is applied to the pile, the neutral axis moves up. Hence, downward skin friction reduces.
- When the applied load is increased, the neutral axis would move upward and eventually disappear. At that point there will be no downward skin friction. The total length of the pile will try to resist the applied load.

15.5.6 Load distribution inside a pile (large load applied) (sandy soils)

- When the applied load is increased, the neutral axis would move up. This occurs since the pile has a higher tendency to move downward due to the applied load (P_2). End bearing load (Q_3) will be higher in this case than the previous end bearing load (Q_2) (Figs. 15.21–15.23).
- End bearing load = Applied load + Weight of the pile − Upward skin friction.
  $Q_3 = P_2$ + Weight of the pile − Upward skin friction (note: no downward skin friction).

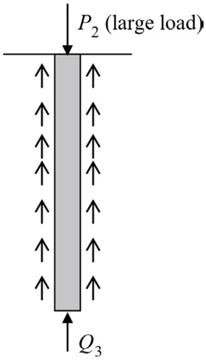


Figure 15.21 Load distribution inside a pile (large load).

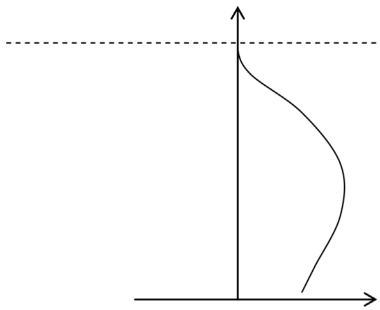


Figure 15.22 Unit skin friction.

15.5.7 Residual stresses in clay soils

Residual stress in sandy soils was discussed earlier. Elastic forces similar to sandy soils would act on a pile driven in clay soils. In addition to elastic forces, hydrostatic forces also would be a factor in clay soils. When a pile is driven, the soil underneath the pile gets pushed down and to the side. This soil movement would create a stress in the surrounding soil (Fig. 15.24).

- Pore pressure in the surrounding soil increases due to induced stress.
- In sandy soils pore pressure would dissipate within minutes and in clay soils it may take months or years.
- When pore pressure dissipates, soil tends to settle. Settling soil around the pile would create a downward force on the pile. This downward force is known as the "Residual Compression." It should be noted here that elastic forces would act in an upward direction on the pile just after driving while hydrostatic forces act downward.

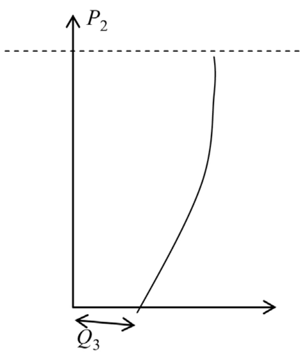


Figure 15.23 Load inside the pile.

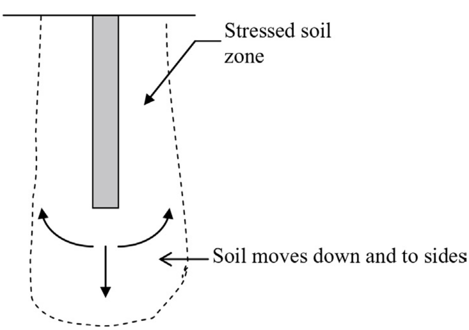


Figure 15.24 Residual stress in clayey soils.

15.5.8 Computation of the loading inside a pile

- Unit skin friction is proportional to the effective stress.
- Hence, unit skin friction at a depth of "Z" can be represented by kZ (k is a constant).
- Total skin friction load at depth Z would be $kZ^2/2$.
- Fig. 15.25a shows the unit skin friction at depth "Z" (or the skin friction per sq. feet at depth "Z"). Fig. 15.25b shows the total skin friction load at depth "Z." Total skin friction load is equal to the area inside the triangle.
  Total skin friction at depth "Z" = $kZ^2/2$
- Fig. 15.25c shows the loading generated inside the pile. "Q" is the total load applied from the top. At depth "Z," soil skin friction has absorbed a load of $kZ^2/2$. Hence, only a load of $[Q - kZ^2/2]$ would be transferred below depth "Z."
  Loading inside the pile at depth "Z" = $Q - kZ^2/2$
- The bottom of the pile shows an end bearing of "P" transferred to the soil underneath from the pile.
- NYC Building Code recommends 75% of the load to be taken by end bearing when the pile tip is located in any type of rock (including soft rock).

15.6 Neutral plane concept

15.6.1 Theory

- During the pile driving process, the soil around the pile would be stressed.
- Due to the high stress generated in surrounding soil during pile driving, water is dissipated from the soil as shown in Fig. 15.26a.
- After pile driving is completed, water would start to come back to the previously stressed soil region around the pile (Fig. 15.26b).
- Due to migration of water, soil around the pile would consolidate and settle.
- It should be mentioned here that settlement of soil surrounding the pile is very small, yet large enough to exert a downward force on the pile. This downward force is known as "Residual Compression."

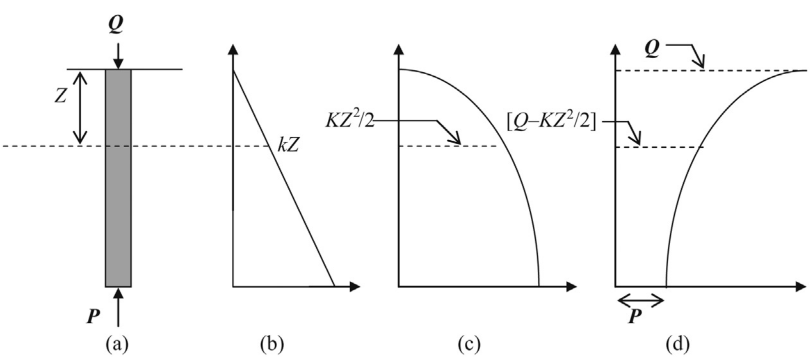


Figure 15.25 Computation of loading inside a pile. (a) Unit skin friction; (b) total skin friction; and (c) load inside the pile.

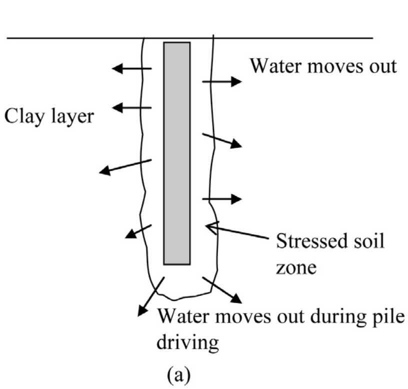


Figure 15.26 Neutral plane concept. (a) During pile driving; and (b) after driving is complete.

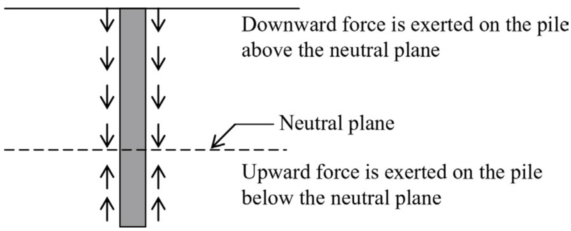


Figure 15.27 Forces at equillibrium.

- Due to the downward force exerted on the pile, the pile would start to move downward.
- Eventually the pile would come to an equilibrium and stop.
- At equilibrium, *upper* layers of soil would exert a downward force on the pile while *lower* layers of soil would exert an upward force on the pile (Fig. 15.27).
- The direction of the force reverses at the neutral plane.

15.6.2 Soil and pile movement (above the neutral plane)

- The soil and the pile move downward above the neutral plane. But the soil would move slightly more in downward direction than the pile. Hence, relative to the pile, soil moves downward.
- This downward movement of soil relative to the pile would exert a downward drag on the pile.

15.6.3 Soil and pile movement (below the neutral plane)

- The soil and the pile move downward below the neutral plane as in the previous case.
- But this time, the pile would move slightly more in downward direction relative to the soil. Hence, relative to the pile, soil moves upward.
- This upward movement of soil relative to the pile would exert an upward force on the pile.

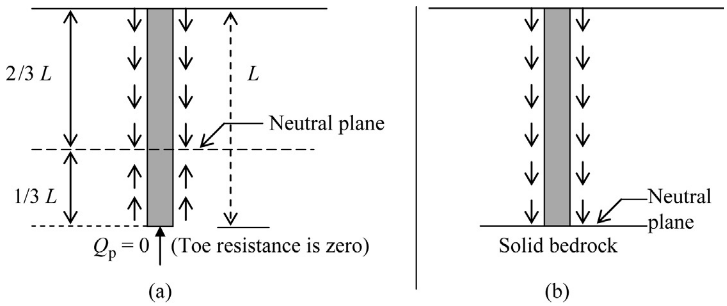


Figure 15.28 Location of neutral plane in floating pile. (a) floating pile (zero end bearing); (b) full end bearing pile.

15.6.4 Soil and pile movement (at neutral plane)

- The soil and the pile move downward at the neutral plane as in previous cases.
- But at the neutral plane the soil and the pile move downward by the same margin.
- Hence, relative movement between soil and pile is zero.
- At neutral plane, no force is exerted on the pile.

15.6.5 Location of the neutral plane

- Exact location of the neutral plane cannot be estimated without elaborate techniques involving complicated mathematics.
- For floating piles, the neutral plane is taken at 2/3 of the pile length (Fig. 15.28).
- In the case of full end bearing piles (piles located on solid bedrock) the neutral plane lies at the bedrock surface. For an upward force to be generated on the pile, the pile has to move downward relative to the surrounding soil.
- Due to the solid bedrock, the pile is incapable of moving downward. Hence, the neutral plane would lie at the bedrock surface.

Bitumen-coated pile design

16

Bitumen-coated piles are used to reduce negative skin friction.

16.1 Causes for negative skin friction

- embankment loads
- groundwater drawdown

16.1.1 Embankment loads

- Negative skin friction occurs above the neutral plane (See chapter: Negative Skin Friction) (Fig. 16.1).
- Negative skin friction occurs due to consolidation of the clay layer. When the clay layer settles, it will drag the pile down.

16.1.2 Negative skin friction due to groundwater drawdown

Effective stress at point "A" (case 1: prior to drawdown) = $\gamma \cdot x_1 + (\gamma - 62.4) \cdot y_1$

Effective stress at point "A" (case 2: after drawdown) = $\gamma \cdot x_2 + (\gamma - 62.4) \cdot y_2$

Here, γ = density of soil (Fig. 16.2).

It can be seen that effective stress at point "A" is larger in case 2 since x_2 is greater than x_1. Higher effective stress would cause the clay to consolidate.

16.2 Bitumen coating

- A bitumen coating should be applied above the neutral plane. In some cases, it may not be necessary to apply bitumen on the full length of the pile above the neutral plane.
- Bitumen can reduce the skin friction by 50–90%.

16.3 Bitumen behavior

It is important to understand the behavior of bitumen prior to designing bitumen-coated piles. Bitumen does not behave as a solid or fluid. It has its own behavior.

Pile Design and Construction Rules of Thumb

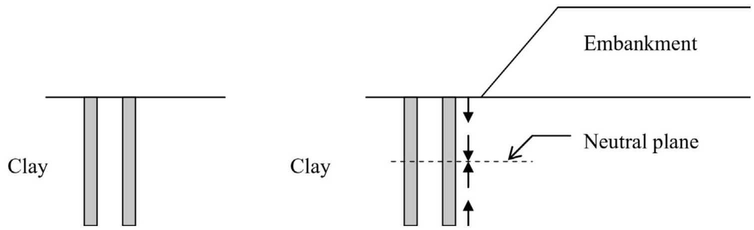


Figure 16.1 Negative skin friction.

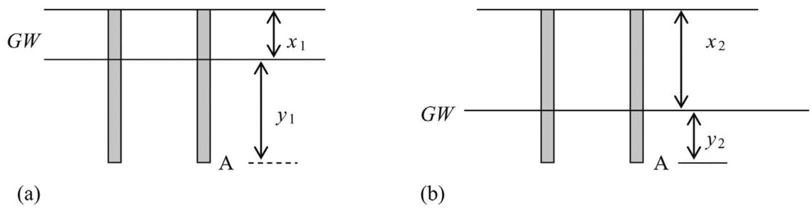


Figure 16.2 Negative skin friction because of groundwater drawdown. (a) Case 1; (b) case 2.

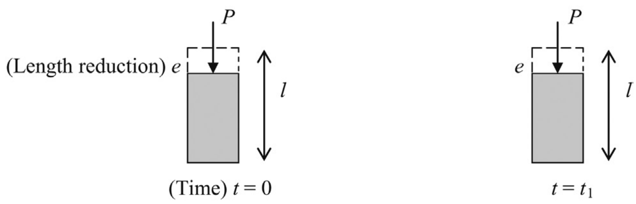


Figure 16.3 Shear strain rate.

16.3.1 Shear strain rate

Geotechnical engineers are very familiar with shear stress and shear strain. On the other hand, "shear strain rate" is rarely encountered in geotechnical engineering.

Consider a steel cylinder (Fig. 16.3).

- When a steel cylinder is subjected to a load of "*P*," it will undergo a length reduction of "*e*." After a time period of t_1, length reduction "*e*" will remain the same (creep behavior of steel is ignored). Stress and strain in steel will not change with time. Hence, shear strain rate is zero for solids for all practical purposes.

16.3.2 Bitumen shear strain rate

The time $t = 0$, $\tau = \tau_1$, $\varepsilon = \varepsilon_1$; $t = t$, $\tau = \tau_2$, $\varepsilon = \varepsilon_2$

Here, τ = shear stress and ε = shear strain (Fig. 16.4).

Figure 16.4 Bitumen shear strain rate.

- Assuming that at time $t = 0$, shear stress is τ_1 and shear strain is ε_1. At time $t = t$, the bitumen would deform and the area would change. Hence, shear stress and shear strain would change.
- At $t = t$, shear stress is τ_2 and shear strain is ε_2 (Fig. 16.4).

$$\text{Hence shear strain rate, } \gamma = \frac{\varepsilon_2 - \varepsilon_1}{t} \tag{16.1}$$

16.3.3 Does shear strain rate vary with temperature?

Bitumen will deform faster at high temperatures. Hence, shear strain rate depends on the temperature.

16.3.4 Shear strain rate is dependent on shear stress

High shear stress will produce a high shear strain rate. Hence, shear strain rate is dependent on shear stress as well (Fig. 16.5).

- When temperature increases from T_1 to T_2, shear strain rate increases for a given shear stress.
- Similarly, the application of a higher shear stress will increase the shear strain rate.

16.3.4.1 Viscosity

Viscosity is defined as the ratio of shear stress to shear strain at a given temperature.

- Fluids with high viscosity will flow slower than fluids with low viscosity. High viscous bitumen will flow slower than water. Bitumen has a higher viscosity than water (Fig. 16.6).

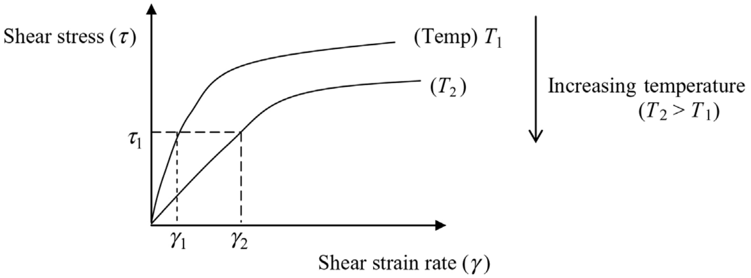


Figure 16.5 Shear strain rate is dependent on shear stress.

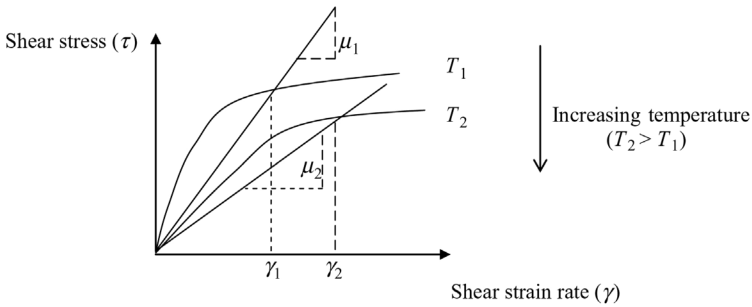


Figure 16.6 Viscosity.

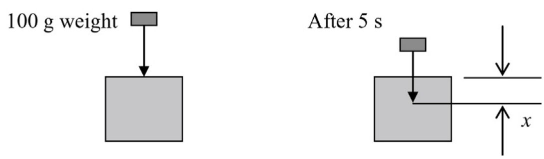


Figure 16.7 PEN number.

$$\text{Viscosity at temperature } (T) = \frac{\text{Shear stress}}{\text{Shear strain rate at temperature } (T)}$$

μ_1, viscosity of bitumen at temperature T_1 and shear strain rate γ_1.
μ_2, viscosity of bitumen of temperature T_2 and shear strain rate γ_2.

16.3.5 Penetration number

Penetration number or the PEN number is defined as the penetration of a standard needle into bitumen under a load of 100 g at 25°C in 5 s (Fig. 16.7).

- A 100-g standard needle is placed on bitumen for 5 s. After 5 s, the penetration is measured in tenths of millimeters.
- If the penetration is 1 mm, then the PEN number would be 10.

16.4 Designing bitumen-coated piles for negative skin friction

Step 1: find the mean ground temperature.

Bitumen viscosity and shear strain rate is dependent upon the ground temperature.
Mean ground temperature is approximately the same as the mean air temperature.
Mean air temperatures (in Fahrenheit) for major US cities and states are as follows: NYC 55°F, Upstate NY 40°F, Florida 70°F, LA 60°F, Arizona 70°F, Seattle 50°F, PA 55°F,

CO 50°F, US states near the Canadian border 40°F. States near the Mexican border 70°F (Visher, 1954).

Step 2: find the required bitumen shear stress (τ).

Typically bitumen shear stress is selected as 10% of the unit negative skin friction. In this case 90% reduction of the negative skin friction occurs. If this is too costly, then bitumen shear stress could be selected as 20% of the unit negative skin friction to reduce the negative skin friction by only 80%. In order to obtain the required bitumen shear stress, it is necessary to find the unit negative skin friction. (See chapters: Pile Design in Clay Soils and Pile Design in Sandy Soils to compute the unit negative skin friction).

16.4.1 Simple formulas to find unit negative skin friction

For clays, unit negative skin friction = $\alpha \cdot C_u$

Here, C_u, undrained shear strength.

For sands, unit negative skin friction = $K \cdot \sigma_v' \cdot \tan\delta$

16.4.2 Example

If the average unit negative skin friction along the shaft was found to be 100 kN/m^2, then shear stress, τ in bitumen is selected to be 10 kN/m^2.

Step 3: bitumen thickness, d.

Typical bitumen thickness varies from 10 mm to 20 mm.

Step 4: settlement rate (SR) (Fig. 16.8).

- After the embankment is constructed, the largest settlement rate occurs at the start. As time goes on settlement rate tapers down.

  Find the settlement rate using the consolidation theory during the first month.

  (1 month = 2.6×10^6 s)

$$\text{Settlement rate (first month) (m/s)} = \frac{\text{Settlement}}{2.6 \times 10^6 \text{ s}}$$

Step 5: bitumen viscosity required to control the downdrag.

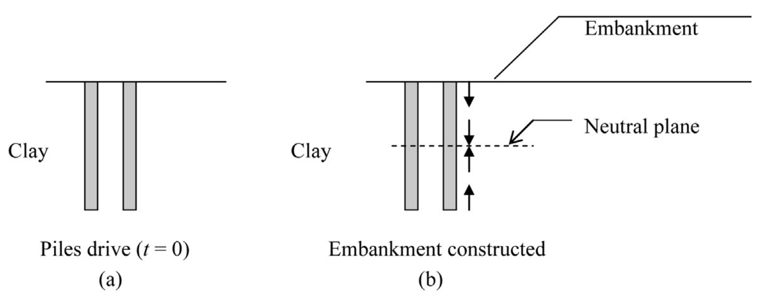


Figure 16.8 Bitumen coated pile design for negative skin friction. (a) Piles driven ($t = 0$); and (b) embankment constructed.

Bitumen viscosity required for downdrag (μ_d) is given by the following equation (Briaud, 1997):

$$\mu_d = \frac{\tau d}{SR} \tag{16.2}$$

Here τ, shear stress in bitumen (kN/m^2); d, thickness of the bitumen layer (m); and SR, settlement rate (m/s) (Briaud, 1997).

Step 6: find a bitumen with viscosity less than μ_d.

Inquire from bitumen manufacturers for a suitable bitumen, that would have a viscosity less than the computed μ_d. Provide the operating temperature (mean ground temperature) to the bitumen manufacturer since viscosity changes with temperature.

Why look for a bitumen that has a viscosity less than μ_d?

If the soil settles faster than calculated, then settlement rate increases. Hence, the right-hand side of Equation (16.2) would decrease. Due to this reason, bitumen with viscosity less than μ_d should be selected. When viscosity goes down fluidity increases.

Design example

A soil is computed to settle by 0.02 m during the first month of loading. A bitumen thickness of 15 mm was assumed. Unit negative skin friction between soil and pile was computed to be 150 kN/m^2. Ground temperature was found to be 50°F. Find the viscosity required to reduce the negative skin friction by 80%.

Solution

- Negative skin friction = 150 kN/m^2
- Bitumen shear stress (τ) = 150 $\times$ 20% = 30 kN/m^2 (to obtain 80% reduction)
- Settlement rate during first month = (0.02/2.6 $\times$ 10^6) m/s = 7.6 $\times$ 10^{-9} m/s
- Bitumen thickness = 15 mm = 0.015 m

Using Equation (16.2),

$$\mu_d = \frac{30 \times 0.015}{7.6 \times 10^{-9}\ \text{kPa s}} = 6 \times 10^7\ \text{kPa s}$$

Specification to the bitumen manufacturer should include the following:

- bitumen should have a viscosity less than 6 $\times$ 10^7 kPa·s at 50°F and at a shear stress level of 30 kN/m^2.

16.5 Bitumen behavior during storage

If bitumen is stored in a high-temperature environment, it would melt and peel off from the pile.

Other than the temperature, time period of storage is also an important factor. If bitumen is stored for a prolonged period of time, it would slowly deform and peel off from the pile.

Step 1: Estimate the time period piles need to be stored.
Step 2: Estimate the mean storage temperature. If piles are to be stored outside, mean air temperature during the storage time period should be estimated.
Step 3: Find the shear strain rate induced in bitumen coating due to gravity.

$$\gamma = \frac{1}{t} \tag{16.3}$$

Here, γ, shear strain rate induced due to gravity (s^{-1}); and t, storage time period (seconds). (Briaud, 1997).

Step 4: compute the viscosity (μ_s) needed to assure proper storage using the following equation:

$$\mu_s = \rho\, t\, d \tag{16.4}$$

Here ρ, density of bitumen measured in kN/m^3; d, bitumen thickness (m); and t, storage time period (in seconds) (Briaud, 1997).

Design example

Assume unit weight of bitumen to be 1100 kg/m^3, bitumen thickness to be 0.015 m, storage time to be 20 days, and storage temperature to be 50°F. Find the viscosity required for proper storage purposes.

Solution

$\mu_s = (\rho t d)$

Density = 1,100 kg/m^3 = 11,000 N/m^3 = 11 kN/m^3

$t = 20 \times 24 \times 60 \times 60 = 1.73 \times 10^6$ s; d (thickness) = 0.015 m

$\mu_s = \rho t d = 11 \times (1.73 \times 10^6) \times 0.015 = 2.85 \times 10^5$ kPa·s

Induced shear strain rate (Equation (16.3)) = $1/(1.73 \times 10^6) = 5.7 \times 10^{-7}$ s^{-1}

Specification to the bitumen manufacturer: provide a bitumen with a viscosity higher than 2.85×10^5 kPa·s at a temperature of 50°F at a shear strain rate of 5.7×10^{-7} s^{-1}.

Note: bitumen with a viscosity higher than the computed value should be used for storage purposes. When the viscosity goes up, fluidity goes down. Hence, high viscous bitumen would be better during storage.

16.6 Bitumen behavior during driving

It is necessary to make sure that the bitumen would not be damaged during driving. In some cases it may not be possible to save the bitumen coating during driving. Predrilling a hole is a common solution to avoid damage.

Step 1: find the shear strain rate during driving. Equation (16.3) could be used to find the shear strain rate.

It is found that after each blow, bitumen recovers. "t" is the time period where the pile hammer is in contact with the pile during pile driving. Usually this is a fraction of a second.

Step 2: find the viscosity needed to maintain bitumen integrity during driving (μ_{dr}).

Low viscous bitumen would be damaged when driving in a high-strength soil (high viscous bitumen is less fluid). Due to this reason, high viscous bitumen should be selected when driving in a high-strength soil.

$$\mu_{dr} = (\tau_{soil})\cdot t \tag{16.5}$$

Here (τ_{soil}) = shear strength of soil (kPa) (Briaud, 1997).

For clay soils, (τ_{soil}) = C_u (usually taken as the undrained shear strength). For sandy soils, (τ_{soil}) = σ' tanϕ' (σ' = effective stress, ϕ' = friction angle). t = time period per one blow (seconds).

Shear strength of sand varies with depth. Hence, the average value along the shaft needs to be taken.

The viscosity of bitumen should be greater than the computed "μ_{dr}."

16.6.1 Temperature

It is assumed that bitumen would be under the same temperature as storage temperature during driving. It is assumed that there is not enough time for the bitumen to reach the ground temperature during driving.

Design example

Assume time period per blow is 0.015 s and shear strength of soil to be 150 kPa. Time period per blow is basically the time period where the pile hammer would be in contact with the pile. The storage temperature of the pile was 50°F. Find the viscosity requirement for pile driving.

Solution

Step 1: find the shear strain rate during driving.

Using Equation (16.3),

γ = 1/0.015 = 66.7 s^{-1}

Step 2: find the viscosity needed to maintain bitumen integrity during driving (μ_{dr}).

$$\mu_{dr} = (\tau_{soil})\, t$$

μ_{dr} = (150) × 0.015 = 2.25 kPa·s (viscosity should be greater than 2.25 kPa·s).

Note: when the viscosity of bitumen goes up fluidity goes down.

Specification to the bitumen manufacturer: provide a bitumen with a viscosity greater than 2.25 kPa·s at a temperature of 50°F and at a shear strain rate of 66.7 s^{-1}.

16.6.2 Final bitumen selection

Selected bitumen should comply with all the conditions (conditions for downdrag, storage, and driving).

- Bitumen viscosity requirement for negative skin friction control: provide a bitumen with a viscosity less than 6×10^7 kPa·s at 50°F at a shear stress level of 30 kN/m^2.
- Bitumen viscosity requirement for storage: provide a bitumen with a viscosity greater than 2.85×10^5 kPa·s at a temperature of 50°F at a shear strain rate of 5.7×10^{-7} s^{-1}.
- Bitumen viscosity requirement for driving: provide a bitumen with a viscosity greater than 2.25 kPa·s at a temperature of 50°F at a shear strain rate of 66.7 s^{-1}.

This could be done by preparing a bitumen selection chart (Fig. 16.9).

16.7 Case study: bitumen-coated piles

16.7.1 24″ pipe piles

Concrete-filled pipe piles were used to support the abutment. These piles were extended to the bedrock. Initial calculations were done to investigate the possibility of ending the piles in the sand layer. The settlements were found to be too large if they were to be ended in the sand layer. The capacity of the piles was estimated to be 150 tons per pile (Fig. 16.10).

16.7.2 Why pipe piles

The clay layer would undergo settlement due to the fill above. When the clay layer settles, it would carry the piles down with it creating negative skin friction (downdrag) on piles. The negative skin friction forces could be as high as 100 tons per pile. The capacity of the piles is not more than 150 tons per pile. The effective capacity (the capacity that is useful) of the pile will be 50 tons per pile. This is not economical. A bitumen coating (1/8″ thick) was used to reduce the downdrag. Bitumen-coated piles were placed on bored holes. Holes were bored and the piles were placed inside the hole.

Note: H-piles have lesser perimeter area compared to similar pipe piles. Hence, H-piles would have less downdrag. On the other hand, H-piles are much more expensive

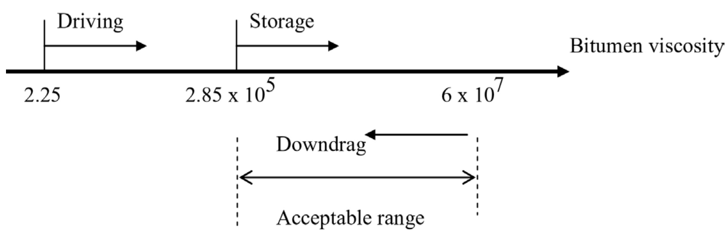


Figure 16.9 Final bitumen selection.

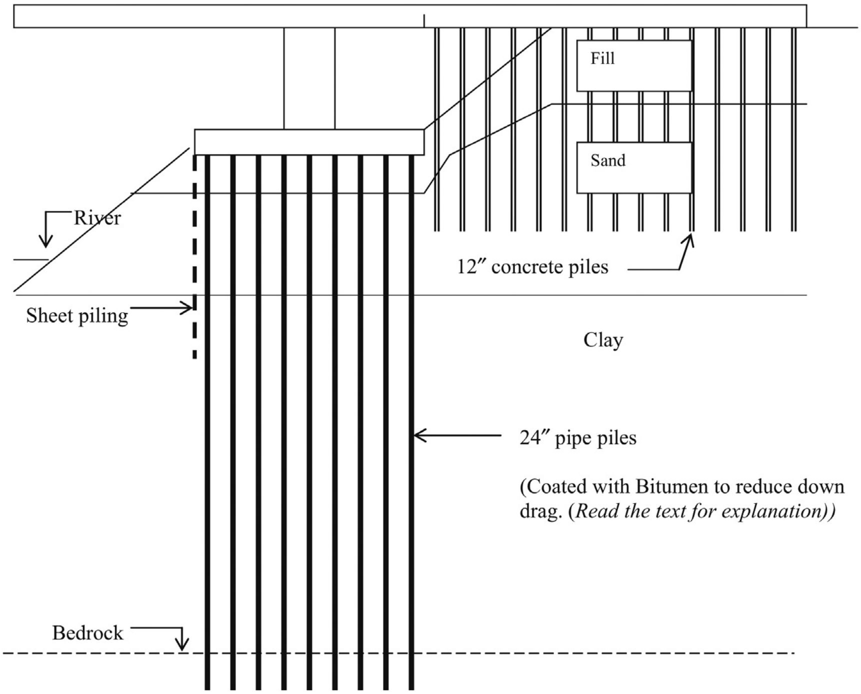


Figure 16.10 Case study: bitumen coated piles.

than pipe piles (without bitumen coating). A cost comparison between bitumen-coated pipe piles and H-piles was done and bitumen-coated pipe piles were selected.

References

Briaud, J.L., 1997. Bitumen election for reduction of downdrag on piles. ASCE Geotech. Geoenviron. Eng. 10, 1061–1090.

Visher, S.S., 1954. Climate Atlas of the United States. Harvard University Press, Cambridge, MA.

Laterally loaded piles

17

Lateral loads are exerted on piles due to wind, soil, and water. In such cases lateral pile capacity needs to be designed to accommodate the loading (Fig. 17.1).

- The deformation of piles due to lateral loading is normally limited to the upper part of the pile. Lateral pile deflection, 8–10 diameter below the ground level, is negligible in most cases.
- Piles, which can carry heavy vertical loads, may be very weak under lateral loads.

17.1 *p–y* curve method

17.1.1 *Problem statement*

We need to predict the displacement at the top of the pile for a given lateral load.

Fig. 17.2 shows a single pile and a pile cap. Wind or seismic (lateral) loads acting on the column will be transferred to the pile cap and then to the pile. For simplicity, we can represent pile, lateral force, and soil (Fig. 17.3).

17.1.2 *Question*

What is the deflection (d) at the top of the pile due to horizontal force (H)? We cannot answer this question without learning the following items.

- What is the magnitude of the force "H"
- What is the diameter of the pile? What material is it made up of?
- What is the type of this pile and its size (circular concrete pile, hollow pipe pile, or I-section)?
- What is the soil type (clay, sand, silt etc.)?
- Give the soil parameters (cohesion, friction, Young's modulus, Poisson's ratio, and density).
- What is the fixity at the top of the pile? Is the pile fixed at the top or pin jointed?

Let us assume that all these parameters are given. Then how do we solve the problem? The soil is represented with springs as shown in Fig. 17.4. Soils are not perfect elastic materials.

When the horizontal force "H" increases, horizontal force at point A (p_A) will also increase. Due to the force p_A, there would be deflection in the pile. The p_A versus y_A curve depends on the soil type, soil strength, and soil density and pile parameters.

Here, p_A, horizontal force at point A; and y_A, deflection at point A.

Hence, we can represent the soil resistance at different locations with p versus y curves as shown in Fig. 17.5.

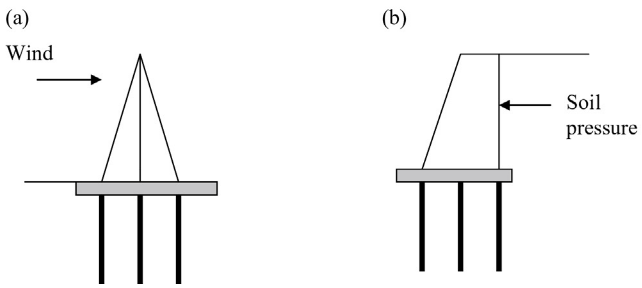


Figure 17.1 Laterally loaded piles. (a) Transmissin tower. (b) Earth retaining structure.

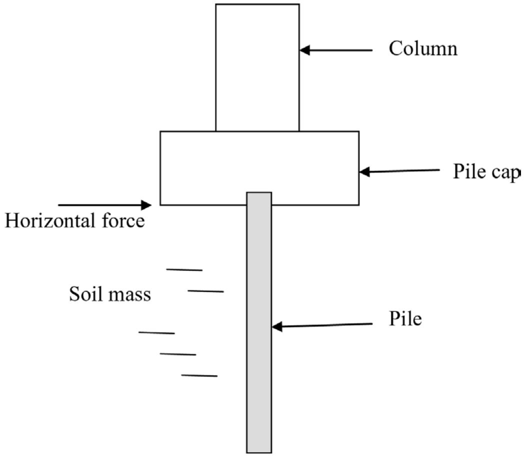


Figure 17.2 The *p–y* curve method.

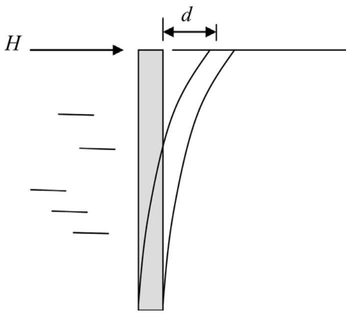


Figure 17.3 Pile, lateral force, and soil.

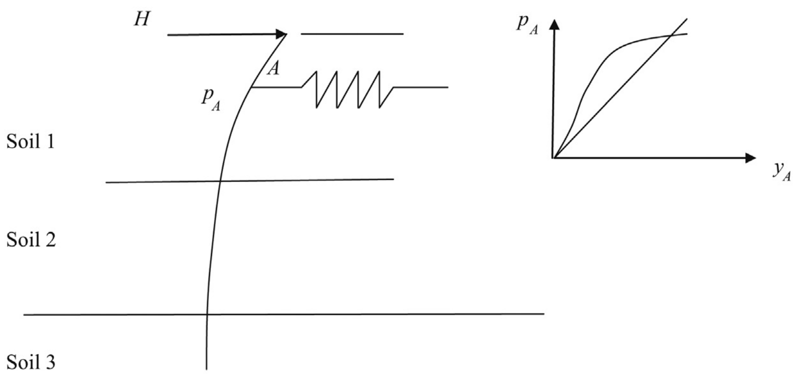


Figure 17.4 Soil representation with springs and elasticity of soil.

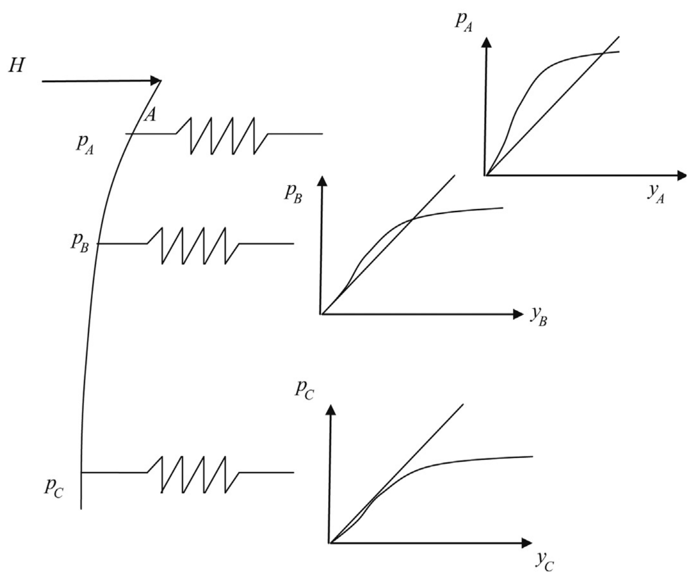


Figure 17.5 Soil resistance at various locations.

In the aforementioned pile, three different points are considered. In reality, many more points will be considered. At each location there would be a corresponding *p* versus *y* curve (force versus deflection). This curve depends on soil properties and pile properties. The *p* versus *y* curves for different soils and for different piles is developed through experiments. Using these experimental data, computer programs are capable of computing the deflection at the top of the pile. The following programs are widely used.

- Lpile: Lpile by Ensoft company
- S-shaft: S-shat by JP Singh Associates
- COM 624p: by FHWA (Federal Highway Administration)

17.2 Lateral loading analysis: simple procedure

17.2.1 Design methodology of laterally loaded piles

It is not possible to conduct a *p*–*y* curve analysis without a computer. The following method is much more simple and can be computed manually.

- It is assumed that a pile is being held by springs as shown in Fig. 17.6. The spring constant or the coefficient of subgrade reaction varies with the depth. In most cases, coefficient of subgrade reaction increases with depth.
- A simplified analysis of lateral loads on piles can be conducted by assuming the coefficient of subgrade reaction to be a constant with depth. For most cases the error induced by this assumption is not significant.
- When a pile is subjected to a horizontal load, it would try to deflect.
- The surrounding soil would generate a resistance against deflection.
- The resistance provided by the soil is represented by a series of springs. The spring constant is taken as the coefficient of subgrade reaction (k).
- In reality (k) changes with depth.
- Simplified analysis is conducted assuming (k) to be a constant.
- For most cases this assumption does not produce a significant error.
- The equation for lateral load analysis is given as follows (Matlock and Reese, 1960):

$$u = 2^{1/2}\ \frac{H}{k} \times \left(\frac{l_c}{4}\right)^{-1} + \frac{M}{k} \times \left(\frac{l_c}{4}\right)^{-2}$$

where u, lateral deflection; H, applied lateral load on the pile (normally due to wind or earth pressure); k, coefficient of subgrade reaction (assumed to be a constant with depth); M, moment induced due to lateral forces (when the lateral load is acting at a height above the ground level, then moment induced also should be taken into consideration); and l_c, critical pile length (below this length, the pile is acting as an infinitely long pile).

Now, l_c is obtained by using the following equation:

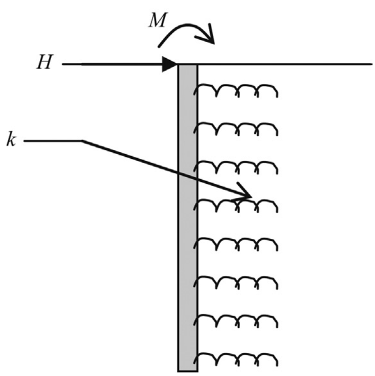


Figure 17.6 Design methodology.

Table 17.1 Coefficient of subgrade reaction (k) versus N (SPT, Johnson and Kavanaugh, 1968)

SPT (N)	8	10	15	20	30
k (kN/m^3)	2.67 E−6	4.08 E−6	7.38 E−6	9.74 E−6	1.45 E−6

$$l_c = 4\left[\frac{(EI)_p}{k}\right]^{1/4}$$

$(EI)_p$, Young's modulus and moment of inertia of the pile. In the case of wind loading, the moment of inertia should be taken against the axis, which has the minimum moment of inertia, since wind load could act from any direction.
In the case of soil pressure and water pressure, the direction of the lateral load does not change. In these situations, the moment of inertia should be taken against the axis of bending.

- A similar equation is obtained for the rotational angle (θ) at the top of the pile.

$$\theta = \frac{H}{k}\times\left(\frac{l_c}{4}\right)^{-2} + 2^{1/2}\frac{M}{k}\times\left(\frac{l_c}{4}\right)^{-3}$$

Derivation of this equation is provided by Matlock and Reese (1960).
ϕ' value of sandy soil can be calculated using the following equation:
$\phi' = 53.881 - 27.6034\ e^{-0.0147N}$ (Peck et al., 1974), where (N, average SPT value of the strata).

- Note: coefficient of subgrade reaction (k) can be obtained using Table 17.1.
Similarly soil parameters for other strata also need to be provided.

17.2.2 Soil parameters for clayey soils

Soil parameters required for clayey soils:

- S_u (undrained shear strength. "S_u" is obtained by conducting unconfined compressive strength tests).
- ε_c (strain corresponding to 50% of the ultimate stress. If the ultimate stress is 3 tsi, then "ε_c" is the strain at 1.5 tsi).
- k_s (coefficient of subgrade reaction).

Coefficient of subgrade reaction for clay soils is obtained from Table 17.2.

Table 17.2 Coefficient of subgrade reaction versus undrained shear strength (Reese, 1975)

	Average undrained shear strength (tsf)		
k_s (lb/in^3)	**0.5–1**	**1–2**	**2–4**
Static	500	1000	2000
Cyclic	200	400	800

References

Johnson, S.M., Kavanaugh, T.C., 1968. The Design of Foundations for Buildings. McGraw Hills, NY.

Matlock, H., Reese, L.C., 1960. Generalized solution for laterally loaded piles. ASCE J. Soil Mech. Found. Eng. 86, 63–91.

Peck, et al.,1974. Foundation Engineering. John Wiley and Sons, NY.

Reese, L.C., 1975. Field Testing and Analysis of Laterally Loaded Piles in Stiff Clay, Proceedings—Offshore Technology Conference, vol. 2. Houston, Texas.

Short course on seismology

18

A general understanding of seismology is needed to design piles for seismic events.

Fig. 18.1 shows a pendulum moving back and forth drawing a straight line prior to an earthquake event. Fig. 18.2 shows the same pendulum drawing a wavy line during an earthquake event.

- Seismographs are designed using the mentioned principle.
- Due to the movement of the pile cap, piles would be subjected to additional shear forces and bending moments.
- Earthquakes occur due to disturbances occurring inside the Earth's crust. Earthquakes would produce three main types of waves:
  1. *P*-waves (primary waves): they are also known as compression waves or longitudinal waves
  2. *S*-waves (secondary waves): they are also known as shear waves or transverse waves
  3. Surface waves: they are shear waves that travel near the surface (Fig. 18.3)

18.1 Faults

Faults are common occurrences in the earth. Fortunately, most faults are inactive and will not cause earthquakes. A fault is a fracture where a block of earth had moved relative to another.

18.1.1 Horizontal fault

- In a horizontal fault, one earth block had moved horizontally relative to the other. The movement is horizontal (Fig. 18.4).

18.1.2 Vertical fault (strike slip faults)

In this type of fault, one block moves in a downward direction relative to the other (Fig. 18.5).

18.1.3 Active fault

An active fault is defined as a fault in which there was an average historic slip rate of 1 mm/year or more during the last 11,000 years (IBC, International Building Code).

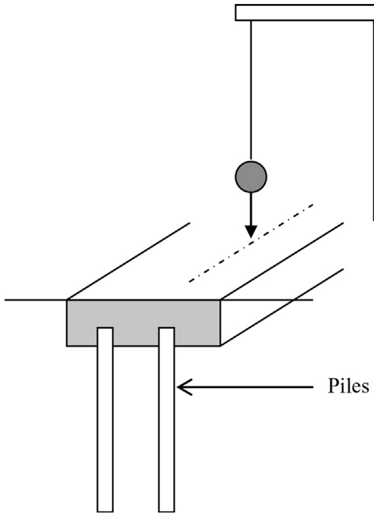


Figure 18.1 No earthquake.

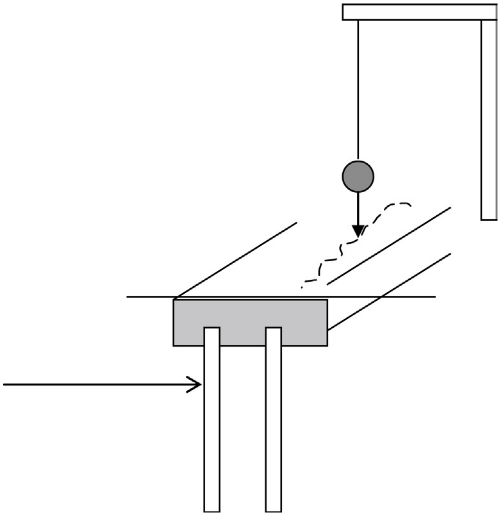


Figure 18.2 Earthquake event.

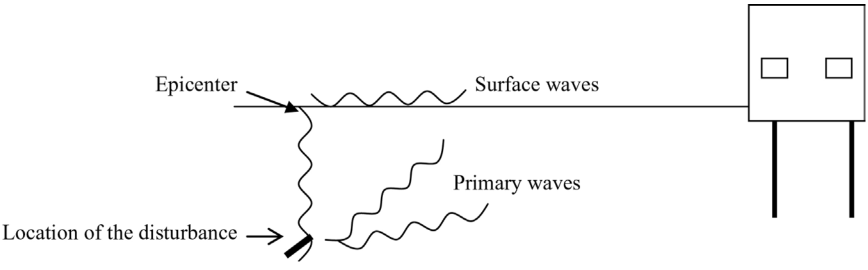


Figure 18.3 Seismic waves.

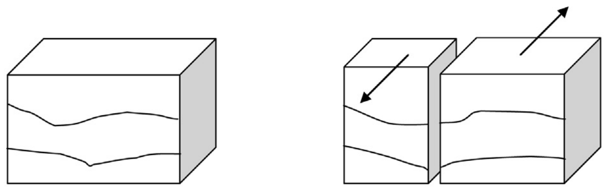


Figure 18.4 Movement of blocks.

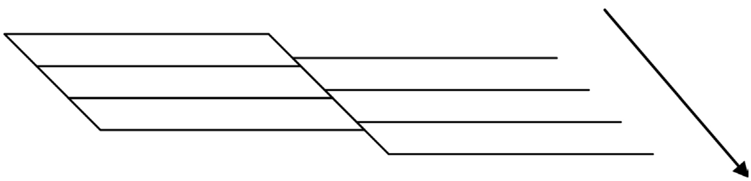


Figure 18.5 Strike slip faults.

18.1.4 Richter magnitude scale (M)

$$M = \log(A) - \log(A_0) = \log\left(\frac{A}{A_0}\right)$$

Here M, Richter magnitude scale; A, maximum trace amplitude during the earthquake; and A_0, standard amplitude (standard value of 0.001 mm is used for comparison. This corresponds to a very small earthquake).

Design example

What is the Richter magnitude scale for an earthquake that recorded an amplitude of (a) 0.001 mm, (b) 0.01 mm, and (c) 10 mm?

Answer

(a) $A = 0.001$ mm
$M = \log(A/A_0) = \log(0.001/0.001) = \log(1) = 0$
$M = 0$
(b) $A = 0.01$
$M = \log(A/A_0) = \log(0.01/0.001) = \log(10) = 1$
$M = 1$
(c) $A = 10$ mm
$M = \log(A/A_0) = \log(10/0.001) = \log(10{,}000) = 4$
$M = 4$

Table 18.1 **Earthquakes recorded**

Location	Date	Magnitude
Chile	1960	9.5
Prince William, Alaska	1964	9.2
Aleutian Islands	1957	9.1
Kamchatka	1952	9.0
Ecuador	1906	8.8
Rat Islands	1965	8.7
India–China border	1950	8.6
Kamchatka	1923	8.5
Indonesia	1938	8.5
Kuril Islands	1963	8.5

USGS Website (United States Geological Survey; www.usgs.org)

18.2 Largest earthquakes recorded

Some of the largest earthquakes that have been recorded on the world have been enlisted in Table 18.1.

18.2.1 Ground acceleration

This is a very important parameter for geotechnical engineers. During an earthquake, soil particles accelerate. Acceleration of soil particles could be either horizontal or vertical.

18.2.2 Seismic waves

The following partial differential equation represents seismic waves:

$$G\left(\frac{\vartheta^2 u}{\vartheta x^2}+\frac{\vartheta^2 u}{\vartheta z^2}\right)=\rho\left(\frac{\vartheta^2 u}{\vartheta t^2}+\frac{\vartheta^2 v}{\vartheta t^2}\right)$$

Here, G, shear modulus of soil; ϑ, partial differential operator; u, horizontal motion of soil; v, vertical motion of soil; and ρ, density of soil.

Fortunately, geotechnical engineers are not called upon to solve this partial differential equation.

A seismic waveform described by the aforementioned equation would create shear forces and bending moments on piles.

18.2.3 Seismic wave velocities

Velocity of seismic waves is dependent upon the soil/rock type. Seismic waves travel much faster in sound rock than in soils (Table 18.2).

Table 18.2 **Seismic wave velocities (Peck et al., 1974)**

Soil/rock type	Velocity (ft/s)
Dry silt, sand, loose gravel, loam, loose rock, and moist fine grained top soil	600–2,500
Compact till, gravel below water table, compact clayey gravel, cemented sand, and sandy clay	2,500–7,500
Weathered rock, partly decomposed rock, and fractured rock	2,000–10,000
Sound shale	2,500–11,000
Sound sandstone	5,000–14,000
Sound limestone and chalk	6,000–20,000
Sound igneous rock (granite and diabase)	12,000–20,000
Sound metamorphic rock	10,000–16,000

Reference

Peck, R.B., Hanson, W.B., Thorburn, T.H., 1974. Foundation Engineering. John Wiley & Sons Inc.

Seismic analysis of piles

19

Earthquakes would cause additional bending moments and shear forces on piles. Earthquake-induced bending moments and shear forces can be categorized into three types:

1. kinematic loads
2. inertial loads
3. loads due to liquefaction

19.1 Kinematic loads

Seismic waves travel at *different velocities in different soils*. Due to these differences, the pile will be subjected to bending and shear forces. This bending is known as kinematic pile bending (Fig. 19.1).

V_{s1} and V_{s2} are seismic wave velocities in layers 1 and 2, respectively. The pile is subjected to differential forces due to different seismic waves arriving in two soil layers. It should be mentioned here that kinematic pile bending could occur in homogeneous soils as well. This is due to the fact that the seismic waveform could have different strengths depending upon the depth and surrounding structures that could dampen the wave in a nonuniform manner.

- Kinematic pile bending could occur in free piles also (piles that are not supporting building structures).
- Maximum bending moment in the pile occurs near the interface of two layers. In Fig. 19.1, maximum bending moment occurs at point "*A*."

19.2 Inertial loads

In addition to kinematic loads, seismic waves could induce inertial loads as well (Fig. 19.2).

19.2.1 Inertial loading mechanism

Inertial loading mechanism on piles is different from the kinematic loading mechanism. Inertial loading occurs due to the building mass acting on piles.

When a seismic wave reaches the pile, the pile would be accelerated. Assume the acceleration at the top of the pile to be "*a*." The pile is not free to accelerate since it is attached to a pile cap and then to a structure on top. Due to the mass of the structure on top, the pile will be subjected to inertial forces and bending moments.

Pile Design and Construction Rules of Thumb

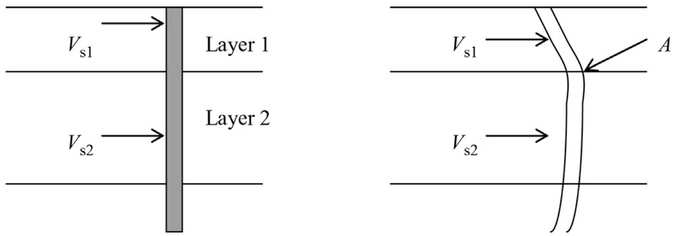


Figure 19.1 Kinematic loads.

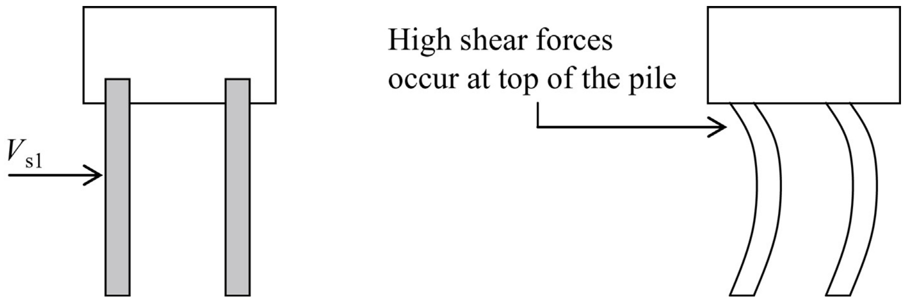


Figure 19.2 Seismic wave induced loads.

Shear force at the top of the pile can be computed using Newton's equation:

Shear force = Ma

Here, M, portion of the building mass acting on the pile and a, acceleration of the pile.

- Inertial loadings due to seismic waves are limited to the top $10d$–$15d$ measured from the surface (here d is the diameter of the pile).
- On the other hand, kinematic loading could occur at any depth. If a pile had failed at a greater depth, then it is reasonable to assume that the pile failed due to kinematic loading.

19.2.2 Soil liquefaction

Soil liquefaction occurs in sandy soils below the groundwater level. During an earthquake loose sandy soils tend to liquefy (reach a liquid-like state). When this happens, the piles would lose the lateral support provided by soil.

19.2.3 Design of piles for kinematic loadings

The Fig. 19.3 depicts the design of piles for kinematic loadings. Here, G, soil shear modulus; k, soil subgrade modulus or the spring constant; E, Young's modulus of soil; E_p, Young's modulus of pile.

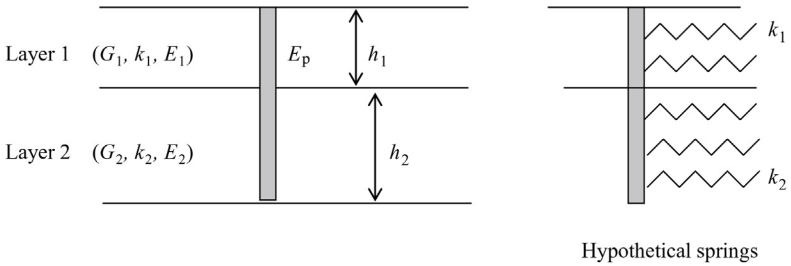


Figure 19.3 Hypothetical springs.

19.2.4 Relationships

$$k = 3G$$

The k/G ratio ranges from 2.5 to 4.0 for soils. Besides, $k = 3G$ is a good approximation for any soil (Dobry and O'Rourke, 1983). It is shown that pile-bending moment is proportional to $(k)^{1/4}$ and any attempt to find an accurate value for "k" is not warranted (Fig. 19.3).

$$\frac{k_1}{E_1} = \frac{k_2}{E_2} = d$$

E_1, E_2 = Young's modulus of soil layers 1 and 2, respectively

Here, "δ" is a dimensionless parameter (Mylanakis, 2001). "δ" can be computed using the following equation.

$$\delta = \frac{3}{1-\nu^2}\left[\left(\frac{E_p}{E_1}\right)^{-1/8}\left(\frac{L}{d}\right)^{1/8}\left(\frac{h_1}{h_2}\right)^{1/2}\left(\frac{G_2}{G_1}\right)^{-1/30}\right]$$

Here d, pile diameter; L, pile length; E_p, Young's modulus of the pile; and h_1, h_2, thickness of soil layers 1 and 2, respectively.

19.2.5 Pile bending strain

Pile bending strain has been shown in Fig. 19.4. Here, y, distance from center to outermost fibers; ε_p, bending strain at the outermost fiber of the pile; E_p, Young's modulus = stress (σ_p)/strain (ε_p).

$$E = \frac{\sigma_p}{\varepsilon_p}$$

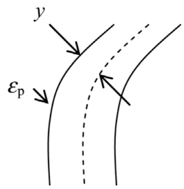


Figure 19.4 Bending strain.

19.2.5.1 Bending equation

$$\frac{M}{I} = \frac{\sigma_p}{y}$$

Here, I = moment of inertia; M, bending moment. For y, see Fig. 19.4.

$$M = \frac{\sigma_p \cdot I}{y} = \frac{E_p \cdot \varepsilon_p \cdot I}{y}$$

If "ε_p" (bending strain) can be deduced, maximum bending moment in the pile can be computed.

19.2.6 Seismic pile design for kinematic loads

19.2.6.1 Step by step procedure

Step 1: find the peak soil shear strain at the soil interface of two layers (Seed and Idriss, 1982).

Interface peak shear strain due to the soil acceleration is given in this equation (Fig. 19.5):

$$\gamma_1 = \frac{r_d \times \rho_1 \times h_1 \times a_s}{G_1}$$

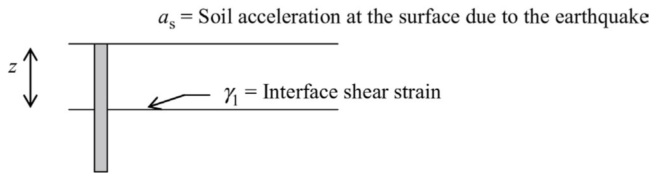


Figure 19.5 Interface shear strain.

Here, γ_1, peak shear strain at the interface of two soil layers; a_s, soil acceleration due to the earthquake at the surface; r_d, depth factor ($r_d = 1 - 0.015z$; z, depth to the interface measured in meters); ρ_1, soil density of the top layer; and h_1, thickness of the top soil layer.

Step 2: find "δ" (the ratio between k and E).

$$\delta = \frac{k_1}{E_1} = \frac{k_2}{E_2}$$

Here, k_1 and k_2, spring constants of layers 1 and 2, respectively; and E_1 and E_2, Young's modulus of layers 1 and 2, respectively.

"δ" is given by the following equation:

$$\delta = \frac{3}{1 - \nu^2}\left[\left(\frac{E_p}{E_1}\right)^{-1/8}\left(\frac{L}{d}\right)^{1/8}\left(\frac{h_1}{h_2}\right)^{1/2}\left(\frac{G_2}{G_1}\right)^{-1/30}\right]$$

Step 3: Find the strain transfer ratio

$$\left(\frac{\varepsilon_p}{\gamma_1}\right)$$

$$\left(\frac{\varepsilon_p}{\gamma_1}\right) = \frac{(c^2 - c + 1)\left\{\left[3\left(\frac{k_1}{E_p}\right)^{1/4}\left(\frac{h_1}{d}\right) - 1\right]c\,(c-1) - 1\right\}}{2\,c^4\left(\frac{h_1}{d}\right)}$$

$$c = \frac{G_2}{G_1}^{1/4}$$

Here, ε_p, bending strain at the outermost fiber of the pile; and γ_1, interface shear strain (computed in Step 1).

Values of G_1, G_2, k_1, k_2, E_p, h_1, h_2, and d are required to compute ε_p.

Step 4: find the bending moment (M) in the pile.

$M = (\sigma_p \cdot I)/y$ (See chapter: Pile Bending Strain)

E (Young's modulus) = stress/strain = σ_p/ε_p.

$$M = \left(\frac{E_p \cdot \varepsilon_p \cdot I}{y}\right)$$

Since "ε_p" was calculated and "y," "I," and E_p are known pile properties, bending moment (M) induced due to earthquake can be deduced.

Design example

Find the kinematic bending moment induced in the pile shown due to an earthquake that produces a surface acceleration of $0.5g$ (Fig. 19.6).

Young's modulus of pile material (E_p) = 3.5×10^7 kPa; pile diameter = 0.5 m.

The following parameters need to be deduced from the given information (G_1, G_2, E_1, ε_p).

Note that instead of soil shear modulus, shear wave velocity is given.

Step 1: Find "G_2/G_1" (shear modulus ratio of two layers).

Shear modulus is proportional to soil density and shear wave velocity. The relationship between shear wave velocity and soil shear modulus is given here.

$$G = \rho \times V_s^2$$

Hence, $G_1 = \rho_1 \times V_{s1}^2$ and $G_2 = \rho_2 \times V_{s2}^2$

$$G_1 = 1.2 \times 60^2 = 4320; G_2 = 1.8 \times 100^2$$

$$\frac{G_2}{G_1} = \frac{(1.8 \times 100^2)}{(1.2 \times 60^2)} = 4.17$$

Step 2: find the Young's modulus of soil layer 1.

The following relationship between Young's modulus and shear modulus can be used to find the Young's modulus of the soil layer:

$$E = 2G(1+\nu)$$

$$E_1 = 2G_1(1+\nu_1) = 2 \times 4,320(1+0.5) = 12,960 \text{ kPa}$$

Step 3: find the ratio (E_p/E_1).

$$\frac{E_p}{E_1} = \frac{(3.5 \times 10^7)}{(12,960)} = 2,700$$

(E_p, Young's modulus of the pile is given and E_1 is computed in Step 2)

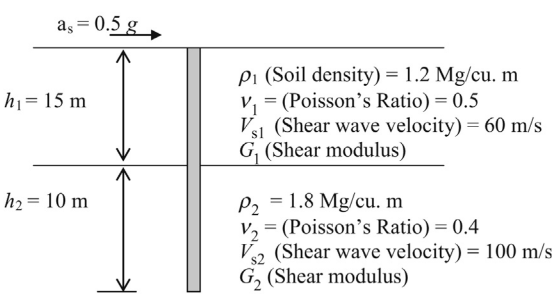


Figure 19.6 Soil layers.

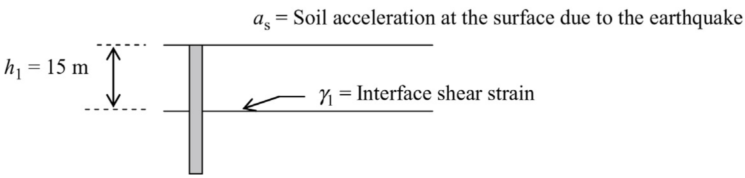


Figure 19.7 Peak soil shear strain at the soil interface of two layers.

Step 4: find the peak soil shear strain at the soil interface of two layers (Fig. 19.7) (Seed and Idriss, 1982).

Peak interface shear strain due to the soil acceleration is

$$\gamma_1 = \frac{r_d \times \rho_1 \times h_1 \times a_s}{G_1}$$

r_d, depth factor; $r_d = 1 - 0.015\,z$;
z, depth to the interface measured in meter

$r_d = 1 - 0.015 \times h_1 = 1 - 0.015 \times 15 = 0.775$;
ρ_1, soil density; $\rho_1 = 1.2\,\mathrm{Mg/m^3}$, $a_s = 0.5\,g$; $G_1 = 4320$ (as computed in Step 1)

$$\gamma_1 = \frac{0.775 \times 1.2 \times 15 \times 0.5 \times 9.81}{4320} = 1.6 \times 10^{-2}$$

Step 5: find "δ" (the ratio between k and E).

$$\delta = \frac{k_1}{E_1} = \frac{k_2}{E_2}$$

Here, k_1 and k_2 = spring constants in layers 1 and 2, respectively, and E_1 and E_2 are Young's modulus in layers 1 and 2, respectively.

"δ" is given by the following equation:

$$\delta = \left(\frac{3}{1-\nu^2}\right) \times \left(\frac{E_p}{E_1}\right)^{-1/8} \left(\frac{L}{d}\right)^{1/8} \left(\frac{h_1}{h_2}\right)^{1/2} \left(\frac{G_2}{G_1}\right)^{-1/30}$$

$\frac{E_p}{E_1} = 2700$ (calculated in Step 3); $\frac{G_2}{G_1} = 4.17$ (as calculated in Step 1);
Pile diameter (d) = 0.5 m (pile diameter is given); L = pile length = 25 m

$$\delta = \frac{3}{1-0.52} \times \left[(2700)^{-1/8} \left(\frac{25}{0.5}\right)^{1/8} \left(\frac{15}{10}\right)^{1/2} (4.17)^{-1/30} \right]$$

$$\delta = 4 \times [0.372 \times 1.63 \times 1.22 \times 0.954] = 2.82.$$

Step 6: find the strain transfer ratio (ε_p/γ_1).

$$\left(\frac{\varepsilon_p}{\gamma_1}\right)=\frac{(c^2-c+1)\left\{\left[3\left(\frac{k_1}{E_p}\right)^{1/4}\left(\frac{h_1}{d}\right)-1\right]c\,(c-1)-1\right\}}{2\,c^4\left(\frac{h_1}{d}\right)}$$

$$c=\left(\frac{G_2}{G_1}\right)^{1/4}=1.429,\ \frac{G_2}{G_1}=4.17\text{ (see step 1)},$$

ε_p = bending strain at the outermost fiber of the pile and

γ_1 = interface shear strain (see step 4).

All parameters in the equation are known except for soil spring constant (k_1). Soil spring constant can be computed using the following equation:

$$\delta=\frac{k_1}{E_1}=\frac{k_2}{E_2}$$

$E_1 = 12{,}960$ kPa (see step 2); $\delta = 2.82$ (from step 5).

$$2.82=\frac{k_1}{12{,}960};\qquad k_1=36{,}547\text{ kPa};\qquad E_p=3.5\times10^7\text{ kPa (pile hammer)}$$

$$\left(\frac{\varepsilon_p}{\gamma_1}\right)=\frac{(c^2-c+1)\left\{\left[3\left(\frac{k_1}{E_p}\right)^{1/4}\left(\frac{h_1}{d}\right)-1\right]c\,(c-1)-1\right\}}{2\,c^4\left(\frac{h_1}{d}\right)}$$

$$\left(\frac{\varepsilon_p}{\gamma_1}\right)=\frac{(1.429^2-1.429+1)\left\{\left[3\frac{36{,}547}{(3.5\times10^7)^{1/4}}\times\left(\frac{15}{0.5}\right)-1\right]\times1.429\times(1.429-1)-1\right\}}{2\times1.429^4\left(\frac{15}{0.5}\right)}$$

$$\left(\frac{\varepsilon_p}{\gamma_1}\right)=\frac{1.613\{[16.17-1]\times0.613-1\}=0.056}{250.2}$$

$$\gamma_1=1.6\times10^{-2}\text{ (from step 4)}$$

Hence, $\varepsilon_p=1.6\times10^{-2}\times0.056=8.96\times10^{-4}$.

Step 7: find the bending moment (M) in the pile.

$M = (\sigma_p I)/y$ (see chapter: Pile Bending Strain)

$$E_p\text{ (Young's modulus of the pile)}=\frac{\text{Stress}}{\text{Strain}}=\frac{\sigma_p}{\varepsilon_p}$$

$$M=\frac{E_p\cdot\varepsilon_p I}{y}\qquad y=\frac{d}{2}=\frac{0.5}{2}=0.25\text{ m}$$

$$I\ (\text{moment of inertia}) = \frac{\pi \cdot d^4}{64} = \frac{\pi \cdot 0.5^4}{64} = 3.068 \times 10^{-3}$$

$$\varepsilon_p = 8.96 \times 10^{-4}\ (\text{see step 6})$$

$$M = \frac{3.5 \times 10^7 \times 8.96 \times 10^{-4} \times 3.068 \times 10^{-3}}{0.25} = 384.8\ \text{kNm}$$

- The pile should be designed to withstand an additional bending moment of 384.8 kN · m induced due to earthquake loading.

19.3 Seismic pile design: inertial loads

As mentioned earlier, inertial loads occur in piles due to shaking of the building.

Assume a pile as shown in Fig. 19.8.

- When the building starts to shake due to an earthquake event, the piles would be subjected to additional shear forces and bending moments.
- Usually, loads occurring in the piles due to building loads are limited to "15*d*" from the ground surface (*d* = diameter of the pile).
- Unfortunately, there is no easy equation to represent the motion of the building and piles. The following partial differential equation has been proposed:

$$\frac{\vartheta V}{\vartheta Z} \cdot \mathrm{d}Z + \rho \cdot \frac{\vartheta^2 y}{\vartheta t^2} \cdot \mathrm{d}Z + k \cdot D\,(y - u) \cdot \mathrm{d}Z = 0$$

Here, V = shear force in the pile at a depth of "Z." Z, depth; d, pile diameter; u, horizontal soil movement due to the earthquake; and v, horizontal pile movement due to the earthquake.

Now, $(u - v)$ = horizontal pile movement relative to soil due to the earthquake; and ρ, pile density.

- The given equation cannot be solved with reasonable accuracy by manual methods. Hence, computer programs are used to calculate the shear forces that develop in piles.
- The "SHAKE" computer program by Schnable Engineering is one such program that could be used for the purpose.

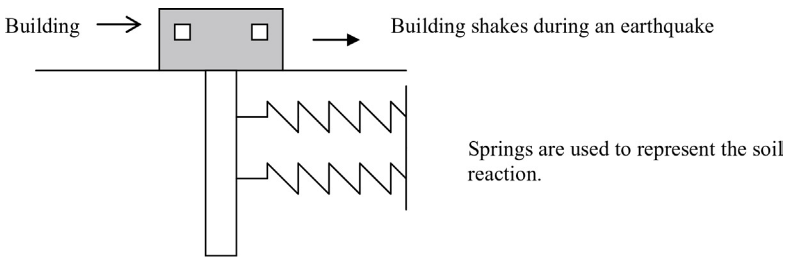


Figure 19.8 Seismic pile design and inertial loads.

19.4 Liquefaction analysis

19.4.1 Theory

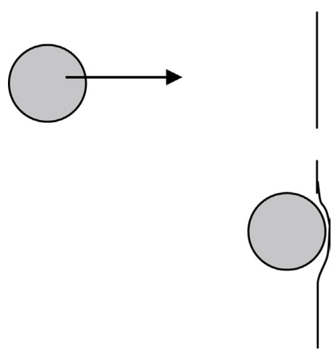
Sandy and silty soils tend to lose strength and turn into a liquid-like state during an earthquake. This happens due to the increase of pore pressure during an earthquake event in the soil caused by seismic waves (Fig. 19.9).

- Liquefaction of soil was thoroughly studied by Bolten Seed and I.M. Idris during the 1970s. As one would expect, the liquefaction behavior of soil cannot be expressed in one simple equation. Many correlations and semiempirical equations have been introduced by researchers. Due to this reason, Professor Robert W. Whitman convened a workshop in 1985 (Ref. 1) on behalf of the National Research Council (NRC). Experts from many countries participated in this workshop and a procedure was developed to evaluate the liquefaction behavior of soils.
- It should be mentioned here that only sandy and silty soils tend to liquefy. Clay soils do not undergo liquefaction.

19.5 Impact due to earthquakes

Imagine a bullet hitting a wall.

The extent of damage to the wall due to the bullet depends on a number of parameters.



19.5.1 Bullet properties

Some properties of the bullet are as follows:

1. velocity of the bullet
2. weight of the bullet
3. hardness of the bullet material

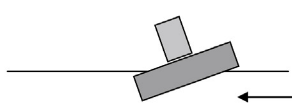


Figure 19.9 Liquefaction analysis.

19.5.2 Wall properties

Some wall properties are as follows:

1. hardness of the wall material
2. type of wall material

The parameters that affect liquefaction are given further.

19.5.3 Earthquake properties

- Magnitude of the earthquake.
- Peak horizontal acceleration at the ground surface (a_{max}).

19.5.4 Soil properties

- Soil strength (measured by standard penetration test (SPT) value).
- Effective stress at the point of liquefaction.
- Content of fines (fines are defined as particles that pass through the #200 sieve).
- Earthquake properties that affect the liquefaction of a soil is amalgamated into one parameter known as cyclic stress ratio (*CSR*).

$$\text{Cyclic stress ratio } (CSR) = 0.65\left(\frac{a_{max}}{g}\right) \times \left(\frac{\sigma}{\sigma'}\right) \times r_d \tag{19.1}$$

Here, a_{max}, peak horizontal acceleration at the ground surface; σ, total stress at the point of concern; σ', effective stress at the point of concern; and r_d,= stress reduction coefficient (this parameter accounts for the flexibility of the soil profile).

$$r_d = 1.0 - 0.00765Z \quad \text{for} \quad Z < 9.15 \text{ m} \tag{19.2}$$

$$r_d = 1.174 - 0.0267Z \quad \text{for} \quad 9.15 \text{ m} < Z < 23 \text{ m} \tag{19.3}$$

Z, depth to the point of concern in meters.

19.5.5 Soil resistance to liquefaction

- As a rule of thumb, any soil that has an SPT value higher than 30 will not liquefy.
- As mentioned, resistance to liquefaction of a soil depends on its strength measured by SPT value. Researchers have found that resistance to liquefaction of a soil depends on the content of *fines* as well.

The following equation can be used for clean sand (clean sand is defined as a sand with less than 5% fines):

$$CRR_{7.5} = \frac{1}{[34-(N_1)_{60}]} + \frac{(N_1)_{60}}{135} + \frac{50}{[10\cdot(N_1)_{60}+45]^2} - \frac{2}{200} \tag{19.4}$$

$CRR_{7.5}$, soil resistance to liquefaction for an earthquake with a magnitude of 7.5 Richter.

Correction factor needs to be applied to any other magnitude. That process is described later in the chapter. The presented equation can be used only for sands (content of fines should be less than 5%). Correction factor has to be used for soils with higher content of fines. That procedure is described later in the chapter.

$(N_1)_{60}$, standard penetration value corrected to a 60% hammer and an overburden pressure of 100 kPa (note that the given equations were developed in metric units).

19.5.6 How to obtain $(N_1)_{60}$

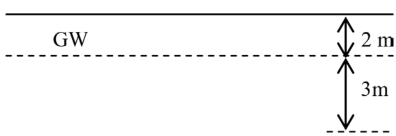
$$(N_1)_{60} = N_m \times C_N \times C_E \times C_B \times C_R \tag{19.5}$$

- N_m, SPT value measured in the field.
- C_N, overburden correction factor = $(P_a/\sigma')^{0.5} P_a = 100$ kPa; σ', effective stress of soil at point of measurement.
- C_E, energy correction factor for the SPT hammer. For donut hammers $C_E = 0.5–1.0$; for trip type donut hammers; $C_E = 0.8–1.3$.
- C_B, borehole diameter correction. For borehole diameters 65–115 mm use $C_B = 1.0$; for borehole diameter of 150 mm, use $C_B = 1.05$; for borehole diameter of 200 mm, use $C_B = 1.15$.
- C_R, rod length correction (rods attached to the SPT spoon would exert their weight on the soil. Longer rods would exert a higher load on soil and in some cases the spoon would go down due to the weight of rods without any hammer blows. Hence, correction is made to account for the weight of rods).

For rod length < 3 m, use $C_R = 0.75$; for rod length 3–4 m, use $C_R = 0.8$; for rod length 4–6 m, use $C_R = 0.85$; for rod length 6–10 m, use $C_R = 0.95$; for rod length 10–30 m, use $C_R = 1.0$.

Design example 19.1

Consider a point at a depth of 5 m in a sandy soil (fines < 5%). Total density of soil is 1800 kg/m^3. The groundwater is at a depth of 2 m. Corrected $(N_1)_{60}$ value is 15. Peak horizontal acceleration at the ground surface (a_{max}) was found to be 0.15g for an earthquake of magnitude 7.5. Check to see whether the soil at a depth of 5 m would liquefy under an earthquake of 7.5 magnitude.



Step 1: find the cyclic stress ratio.

$$\text{Cyclic stress ratio } (CSR) = 0.65\left(\frac{a_{max}}{g}\right) \times \left(\frac{\sigma}{\sigma'}\right) \times r_d \tag{19.6}$$

Here a_{max}, peak horizontal acceleration at the ground surface = 0.15g; σ, total stress at the point of concern; σ', effective stress at the point of concern; r_d, stress reduction coefficient (this parameter accounts for the flexibility of the soil profile).

$$r_d = 1.0 - 0.00765\,Z \quad \text{for} \quad Z < 9.15\,\text{m} \quad (Z \text{ is depth to the point of concern in meters}) \tag{19.7}$$

$$r_d = 1.174 - 0.0267\,Z \quad \text{for} \quad 9.15\,\text{m} < Z < 23\,\text{m} \tag{19.8}$$

$$\sigma = 5 \times 1800 = 9000\,\text{kg/m}^2$$

$$\sigma' = 2 \times 1800 + 3\,(1800 - 1000) = 6000\,\text{kg/m}^2$$

(Density of water = 1000 kg/m^2)

Since the depth of concern is 5 m (which is less than 9.15 m) use Equation (19.7) to find $r_{d.}$

$$r_d = 1.0 - 0.00765\,Z \quad \text{for} \quad Z < 9.15\,\text{m}; \quad r_d = 1.0 - 0.00765 \times 5 = 0.962$$

$$\text{Hence, } CSR = 0.65 \times (0.15) \times \frac{9000}{6000} \times 0.962 = 0.1407$$

Step 2: find the soil resistance to liquefaction.

$$CRR_{7.5} = \frac{1}{[34 - (N_1)60]} + \frac{(N_1)60}{135} + \frac{50}{[10 \cdot (N_1)_{60} + 45]^2} - \frac{2}{200} \tag{19.9}$$

$(N_1)_{60}$ value is given to be 15. Hence, $CRR_{7.5} = 0.155$.

Since soil resistance to liquefaction (0.155) is larger than the CSR value (0.1407), the soil at 5 m depth will not undergo liquefaction for an earthquake of magnitude 7.5.

Correction factor for magnitude

As you are aware Equation (19.4) (for $CRR_{7.5}$) is valid only for earthquakes of magnitude 7.5. Correction factor is proposed to account for magnitudes different from 7.5.

$$\text{Factor of safety (FOS) is given by} = (CRR_{7.5}/CSR) \tag{19.10}$$

$CRR_{7.5}$, resistance to soil liquefaction for a magnitude of 7.5 earthquake and CSR, cyclic stress ratio (which is a measure of the impact due to the earthquake load).

Factor of safety for any other earthquake is given by following equation:

$$\text{Factor of safety } (FOS) = \frac{CRR_{7.5}}{CSR} \times MSF \tag{19.11}$$

MSF is magnitude-scaling factor, given in Table 19.1.

Participants of the 1985 NRC (National Research Council) conference gave the freedom to engineers to select either of the values suggested by Idris or Andrus and Stokoe. As you can see Idris values are more conservative, and in noncritical buildings, such as warehouses, the engineers may be able to use Andrus and Stokoe values.

Design example 2

$CRR_{7.5}$ value of a soil was found to be 0.11. CSR value for the soil was computed to be 0.16. Will this soil liquefy for an earthquake of 6.5 magnitude?

Step 1: Find the factor of safety.

Factor of safety (FOS) = ($CRR_{7.5}$ /CSR) × MSF

Table 19.1 Magnitude scaling factors

Earthquake magnitude	MSF suggested by Idris (1995)	MSF suggested by Andrus and Stokoe
5.5	2.2	2.8
6.0	1.76	2.1
6.5	1.44	1.6
7.0	1.19	1.25
7.5	1.00	1.00
8.0	0.84	
8.5	0.72	

MSF for an earthquake of 6.5 = 1.44 (Idris)
Factor of safety = (0.11/0.16) × 1.44 = 0.99 (soil would liquefy).
Use *MSF* given by Andrus and Stokoe.
Factor of safety = (0.11/0.16) × 1.6 = 1.1 (soil would not liquefy).

Correction factor for content of fines

Equation (19.4) was developed for clean sand with fines content less than 5%. Correction factor is suggested for soils with higher fines contents.

$$CRR_{7.5} = \frac{1}{[34 - (N_1)_{60}]} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \cdot (N_1)_{60} + 45]^2} - \frac{2}{200} \tag{19.12}$$

Corrected $(N_1)_{60}$ value should be used in the aforementioned equation for soils with higher fines content.

The following procedure should be followed to find the correction factor:

- Compute $(N_1)_{60}$ as in the previous case.
- Use following equations to account for the fines content:

$$(N_1)_{60C} = a + b\,(N_1)_{60} \qquad (N_1)_{60C} = \text{Corrected } (N_1)_{60} \text{ value}$$

$$a = 0 \quad \text{for} \quad FC < 5\% \; (FC = \text{fines content}) \tag{19.13}$$

$$a = \exp\left(\frac{1.76 - 190}{FC^2}\right) \quad \text{for} \quad 5\% < FC < 35\% \tag{19.14}$$

$$a = 5.0 \quad \text{for} \quad FC > 35\% \tag{19.15}$$

$$b = 1.0 \quad \text{for} \quad FC < 5\% \tag{19.16}$$

$$b = \left[0.99 + \left(\frac{FC^{1.5}}{1000}\right)\right] \quad \text{for} \quad 5\% < FC < 35\% \tag{19.17}$$

$$b = 1.2 \quad \text{for} \quad FC > 35\% \tag{19.18}$$

Design example 3

$(N_1)_{60}$ value for soil with 30% fines content was found to be 20. Find the corrected $(N_1)_{60C}$ value for that soil.

Step 1: $(N_1)_{60C} = a + b(N_1)_{60}$

$$\text{For } FC = 30\%; \quad a = \exp\left(\frac{1.76 - 190}{FC^2}\right) = \exp\left(\frac{1.76-190}{30^2}\right) = 4.706$$

$$\text{For } FC = 30\%; \quad b = \left[0.99 + \frac{FC^{1.5}}{1000}\right] = 1.154 \tag{19.19}$$

$$(N_1)_{60C} = 4.706 + 1.154 \times 20 = 27.78$$

Design example 4

Consider a point at a depth of 5 m in a sandy soil (fines = 40%). Total density of soil is 1800 kg/m^3. The groundwater is at a depth of 2 m (γ_w = 1000 kg/m^3). Corrected $(N_1)_{60}$ value is 15 (all the correction parameters C_N, C_E, C_B, and C_R are applied, except for the fines content). Peak horizontal acceleration at the ground surface (a_{max}) was found to be 0.15g for an earthquake of magnitude 8.5. Check to see whether the soil at a depth of 5 m would liquefy under this earthquake load.

Step 1: find the cyclic stress ratio.

$$\text{Cyclic stress ratio } (CSR) = 0.65\left(\frac{a_{max}}{g}\right) \times \left(\frac{\sigma}{\sigma'}\right) \times r_d \tag{19.20}$$

$$\sigma = 5 \times 1800 = 9000 \text{ kg/m}^2; \qquad \sigma' = 2 \times 1800 + 3\,(1800 - 1000) = 6000 \text{ kg/m}^2$$

Since the depth of concern is 5 m (which is less than 9.15 m) use Equation (19.2)

$$r_d = 1.0 - 0.00765\,Z \quad \text{for} \quad Z < 9.15 \text{ m}; \quad r_d = 1.0 - 0.00765 \times 5 = 0.962$$

$$\text{Hence, } CSR = 0.65 \times (0.15) \times \left(\frac{9000}{6000}\right) \times 0.962 = 0.1407$$

Step 2: provide the correction factor for fines content.

For soils with 40% fines a = 5 and b = 1.2 (Equation (19.15) and (19.18)).

$$(N_1)_{60C} = a + b(N_1)_{60}$$
$$(N_1)_{60C} = 5 + 1.2 \times 15 = 23$$

Step 3: find the soil resistance to liquefaction.

$$CRR_{7.5} = \frac{1}{[34 - (N_1)_{60}]} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \cdot (N_1)_{60} + 45]^2} - \frac{2}{200} \tag{19.21}$$

$(N_1)_{60C}$ value is found to be 23 (see step 2)

Hence, $CRR_{7.5}$ = 0.25 (from Equation (19.4))

$$\text{Factor of safety } (FOS) = \left(\frac{CRR_{7.5}}{CSR}\right) \times MSF \tag{19.22}$$

MSF, magnitude scaling factor.
Obtain *MSF* from Table 19.1.
MSF = 0.72 for an earthquake of 8.5 magnitude (table by Idris).
CSR = 0.1407 (see step 1).
Factor of safety (*FOS*) = (0.25/0.1407) × 0.72 = 1.27.
The soil would not liquefy.

19.6 General guidelines for seismic pile design

The IBC (International Building Code) classifies sites based on SPT value and cohesion values (Table 19.2).

19.6.1 Precast concrete piles

19.6.1.1 Class C sites

- Longitudinal reinforcements shall be provided with a minimum steel ratio of 0.01. Lateral ties should be not less than 1/4 in thick.
- Lateral ties should not be placed more than 6 in. apart.
- Longitudinal and lateral reinforcements should be provided for the full length of the pile.

19.6.1.2 Class D, E, and F

- Class C requirements must be met.
- Additionally, lateral ties should not be placed more than 4 in. apart.

Table 19.2 Shear velocity

Site class	Soil/rock type	SPT	Cu (psf)	Soil shear velocity (V_s)
A	Hard rock	N/A	N/A	$V_s > 5{,}000$
B	Rock	N/A	N/A	$2{,}500 < V_s < 5{,}000$
C	Very dense soil and soft rock	$N > 50$	$C_u > 2{,}000$	$1{,}200 < V_s < 2{,}500$
D	Stiff soil	$15 < N < 50$	$1{,}000 < C_u < 2{,}000$	$600 < V_s < 1{,}200$
E	Soft soil	$N < 15$	$C_u < 1{,}000$	$V_s < 600$
F	Peats, liquefiable soils and high plastic soils (PI > 75%)			

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Batter pile design

20

Batter piles are widely used for retaining walls to resist lateral forces. Batter piles have higher lateral load capacity than vertical piles. Bridge abutments, retaining walls, and platforms can benefit from batter piles.

- Usually in most cases, one row of piles is battered as in Fig. 20.1. In some cases it is necessary to have more than one row of batter piles, as in Fig. 20.2.

20.1 Theory

20.1.1 Forces on batter piles

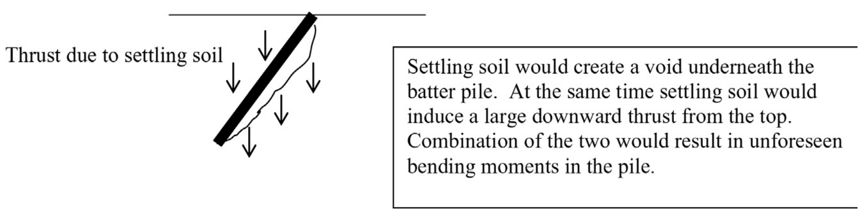
Batter piles are capable of resisting significant lateral forces. Lateral resistance of batter piles comes from two sources:

1. horizontal component of the axial reaction
2. horizontal resistance due to soil (H)

Note that horizontal resistance (H) and horizontal component of the pile axial reaction are two different quantities (Plate 20.1).

20.1.2 Negative skin friction

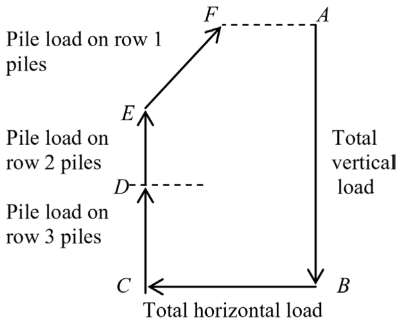
Batter piles should be avoided in situations where negative skin frictional forces can be present. Settling soil could induce large bending moments in batter piles.



20.1.3 Force polygon

Please refer to Fig. 20.1 for the following force polygon (calculation of loads will be shown later in the chapter):

Pile Design and Construction Rules of Thumb



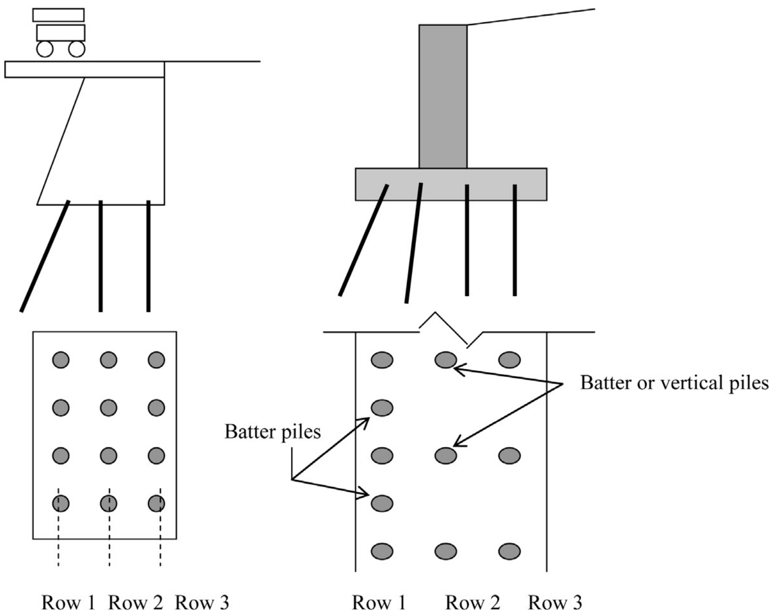


Figure 20.1 Batter piles. (a) Bridge abutments; and (b) typical profile for retaining wall with batter piles.

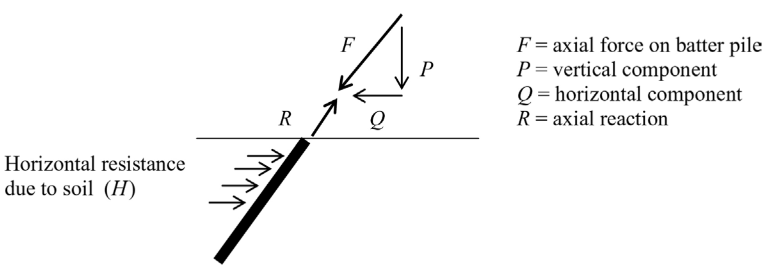


Figure 20.2 Forces in batter piles.

Plate 20.1 Batter piles.

Line *AB*: calculate the total vertical load on piles. Draw the total vertical load.

Line *BC*: draw the total horizontal load (load due to wind or water).

Piles should be able to withstand these two loads.

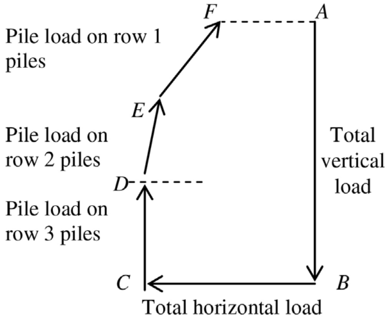
Line *CD*: draw the load on piles along row 3. These piles are vertical.

Line *DE*: draw the load on piles along row 2.

Line *EF*: draw the load on piles along row 1. These are batter piles.

Line *FA*: this line indicates the lateral force required to stabilize the piles. Lateral resistance of pile should be more than the load indicated by line *FA*.

Please refer to Fig. 20.1b for the following force polygon:



Line *AB*: calculate the total vertical load on piles. Draw the total vertical load.
Line *BC*: draw the total horizontal load.
Line *CD*: draw the load on piles along row 3. These piles are vertical.
Line *DE*: draw the load on piles along row 2. These piles are batter piles.
Line *EF*: draw the load on piles along row 1.
Line *FA*: this line indicates the lateral force required to stabilize the piles. Lateral resistance of the pile should be more than the load indicated by line *FA*.

Design example 20.1

Compute the loads on piles due to the retaining wall shown. Assume lateral earth pressure coefficient at rest (K_0) to be 0.5. Piles in the front are battered at 20 degrees and center piles are battered at 15 degrees to the vertical. Assume the piles have been placed at 3 ft. intervals (Fig. 20.3).

Horizontal stress at bottom of the wall; $K_0 \times 110 \times 20 = 1100$ psf
H, horizontal force = area of the stress triangle = $1{,}100 \times 20/2 = 11{,}000$ lb.

Step 1: horizontal force.
Horizontal force H, acts 6.666 ft. from the base ($y = 20/3$).
Moment (M) = $11{,}000 \times 6.666$ lb ft. = 73,333 lb ft. per linear ft. of wall.
Moment in 3 linear ft. section = $3 \times 73{,}333 = 219{,}999$ lb ft.
Step 2: weight of soil and concrete (Fig. 20.4).
Weight of soil resting on the retaining wall (height = 18, width = 5) = $5 \times 18 \times 110 = 9900$ lb.
Weight of concrete = $(2 \times 9 + 2 \times 18) \times 160 = 8640$ lb (160 pcf = concrete density).
Total weight = 18,540 lb per linear feet of wall.
Total weight (W) in a 3-ft. section = 55,620 lb.

There are three piles in the 3-ft. section. Hence, each pile carries a vertical load of 18,540 lb.

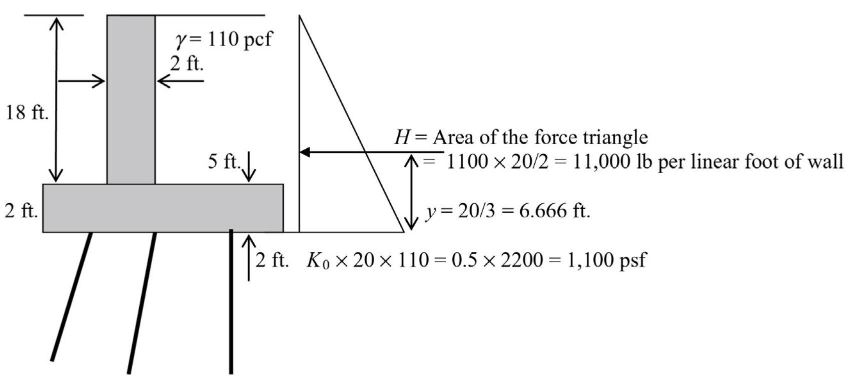


Figure 20.3 Design example.

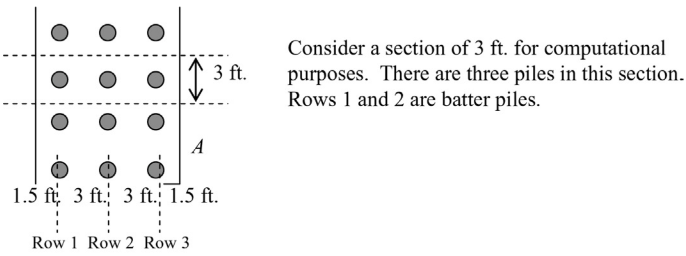


Figure 20.4 Horizontal force.

Step 3: overturning moment (Fig. 20.5).

Overturning moment = resisting moment

Take moments around point *"E"*

$R \times L = W \times L = M = H.\ y$ = 219,999 lb ft. (see step 1)

L = 219,999/W = 219,999/55,620 = 3.96 ft.

M = overturning moment due to H.

Step 4: stability (Fig. 20.6).

The center of gravity of the piles lies along the center of the middle row. Vertical reaction acts 3.96 ft. from the edge.

Distance to the reaction from the center of the pile system = 4.5 − 3.96 = 0.54 ft.

Moment around the center line = $R \times 0.54 = 55{,}620 \times 0.54$ = 30,038.4 lb per 3-ft. section.

Note that moment around the centerline is different from the moment around the edge (point "*E*").

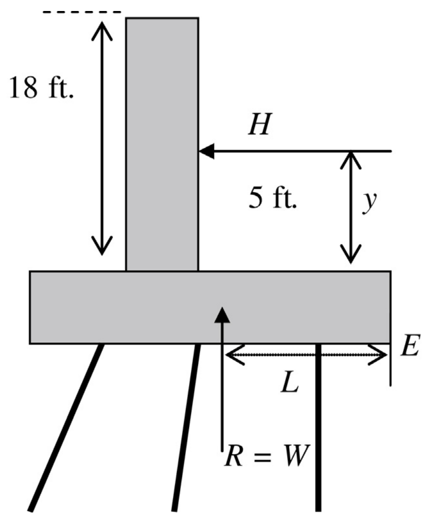


Figure 20.5 Weight of soil and concrete.

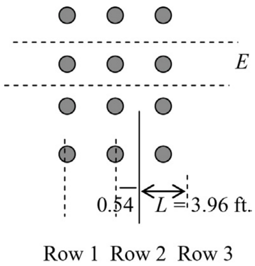


Figure 20.6 Overturning moment.

Step 5: additional load due to bending moments.

- Each pile carries a vertical load of 18,540 lb (see step 2).
- Due to the moment, the base of the retaining wall would undergo a bending moment. This bending moment would create additional stresses on piles. Stress developed on piles due to bending is given by the following equation:

$$\frac{M}{I} = \frac{\sigma}{y},$$

where M = bending moment, σ = bending stress, I = moment of area, and y = distance.
Find the moment of area of piles by taking moments around the centerline of the footing.
Moment of area of row 1 = $A \times 3^2 = 9A$ (A is area of the piles).
Moment of area of row 2 = $A \times 0 = 0$ (distance is taken from the center of gravity of piles).
Moment of area of row 3 = $A \times 3^2 = 9A$.
Total moment of area = 18 A.

$$\text{Bending load on piles in row 1 } (\sigma) = \frac{M}{I} \cdot y = \frac{30{,}034.8}{18\ A} \times 3 = \frac{5005.8}{A}\ \text{lb/ft}^2$$

M = 30,034.8 lb per 3-ft. section (see step 4)
y = 3 ft. (distance from center line to corner row of piles)

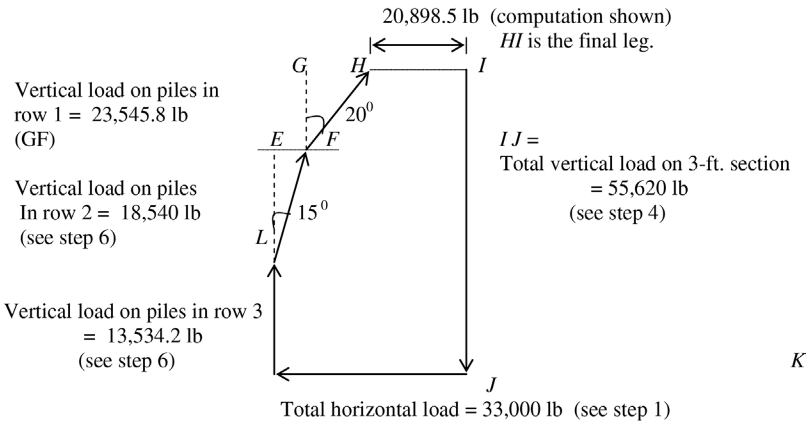
$$\text{Bending load per pile} = \frac{5005.8}{A} \times A = 5005.8\ \text{ lb per pile}$$

Step 6: total loads on piles.

Total vertical load on pile = vertical load + load due to bending.
Total vertical load on piles in row 1 = 18,540 + 5005.8 = 23,545.8 lb.
Bending load is positive for piles in row 1.
Total vertical load on piles in row 2 = 18,540 + 0 = 18,540 lb (since y = 0).
Load due to bending is zero for row 2.
Total vertical load on piles in row 3 = 18,540 − 5005.8 = 13,534.2 lb.
Bending load is negative for piles in row 3.

Step 7: force polygon.

Draw the force polygon for the 3-ft. section selected. Total horizontal load for a 3-ft. section is 33,000 lb (11,000 lb per linear foot. See step 1). Total vertical load for a 3-ft. section is 55,620 lb. See step 4. Start drawing the force polygon from *I* to *J*. *HI* is the final leg.



$EF = 18{,}540 \times \tan 15 = 4451$ lb (See Fig. 20.6 for EF)

$GH = 23{,}545.8 \tan 20 = 7{,}650.5$ lb.

Total horizontal load resisted = 4451 + 7650.5 = 12,101.5 lb.

Additional load that need to be resisted = 33,000 – 12,101.5 = 20,898.5 lb.

This load needs to be resisted by the soil pressure acting on the piles laterally (see Fig. 20.7).

The lateral pile resistance should not be confused with axial pile resistance. In the case of a vertical pile, axial load is vertical and there is no horizontal component to the axial load.

On the other hand, when a horizontal load is applied to a vertical pile, it would resist. Hence, all piles (including vertical piles) have a lateral resistance (Fig. 20.7).

Axial load "*P*" can be broken down into "*X*" and "*Y*" components. Computation of lateral resistance of the pile due to soil pressure is given in a further chapter (see chapter: Lateral Pile Resistance).

There are three piles in the considered 3-ft. section. Each pile should be able to resist a load of 6,966.5 lb. Use the principles given in chapter: Lateral Pile Resistance to compute the lateral pile capacity due to soil pressure.

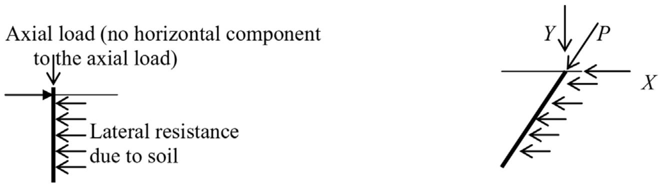


Figure 20.7 Lateral resistance.

Factor of safety (FOS) of 2.5–3.0 should be used. For instance, the required vertical pile capacity for piles in row 1 is 23,545 lb. The piles should have an ultimate pile capacity of 70,635 lb assuming a FOS of 3.0.

Note: no credit should be given to the resistance due to soil friction acting on the base of the retaining wall. Piles would stress the soil due to the horizontal load acting on them. Stressed soil underneath the retaining wall will not be able to provide any frictional resistance to the retaining wall.

If piles were to fail, piles would pull the soil with them. Hence, soil would not be able to provide any frictional resistance to the retaining structure.

Design example 20.2

Compute the loads on piles due to the retaining wall shown. Assume lateral earth pressure coefficient at rest (K_0) to be 0.5. Piles in the front are battered at 20 degrees and center piles are battered at 15 degrees to the vertical (this problem is similar to the previous example, except for the pile configuration, Fig. 20.8).

Step 1: horizontal force H acts 6.666 ft. from the base.

Moment (M) = 11,000 × 6.666 lb ft. = 73,333 lb ft. per linear foot of wall.

Moment in 6 linear ft. section = 6 × 73,333 = 439,998 lb ft.

Assume the piles have been placed at 3-ft. intervals (Fig. 20.9).

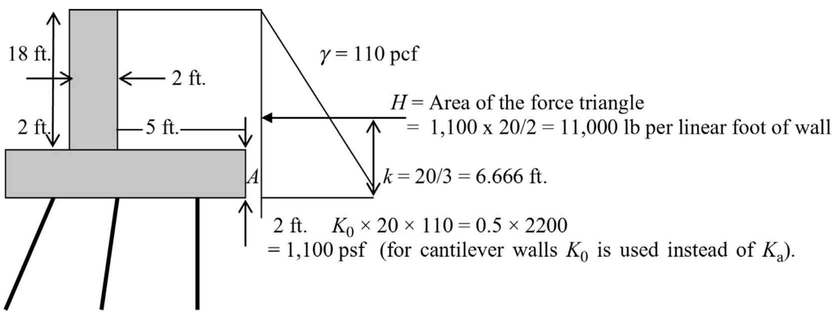


Figure 20.8 Compute pile loads.

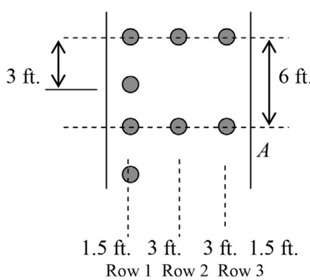


Figure 20.9 Assume piles are placed at 3 ft. intervals.

Consider a section of 6 ft. for computational purposes. There are six half piles and one full pile in this section. Rows 1 and 2 are batter piles. Row 1 has twice as more piles than rows 2 and 3.

Weight of soil resting on the retaining wall = $5 \times 18 \times 110 = 9900$ lb.

Weight of concrete = $(2 \times 9 + 2 \times 18) \times 160 = 8640$ lb (160 pcf = concrete density).

Total weight = 18,540 lb per linear foot of wall.

Total weight (W) in a 6-ft. section = 111,240 lb.

There are four piles in the 6-ft. section. Hence, each pile carries a load of 27,810 lb.

Take moments around point "E"

$R \times L = W \times L = M = H \cdot k = 439{,}998$ lb ft.

$L = 439{,}998/111{,}240 = 3.96$ ft.

20.1.4 Center of gravity of piles

There are six half piles and one full pile in this section (Fig. 20.10 and Fig. 20.11).

There are two half piles and one full pile in row 1.

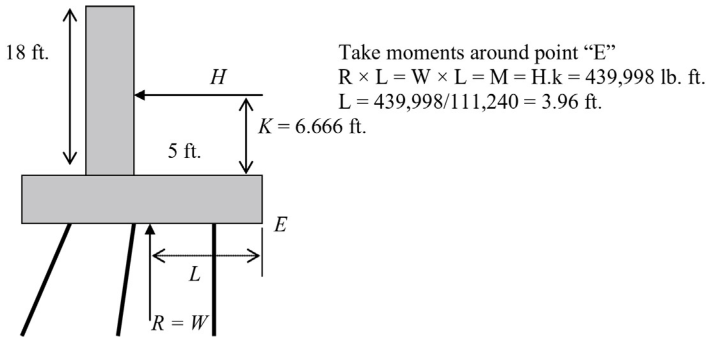


Figure 20.10 Taking moments around point "*E*".

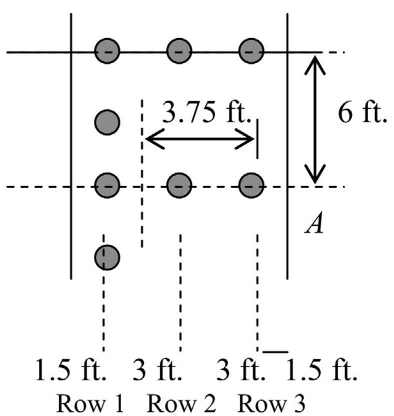


Figure 20.11 Center of gravity of piles.

There are two half piles in row 2 and two half piles in row 3 (total piles in the section = 4).

Take moments around row 3.

$(2 \times 6 + 1 \times 3)/4 = 3.75$ ft.

- As per these calculations, the center of gravity of piles is located 3.75 ft. from row 3.
- Moment of area of the system has to be computed from the center of gravity of piles.

Distance to row 1 from center of gravity = (6 − 3.75) ft. = 2.25 ft.
Distance to row 2 from center of gravity = 0.75 ft.
Distance to row 3 from center of gravity = 3.75 ft.

Step 2: Each pile carries a load of 27,810 lb (see step 1).

- Stress developed on piles

$$\frac{M}{I} = \frac{\sigma}{y}.$$

Here, M = bending moment, σ = bending stress, I = moment of area, and y = distance.

Find the moment of area of piles.

20.1.5 Row 1

Moment of area of row 1 = $A \cdot r^2 = A \times 2.25^2 = 10.12\,A$ (A = area of piles).

Moment of area of row 2 = $A \times 0.75^2 = 0.56\,A$.

(Distance is taken from the center of gravity).

Moment of area of row 3 = $A \times 3.75^2 = 14.06\,A$.

Total moment of area = 24.74 A.

Bending moment (M) = 439,998 ft. lb (see step 1).

$$\text{Bending load on piles in row } 1(\sigma) = \frac{M}{I} \cdot y = \frac{439{,}998}{24.74\ A} \times 2.25 = \frac{40{,}016}{A}\ \text{lb}.$$

$$\text{Bending load on first row piles} = \frac{40{,}016 \times 2A}{A} = 80{,}032\ \text{lb}.$$

There are two piles in the first row. Two half piles and one full pile is equivalent to two piles.

Total load on pile = vertical load + load due to bending.

Total load on piles in row 1 = 27,810 × 2 + 80,032 = 135,652 lb.

Vertical load per pile is 27,810 lb. Since there are two piles in row 1, 27,810 lb is multiplied by 2.

Step 3: row 2.

$$\text{Bending load on piles in row } 2(\sigma) = \frac{M}{I} \cdot y = \frac{439{,}998}{24.74\ A} \times 0.75 = \frac{13{,}339}{A}\ \text{lb}.$$

$$\text{Bending load per pile} = \frac{13{,}339}{A} \times A = 13{,}339\ \text{lb per pile}.$$

There are two half piles in row 2, which amounts to one pile.
Multiply by area of the pile to convert the load from lb per ft.^{2} to lb per pile.
Total load on piles in row 2 = 27,810 – 13,339 = 14,471 lb.
(Since center of gravity is on the opposite side, bending moment creates a tensile force on the pile.)

Step 4: row 3.

$$\text{Bending load on piles in row 3}(\sigma) = \frac{M}{I} \cdot y = \frac{439{,}998}{24.74\ A} \times 3.75 = \frac{66{,}693}{A}\ \text{lb.}$$

$$\text{Bending load per pile} = \frac{66{,}693}{A} \times A = 66{,}693\ \text{ lb per pile.}$$

Row 3 has two half piles, which amounts to one pile.
Total load on piles in row 3 = 27,810 − 66,693 = −38,883 lb per pile
The value 27,810 lb is the vertical load on piles and 66,693 lb is the load due to bending, which is tensile.
Total force on piles in row 3 is tensile.

Step 5: draw the force polygon.
NM = 14,471 × tan 15 = 3,877 lb.
GH = 135,652 × tan 20 = 49,373 lb (this load accounts for two piles in row 1).

Total horizontal load resisted by axial forces of piles = 49,373 + 3,877 = 53,250 lb.

Additional load that needs to be resisted = 66,000 – 53,250 = 12,750 lb.
This load needs to be resisted by the soil pressure acting on piles laterally.
Each pile should have a lateral pile capacity of 12,750/4 lb = 3,188 lb.

Pile design software

21

21.1 Software

There are many software available in the market. Some of these are the following:

- APile by Ensoft: This program calculates the axial capacity of piles
- Lpile by Ensoft: This program calculates the lateral load capacity of piles
- Pile by Oasis Inc.: Axial capacity is calculated

The logic and equations used by each program are available. Not all programs use the same equations or parameters.

21.1.1 General input of data

All computer programs need soil profile data. Soil layers and their properties need to be input into the program.

Typically, a box, as shown here needs to be filled.

Soil layers	Top elevation	Bottom elevation
1		
2		
3		

The top and bottom elevation of each soil layer is required.

Then, soil properties are inserted into another box.

Thereafter, typically, a box as shown next needs to be filled.

Soil layers	Friction angle	Cohesion	Unit weight
1			
2			
3			

Groundwater elevation needs to be provided.

21.1.2 Pile information

Piles	Top diameter	Tip diameter	Length of tapered portion
1			
2			
3			

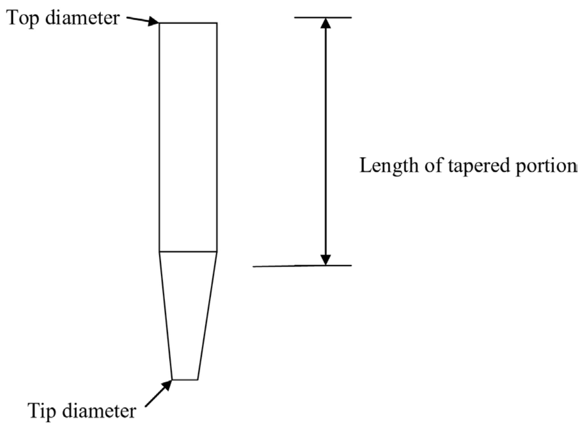


Figure 21.1 Pile information.

The length of the tapered portion is zero for nontapered piles (Fig. 21.1). Also, the top diameter and tip diameter is the same for nontapered piles.

Pile material, open-end pipe, close-end pipe PILE, H-pile, and so on, have to be treated as inputs.

The program will provide skin friction values and end bearing values.

21.1.3 Lateral pile capacity

Lpile by Ensoft is a widely used program to compute lateral pile capacity.

21.2 Pile design: finite element method

- The finite element method is considered to be the most powerful mathematical method that exists today to solve piling problems.
- Any type of soil condition could be simulated using finite element method.
- A complicated soil profile is shown in Fig. 21.2. Nodes in each of the finite element are given the soil properties of that layer such as ϕ, γ (density), cohesion, SPT (N) value, and so on.
- Nodes of elements in layer 1 are given the soil properties of layer 1. Similarly, nodes on layer 2 will be given the soil properties of layer 2.
- Due to this flexibility, isolated soil pockets also can be effectively represented.

21.2.1 Representation of time history

- The capacity of a pile is dependent on the history of loading. A pile that was loaded gradually would have a higher capacity than a pile that was loaded rapidly.
- Assume that a developer is planning to construct a 10-story building in 5 years. In this case, full building load on piles would gradually develop in a time period of 10 years.

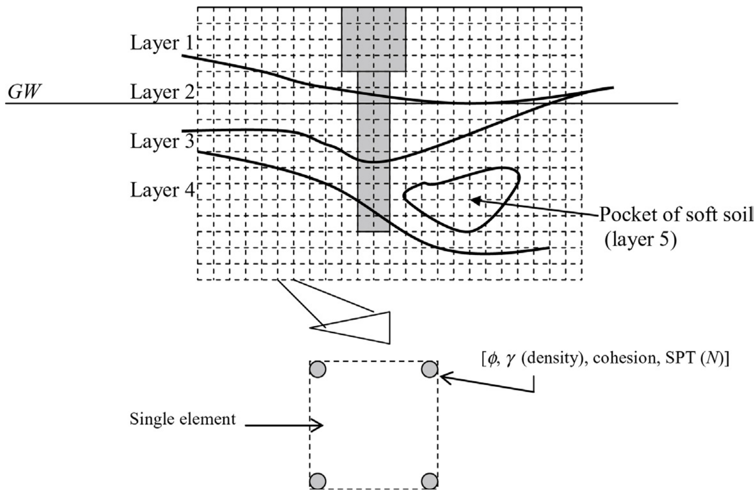


Figure 21.2 Finite element method.

- On the other hand, the developer would change his mind and decide to construct the 10-story building in 2 years. In this case, full load on piles would develop in 2 years. If the piles were to be fully loaded in 2 years the capacity of piles would be less than the first scenario.
- In such situations, finite element method could be used to simulate the time history of loading.

21.2.2 Groundwater changes

Change of groundwater conditions also affects the capacity of piles. Change of groundwater level can be simulated easily in finite element method.

21.2.3 Disadvantages

The main disadvantage of the finite element method is its complex nature. In many cases engineers may wonder whether it is profitable to perform a finite element analysis.

21.2.4 Finite element computer programs

Computer programs are available with finite element platforms. These programs can be used to solve a wide array of piling problems. The user is expected to have a working knowledge of finite element analysis to use these programs. More specialized computer programs are also available in the market. These programs do not require the knowledge of finite element analysis.

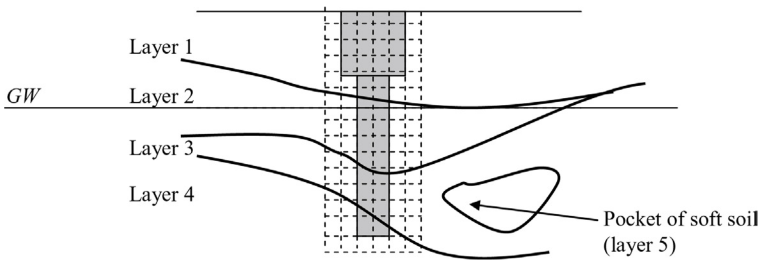


Figure 21.3 Boundary element method.

21.2.5 Boundary element method

- The boundary element method is a simplified version of the finite element method. In this method, only the elements at boundaries are considered.
- Only the elements at soil pile boundary are represented (Fig. 21.3).
- In this method, full soil profile is not represented. As one can see, the isolated soft soil pocket is not represented.
- On the other hand, fewer elements would make the computational procedure much simpler than the finite element method.

Pile driving methods

22

This chapter is dedicated to pile driving methods, pile driving rigs, and equipment.

22.1 Early history of pile driving

Driving a stick into the ground can be done by one person with a hammer.

- In earlier times, piles were driven by dropping a weight on the pile.
- The weight was lifted using various lever and pulley mechanisms.
- Guiding systems were developed to guide the weight so that it would fall vertically.

In the simple pile driving mechanism shown in Fig. 22.1, the following components are used:

1. a weight lifting mechanism (in this case a pulley)
2. guide rails to make sure that the hammer falls vertically

Early engineers who used this simple mechanism found out that it is more productive to use a smaller fall height. If the fall height were increased, then time taken for a blow also would increase. It was more productive to have many low-energy blows than few high-energy blows. It was noted that damage to piles also could be minimized with low-energy blows.

22.2 Steam-operated pile hammers

22.2.1 Single-acting steam hammers

Steam is allowed to enter the pile hammer chamber from the bottom. The hammer is lifted due to steam pressure. After lifting the hammer, the steam inflow is cutoff. The hammer would then drop onto the pile (Fig. 22.2).

22.2.2 Double-acting steam hammers

Steam is allowed to enter the pile hammer chamber from the bottom. The hammer is lifted due to steam pressure. After lifting the hammer, the steam inflow is cutoff. When the hammer starts to drop, a second stream of steam is sent to the chamber from the top. The hammer will fall faster due to gravity and the steam pressure above.

- Note: today compressed air is used instead of steam.

Pile Design and Construction Rules of Thumb

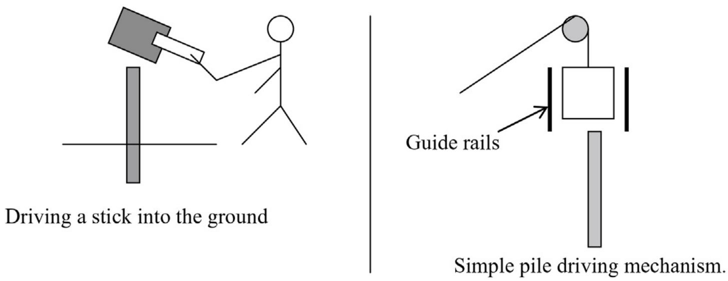


Figure 22.1 Driving a stick into the ground.

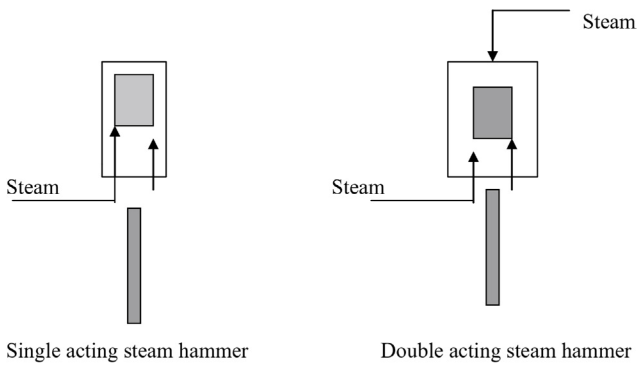


Figure 22.2 Steam-operated pile hammers.

22.3 Diesel hammers

This section talks about diesel hammers in general.

22.3.1 Single-acting diesel hammers

- Fig. 22.3 shows a single-acting diesel hammer. Double-acting hammers would have two combustion chambers (Fig. 22.4).
- Single-acting hammers have open-end tops while double acting hammers have close-end tops. Diesel would be injected to the lower and upper chambers in double-acting hammers.
- In the case of single acting diesel hammers, the hammer moves downward only due to the gravitational force.
- In the case of double-acting diesel hammers, the hammer moves downward due to gravitational force and the explosion force in the *upper* combustion chamber.
- Double-acting hammers are capable of generating much more force than single-acting hammers.

22.3.2 Double-acting diesel hammers

- Noise: another major complaint against diesel hammers is the “noise.” There are special diesel models available from major manufacturers with less noise.
- Operation in cold weather: single- and double-acting diesel hammers are known to operate well under cold weather conditions.

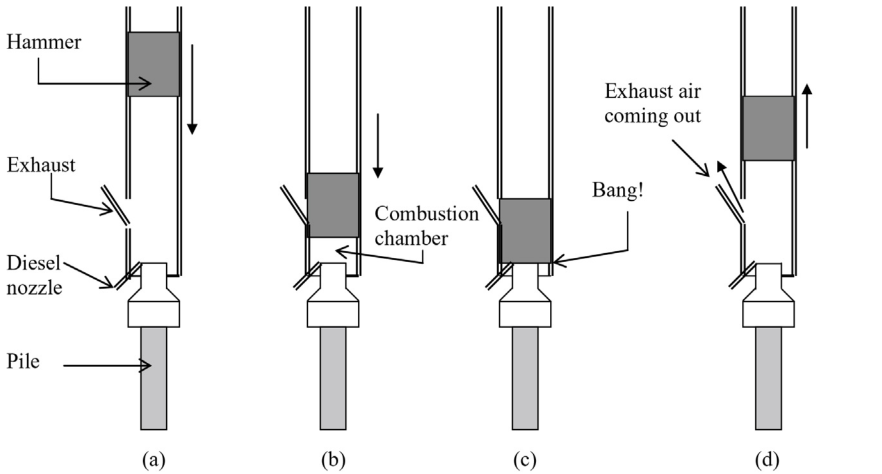


Figure 22.3 Diesel hammer. (a) The hammer is raised and ready to fall. (b) The hammer is dropping. Combustion chamber is filled with compressed air. At this point diesel is injected to the combustion chamber through the nozzle. (c) "*Bang!*" Impact. When the impact occurs, diesel in the combustion chamber would ignite and an explosion would occur. (d) The explosion inside the combustion chamber would raise the hammer. Exhaust air would come out of the exhaust air outlet. The cycle would repeat.

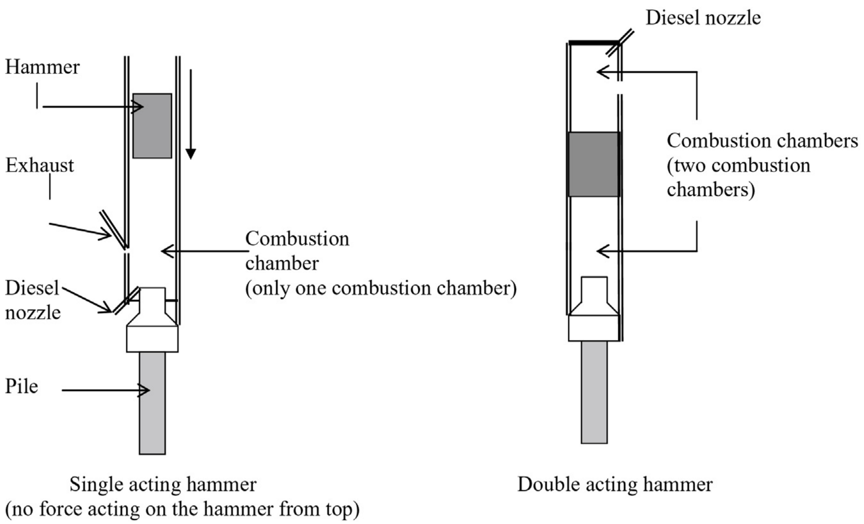


Figure 22.4 Double-acting diesel hammer.

- Hammer as a pile extractor: single-acting hammers cannot be used as pile extractors. Double-acting hammers are inverted and can be used as pile extractors.
- Frequency: single-acting hammers have a frequency from 50–60 blows per minute while double-acting hammers could be as high as 80 blows per minute.
- Soft soil conditions: diesel hammers could stall when driving in soft soil conditions.

- Air pollution: it is not a secret that diesel hammers create diesel exhaust after each stroke. Due to this reason, many engineers are reluctant to specify diesel hammers for urban pile driving work. The latest models have much better track record for cleanliness. This aspect needs to be investigated prior to specifying a diesel hammer.
- Energy of diesel hammers could be as high as 500,000 ft. lb. Large diesel hammer D200-42 by Delmag imparts an energy of 500,000 ft. lb to the pile at maximum rating. It has a hammer of 44,000 lb (22 tons) and a stroke of 11 ft. Furthermore, it could provide 30–50 blows per minute as well.
- On the smaller side of the scale, D2 by Delmag has a hammer of 484 lb with a stroke of 3′ 8″ providing an energy of 1000 ft. lb with 60–70 blows per minute.

22.3.3 Environment-friendly diesel hammers

As an answer to the air pollution problem, new generation of diesel hammers are manufactured that use "biodiesel." Biodiesel is made of soybean oil. Biodiesel is non-toxic and biodegradable.

Biodiesel hammers can be used for projects in urban areas where exhaust gases are regulated.

Example: biodiesel hammer by ICE (www.iceusa.com)

22.3.4 Diesel hammer manufacturers

Delmag (www.delmag.de), Berminghammer (www.Berminghammer.com), APE (www.apevibro.com), MKT (www.mktpileman.com), ICE (www.iceusa.com), HMC (www.hmc-us.com), Mitsubishi, Kobe.

22.4 Hydraulic hammers

- Instead of air or steam, hydraulic fluid is used to move the hammer.
- Hydraulic hammers are much cleaner since they do not emit exhaust gases.
- Hydraulic hammers are less noisy than most other impact hammers.
- Unfortunately hydraulic hammers are expensive to buy and rent.
- Hydraulic hammers are capable of working underwater as well.
- Unlike steam or air hammers, the energy of hydraulic hammers can be controlled.

22.4.1 Mechanism

Step 1: valve "A" is opened and valve "B" is closed. When valve "A" is opened, high-pressure fluid would come from chamber "H" to chamber "G" and lift the piston and the hammer.

Step 2: after the piston is raised, valve "A" is closed and valve "B" is opened. Then high-pressure fluid inside fluid chamber "G" would go into the low-pressure chamber "L". Piston and the hammer would start to fall. Free fall is intensified by the high-pressure compressed air inside the chamber on top. The process would repeat.

- Usually, any free falling object would fall under gravitational acceleration "g." But due to high-pressure compressed air on the top chamber, acceleration of the hammer could be raised to $2g$ or more (Fig. 22.5).

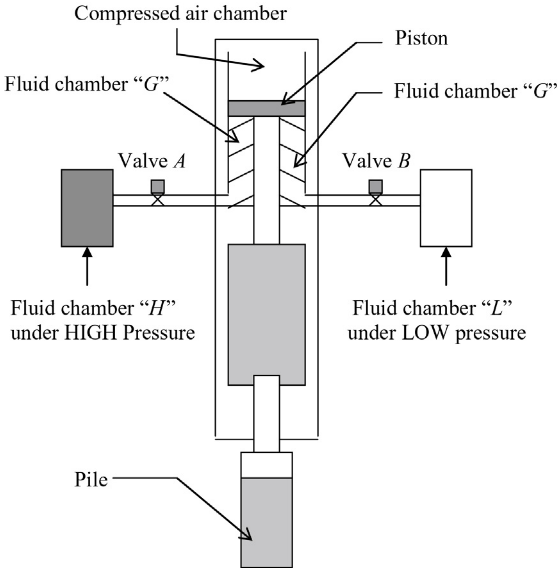


Figure 22.5 Mechanism of hydraulic hammer.

- Large hydraulic hammers could impart energy as high as 2.5 million ft. lb on the pile.
- For example, the MHU 3000T by MENCK can impart energy of 2.43 million ft. lb. It has a striking part weighing 198.4 tons with a stroke of 5.5 ft.
  Energy = weight of striking part × stroke = 396,800 lb × 5.5 ft. = 2.28 million ft. lb.
  This giant is also capable of providing 40 blows per minute. Additional energy is obtained from the high-pressure gas inside the chamber above the piston.
- The S-2300 hammer by IHC has an energy rating of 1.6 million ft. lb with a hammer weighing 115 tons. It could provide 30−80 blows per minute as well.
  Stroke of S-2300 hammer (assuming free fall) = 1,600,000/(115 × 2,000) = 6.95 ft.
  Stroke = energy/eight of the striking part of the hammer
  But the stroke of the S-2300 hammer is only 3.5 ft. Higher energy is obtained from the high-pressure gas inside the chamber above the piston.
- IHC provides two hammer types. The "S" series by IHC has a lighter hammer and high hammer speed. These hammers are ideal for steel piles. IHC "SC" series hammers have heavier hammers and slow hammer speed. These are good for concrete piles.
- On the smaller side of the scale the HMC 28H has an energy rating of 21,000 ft. lb with a hammer of 16,500 lb with a stroke of 1 ft.
- Energy can be varied: energy imparted to the pile can be controlled by varying the pressure in the gas chamber above the piston. The control panel can be hooked to a printer to obtain printouts.
- Control panel: electronic control panels are available with the latest models. A typical control panel would provide energy, engine rpm, and so on.
- Noise reduction: special housings have been developed for the purpose of noise reduction.
- Energy per blow: the operator can adjust the energy per blow.
- Blow rate: hydraulic hammers are capable of providing high blow rates (50−200 blows per minute).

A typical pile driving inspection report is shown in Table 22.1

Table 22.1 Pile driving inspection report

Pile information	The equipment	The hammer
• Pile type: wood__ H pile__ concrete__ pipe pile__ • Pile diameter: (wood) • Tip_______butt________ H pile: section_________ • Weight per foot________ • Pipe pile: Outer diameter_______ Inner diameter_______	Rig type: _____________ Mandrel:_______________ Follower_______________ Cap: type_____weight:______	Make:____________ Model:___________ Rated Energy:______ Wt of Ram:_______ Stroke:__________ Hammer cushion: thickness_______

Project Name:_______________ Address_________________ Project #_______________
Ground elevation____________ Pile toe elevation___________ Length driven__________
Pile cutoff elevation__________ Pile splicing information__________

Contractor information:
*Contractor's name:*___________________ *Field foreman's name:*___________________
*Contractor's address:*____

*Pile inspector's name*________________ *Pile inspection company:*__________________
Weather: temp:________ rain________

Feet	No. of blows	Speed (blows per minute)	Remarks	Feet	No. of blows	Speed (blows per minute)	Remarks
1					26		
2					27		
3					28		
4					29		
–					–		
–					–		
–					–		
25					50		

22.4.2 Pile driving inspection report

Please see Table 22.1 for a better understanding of the pile driving inspection report.

22.5 Vibratory hammers

Vibratory hammers use vibration as the method of penetration. Vibratory hammers are increasingly becoming popular (Plates 22.1 and 22.2).

Vibratory hammers consist of three main components:

1. gear case
2. vibration suppressor
3. clamp

Plate 22.1 Vibratory hammer.

Plate 22.2 Vibratory hammer attached to a pile.

22.5.1 Principle of the vibratory hammer

- The gear case has eccentric rotating weights.
- Due to the eccentricity of the weight, the gear case moves up and down (or vibrates up and down).
- These vibrations are transferred to the pile below.
- Suppressor above the gear case would suppress any up and down vibrations. The suppressor is made up of springs.
- Due to the suppressor, holding unit will not feel any vibrations.
- Holding unit could be attached to an arm of a backhoe, hung by a crane or by a helicopter in rare occasions.
- Up and down movement of the gear case is dependant upon the RPM (revolutions per minute) of weights.
- Imagine a weight attached to a rod. If the weight is attached to the rod at the center of gravity, there would not be any vibrations during rotation (Fig. 22.6).
- On the other hand, if the weight is attached from a point other than the center of gravity, then vibrations would occur (Fig. 22.7).
- In the case shown in Fig. 22.7a, the rod will not move up and down when the mass is rotated. In this case, the rotating mass is attached to the rod at the center of gravity.
- In the case shown in Fig. 22.7b, the rod will *move up and down* when the mass is rotated. In this case, the rotating mass is *not* attached to the rod at the center of gravity.
- This principle is used for vibratory hammers. The gear case contains a rotating mass attached to a rod as shown in Fig. 22.7b with an eccentricity.
- Eccentricity = distance between point of attachment and center of gravity (Fig. 22.8).
- Amplitude: up and down movement of the vibratory hammer. Amplitude could be from 0.2 in. to 1.5 in.
- Frequency: number of up and down movements of the vibratory hammer per minute. Typically frequency ranges from 1000 rpm to 2000 rpm (revolutions of rotating weights per minute).
- Power pack: vibratory hammers are usually powered by a power pack (Table 22.2).

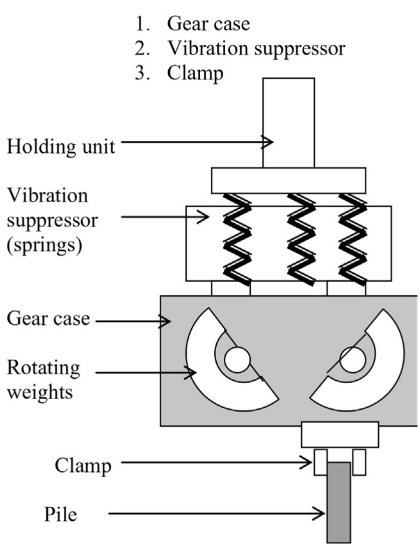


Figure 22.6 Mechanism of vibratory hammer.

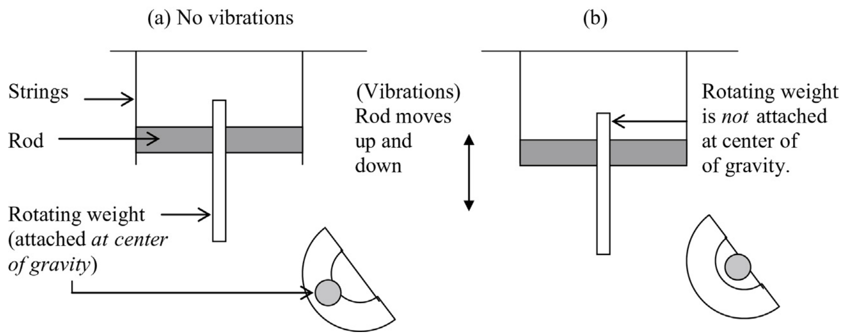


Figure 22.7 Moving parts of vibratory hammer.

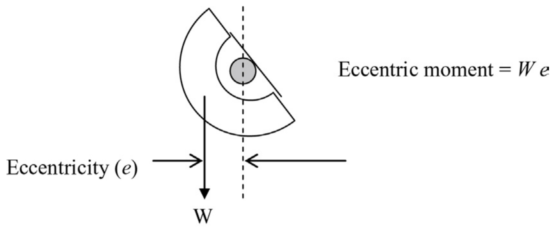


Figure 22.8 Eccentricity.

Table 22.2 Some vibratory hammer specifications

Manufacturers	Models	Frequency (rpm)	Amplitude (in)	Maximum pull (tons)	Pile clamp force (tons)	Eccentric moment (in. lb)
Dawson	EMV 300	2400	0.58	8.8	40	400
Tramac	428B	3000	0.35	14	56	348
HPSI	40E	2200	0.875	15	30	400
ICE	216E	1600	1.02	45	50	1100

22.5.1.1 Some vibratory hammer specifications

- Electronic monitoring: most vibratory hammers come with electronic monitoring devices. These devices would provide information such as frequency, amplitude, maximum pull, and eccentric moment, at any given moment.
- Sands and clays: vibratory hammers are ideal for sandy soils. When the pile vibrates, the soil particles immediately next to the tip of the pile would liquefy and resistance to driving would diminish. In clay soils, this process does not occur. Usually, vibratory pile hammers are not effective for driving in clay soils. If one were to encounter clay soil during driving, larger amplitude should be used for clay soils. This way, clay soil can be moved and the pile can progress. When a smaller amplitude is selected, pile and surrounding clay can move up and down together without any progress.

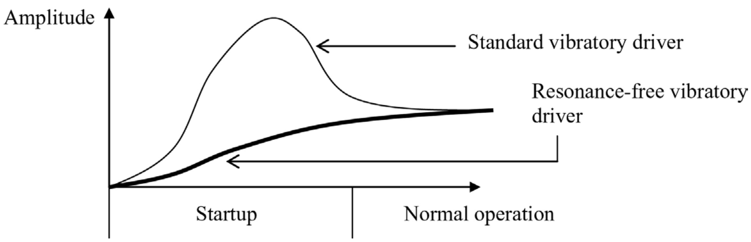


Figure 22.9 Change of amplitude.

- Popular dynamic formulas cannot be used: one major disadvantage of vibratory hammers is the inability to use popular dynamic formulas (many engineers and city codes are used to specifying the end of pile driving using a certain number of blows per foot).

22.5.2 Resonance-free vibratory pile drivers

- The highest amplitude occurs at the startup and finish of a vibratory pile driver. At the startup, weights inside the vibratory driver accelerate to achieve a high velocity. This induces high amplitude at the beginning. High amplitude could cause damage to nearby buildings.
- At startup standard vibratory drivers generate high amplitudes as shown in Fig. 22.9. Specially designed resonance-free vibratory drivers do not generate high amplitudes during startup.
- Resonance-free vibratory drivers may be selected when driving near buildings.

22.6 Pile driving procedure

The following procedure is generally followed during driving of piles.

- A stake is driven at the location of the pile.
- The pile is straightened and kept upright on the location marked.
- The plumbness of the pile is checked.
- The pile-driving hammer is lowered to the top of the pile.
- A few light blows are given and the pile is checked for plumbness.
- Full driving starts.
- Rate of penetration is recorded.
- Rate of penetration = number of blows per inch
- The pile is driven to the planned depth. In some cases pile driving is stopped when the required rate of penetration is achieved.
- The inspector should keep an eye on the rate of penetration of the pile. Unusually high blows per inch may indicate boulders or bedrock. Some piles (especially timber piles) would get damaged if they were to hit a boulder.
- After the pile is driven as per the required criteria, the pile is cutoff from the top.

Pile lead is a steel truss that holds the pile (Fig. 22.10). The pile hammer is hung from a special boom in the truck. Companies have come up with various ingenious designs to connect the hammer, lead, and the pile (Plates 22.3–22.5).

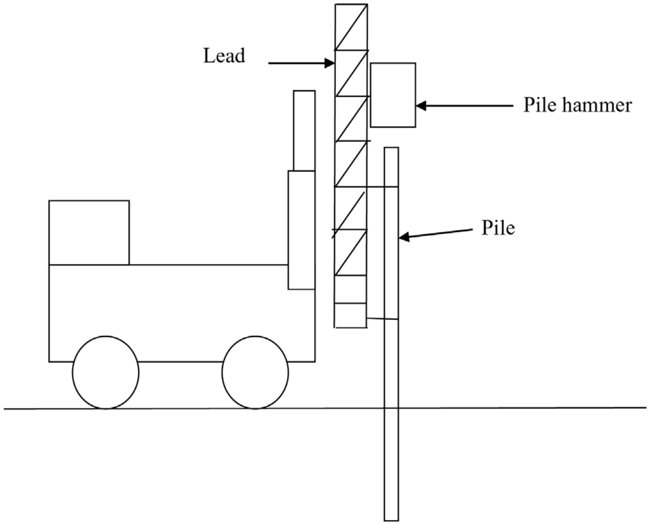


Figure 22.10 Pile rig configuration.

Plate 22.3 Pile lead and a pile shown here. Pile is on the right-hand side of the photo.

Plate 22.4 Pile hammer.

Plate 22.5 Piles cutoff to proper height.

22.7 Pile selection guide

Selection of piles in most cases is done using wave equation programs. If wave equation analysis is not conducted, the following table can be used as an approximate guide (Tables 22.3 and 22.4 were prepared by adapting the table presented in a further chapter (see chapter: Pile Driving Equipment, US Army Corps of Engineers, July 1997).

22.7.1 Sandy soils

All about piling in sandy soils has been presented in Table 22.3.

22.7.2 Clay soils

For more details on piling in clay soils, please refer Table 22.4.

22.8 General guidelines for selecting a pile hammer

22.8.1 Single-acting steam and air hammers

- Dense sands and stiff clays need heavy hammers with low blow counts. This makes single-acting hammers ideal for such situations.

22.8.2 Double-acting steam and air hammers

- Double-acting hammers have light hammers compared to single-acting hammers with the same energy level. Light hammers with high velocity blows are ideal for medium dense sands and soft clays.

22.8.3 Vibratory hammers

- Avoid vibratory hammers for concrete and timber piles. Vibratory hammers could create cracks in concrete.
- Avoid vibratory hammers for clayey soils. Vibratory hammers are best suited for loose to medium sands.
- Vibratory hammers are widely used for sheet piles since it may be necessary to extract and reinstall piles. Extraction of piles can be readily done with vibratory hammers.
- In loose to medium soil conditions, sheet piles can be installed at a much faster rate by vibratory hammers.

22.8.4 Hydraulic hammers

- Hydraulic hammers provide an environmental-friendly operation. Unfortunately, rental cost is high for these hammers.

Table 22.3 **Sandy soils**

SPT (*N*) values	Soil types	Timber piles	Open end pipe piles	Closed end pipe piles	H-Piles	Sheet piles	Concrete piles
0–3	Very loose	DA, SA (A, S, H)	DA, SA, V (A, S, H)	DA, SA, V (A, S, H)	DA, SA, V (A, S, H)	DA, SA, V (A, S, H)	DA, SA (A, S, H)
4–10	Loose	DA. SA (A, S, H)	DA, SA, V (A, S, H)	DA, SA, V (A, S, H)	DA, SA, V (A, S, H)	DA, SA, V (A, S, H)	DA, SA (A, S, H)
10–30	Medium	SA (A, S, H)	DA, SA, V (A, S, H)	DA, SA, V (A, S, H)	DA, SA, V (A, S, H)	DA, SA, V (A, S, H)	SA (A, S, H)
30–50	Dense	SA (A, S, H)	DA, SA, V (A, S, H)	SA, V (A, S, H)	SA, V (A, S, H)	DA, SA, V (A, S, H)	SA (A, S, H)
50 +	Very dense	SA (A, S, H)	SA (A, S, H)	SA (A, S, H)	SA (A, S, H)	DA, SA, V (A, S, H)	SA (A, S, H)

DA, double acting; SA, single acting; A, air; S, steam; H, hydraulic; V, vibratory.

Table 22.4 **Clay soils**

SPT (*N*) values	Soil types	Timber piles	Open-end pipe piles	Close- end pipe piles	H-Piles	Sheet piles	Concrete piles
0–4	Very soft	DA, SA (A, S, H)	DA, SA, V (A, S, H)	DA, SA (A, S, H)	DA, SA, V (A, S, H)	DA, SA,V (A, S, H)	DA, SA (A, S, H)
4–8	Medium	DA, SA (A, S, H)	DA, SA, V (A, S, H)	SA (A, S, H)	DA, V (A, S, H)	DA, SA, V (A, S, H)	SA (A, S, H)
8–15	Stiff	SA (A, S, H)	DA, SA (A, S, H)	SA (A, S, H)	DA, SA (A, S, H)	DA, SA (A, S, H)	SA (A, S, H)
15–30	Very stiff	SA (A, S, H)	SA (A, S, H)	SA (A, S, H)	SA (A, S, H)	SA (A, S, H)	SA (A, S, H)
30+	Hard	SA (A, S, H)	SA (A, S, H)	SA (A, S, H)	SA (A, S, H)	SA (A, S, H)	SA (A, S, H)

Table 22.5 Noise level (White et al., 1999)

Installation methods	Observed noise levels (dB)	Distance of observation (m)
Pressing (jacking)	61	7
Vibratory (medium frequency)	90	1
Drop hammer	98–107	7
Light diesel hammer	97	18

22.8.5 Noise level

Another important aspect of selection of piles is the noise level. Table 22.5 is provided by White et al. (1999).

22.9 ASTM standards

22.9.1 Timber piles

22.9.1.1 ASTM

- ASTM D25: specification for round timber piles
- ASTM D1760: pressure treatment of timber products
- ASTM D2899: establishing design stresses for round timber piles

22.9.1.2 AWPA standards for timber piles: (American Wood Preservers Association, Woodstock, MD)

- C 3: pile preservative treatment by pressure processes
- C 18: pressure-treated material in marine environment
- M 4: standard for the care of preservative-treated wood products

22.9.2 Steel H-piles

ASTM A690: high-strength low-alloy steel H-piles and sheet piles for use in marine environments.

22.9.3 Steel pipe piles

A 252: specification for welded and seamless steel pipe piles.

22.9.4 Pile testing

- ASTM D1143: method of testing of piles under static axial compressive loads
- ASTM D3689: method of testing piles under static axial tensile loads
- ASTM D3966: testing piles under lateral loads
- ASTM D4945: high-strain dynamic testing of piles

22.10 ACI (American Concrete Institute) standards for general concreting

- ACI 304: recommended practice for measuring, mixing, transportation and placing concrete
- ACI 305: recommended practice for hot weather concreting
- ACI 306: recommended practice for cold weather concreting
- ACI 308: recommended practice for curing of concrete
- ACI 309: recommended practice for consolidation of concrete
- ACI 315: recommended practice for detailing of reinforced concrete structures
- ACI 318: requirements for structural concrete
- ACI 347: recommended practice for concrete formwork
- ACI 403: recommended practice for use of Epoxy compounds with concrete
- ACI 517: recommended practice for atmospheric pressure steam curing of concrete

22.11 Design stresses and driving stresses

Permissible design and driving stresses for various piles are given in Table 22.6 Pile Driving Equipment, US Army Corps of Engineers, July 1997.

22.12 Vibratory hammers: design of piles

Though many piles have been driven in clay soils using vibratory hammers, vibratory pile hammers are best suited for sandy soils. Vibratory hammers are less noisy and do not cause pile damage as compared to pile driving hammers.

One of the major problems of vibratory hammers is the unavailability of credible methods to compute the bearing capacity of piles based on penetration rates. On the other hand, pile-driving formulas can be used to compute ultimate bearing capacity of piles driven with drop hammers.

Vibratory pile driving depends on

- Pile and soil characteristics
- Elastic modulus of pile and soil (E_p and E_s)
- Lateral earth pressure coefficient (K_0)
- Relative density of soil (D_r)
- Vibratory hammer properties

22.12.1 Vibratory hammer properties

The effectiveness of a vibratory hammer depends on the following:

- Frequency of the vibratory hammer (frequency = number of vibrations per second).
- Amplitude of the hammer (distance traveled during up and down motion of the vibratory hammer).
- Vibrating weight of the hammer.

Table 22.6 **Pile types, driving stresses, and design stresses**

Pile types	Authoritative codes	Design stress (permissible)	Driving stress (permissible)
Timber piles			
Douglas Fir	ASTM D 25	1.2 ksi	3.6 ksi
Red Oak	ASTM D 25	1.1 ksi	3.3 ksi
Eastern Hamlock	ASTM D 25	0.8 ksi	2.4 ksi
Southern pine	ASTM D 25	1.2 ksi	3.6 ksi
Reinforced concrete Piles (Non–prestressed)	ACI 318 (concrete) ASTM A615 (reinforced steel)	$0.33\,f'_c$ (compression) f'_c = strength at 28 days (use gross cross-sectional area for compression and net concrete area for tension)	$0.85\,f'_c$ (compression) $3\,(f'_c)^{1/2}$ (tension)
Prestressed concrete	ACI 318 (concrete)	$0.33\,f'_c$ (compression) f'_c = Strength at 28 days	For compression: ($0.85\,f'_c$ − effective prestress)
Minimum effective prestress = 0.7 ksi Minimum 28 day concrete strength (f'_c) = 5.0 ksi	ASTM A615 (reinforced steel)	Use gross cross-sectional area for compression and net concrete area for tension.	For tension: $3(f'_c)^{1/2}$ + effective prestress
Steel pipe piles	ASTM A252 for steel pipe ACI 318 for concrete filling	9 ksi for steel	$0.9\,f_y$ f_y = yield stress
Concrete cast-in-shell driven with mandrel	ACI 318 for concrete	$0.33\,f'_c$	Not applicable
Concrete cast-in-shell driven without mandrel	ASTM A36 for core ASTM A252 for pipe ACI 318 for concrete		$0.9\,f_y$ for steel shell (f_y = Yield stress)
Steel HP section piles	ASTM A36	9 ksi	$0.9\,f_y$

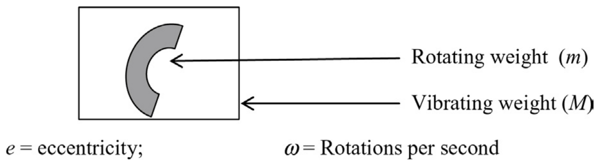


Figure 22.11 Properties of vibratory hammer.

- Surcharge weight of the hammer (in addition to the vibrating weight, a nonvibrating weight is also attached to the hammer to produce a downward thrust. Bias weight is another name for the surcharge weight).

22.12.2 Definitions

- Eccentric moment = rotating weight (m) × eccentricity (e)
- Centrifugal force = $C \cdot m \cdot e \cdot \omega^2$

m = eccentric weight or the rotating weight. This is different than the vibrating weight (M). Vibrations are caused by the rotating weight. The rotating weight is attached to a housing, and the whole unit is allowed to vibrate due to the rotating weight.

Vibrating weight = rotating weight + bias weight + weight of the housing.

C = constant depending on the vibratory hammer (Fig. 22.11).

- Eccentric moment = $m \cdot e$
- A (amplitude) = distance travel during up and down motion of the vibrating weight.

Some parameters of vibratory pile hammers are given further.

22.12.3 Vibratory pile hammer data (ICE: Model 416L)

Eccentric moment ($m \cdot e$) = 2200 lb in = 2534 kg · cm

Frequency (ω) = 1600 vpm (vibrations per minute)

Centrifugal force = 80 tons = 712 kN

The pile hammer manufacturer ICE provides the following equation to compute the centrifugal force for their hammers.

22.12.3.1 Units (US tons)

Centrifugal force = 0.00005112 × ($m \cdot e \cdot \omega^2$) tons, "m" should be in pounds (lb), "e" should be in "inches," and "ω" should be in vibrations per second.

22.12.3.2 Units (kN)

Centrifugal force = 0.04032 × ($m \cdot e \cdot \omega^2$) kN, "m" should be in kilograms (kg), "e" should be in "meters," and "ω" should be in vibrations per second.

Design example

Find the centrifugal force of a vibratory pile hammer, which has an eccentric weight (m) of 1000 kg, an eccentricity (e) of 2 cm, and a vibrating frequency (ω) of 30 vibrations per second.

Solution

$$\begin{aligned}\text{Centrifugal force} &= 0.04032\times(m\cdot e\cdot\omega^2)\ \text{kN}\\ &= 0.04032\times(1000\times0.02\times900)\ \text{kN} = 725\ \text{kN}\end{aligned}$$

Ultimate pile capacity

Ultimate capacity of a pile driven using a vibrating hammer can be obtained by using the equation proposed by Feng and Deschamp (2000):

$$Q_u = \frac{3.6(Fc+11W_b)}{[1+V_p(OCR)^{1/2}]}\frac{L_e}{L}$$

Q_u, ultimate pile capacity (kN), F_c = centrifugal force (kN); W_b, surcharge weight or bias weight (kN); V_p, penetration velocity (m/min); OCR, overconsolidation ratio; L_e, embedded length of the pile (m); and L, total length of the pile (m).

Design example

A vibratory hammer has the following properties:

Surcharge weight of the hammer = 8 kN; centrifugal force = 600 kN

Find the penetration rate (V_p) required to obtain an ultimate pile capacity of 800 kN. Assume the pile to be fully embedded and OCR of the soil to be 1.0.

Solution

$$Q_u = \frac{3.6(F_c+11W_b)}{[1+V_p(OCR)^{1/2}]}\frac{L_e}{L}$$

$$800 = \frac{3.6\,(600+11\times8)}{[1+V_p\,(1)^{1/2}]}\frac{L_e}{L}$$

$L_e = L$, since the pile is fully embedded.

$V_p = 2.096$ m/min

The pile should penetrate 2.096 m/min or less to achieve an ultimate pile capacity of 800 kN.

22.12.4 Penetration rate and other parameters

See Fig. 22.12 for more details on penetration rate and other parameters.

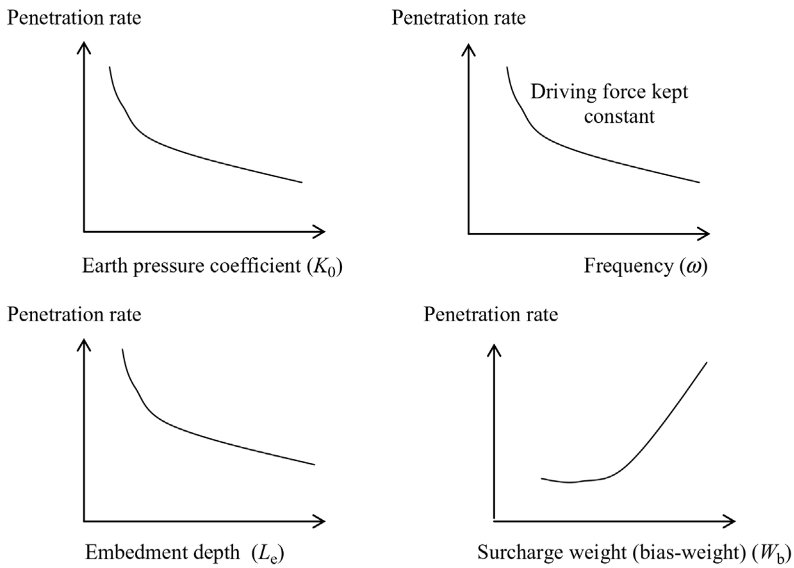


Figure 22.12 Penetration rate and other parameters.

22.12.5 Vibratory hammers (International Building Code)

The International Building Code (IBC) states that vibratory drivers shall be used only when pile load capacity is verified by pile load tests.

22.13 Pile driving through obstructions

- Most pile driving projects encounter obstructions. The types of obstructions usually encountered are the following:
  1. boulders fill material (concrete, wood, debris)

22.13.1 Obstructions occurring at shallow depths

If the obstruction to pile driving is occurring at a shallow depth, a number of alternatives are available.

22.13.1.1 Excavate obstructions using a backhoe

This method may be cost prohibitive for large piling projects. The reach of most backhoes are limited to 10–15 ft. and any obstruction below that level would remain (Fig. 22.13).

22.13.1.2 Use specially manufactured piles

Most pile driving contractors have specially designed thick-walled pipe piles or H-piles that would penetrate most obstructions. Usually, these piles are driven to

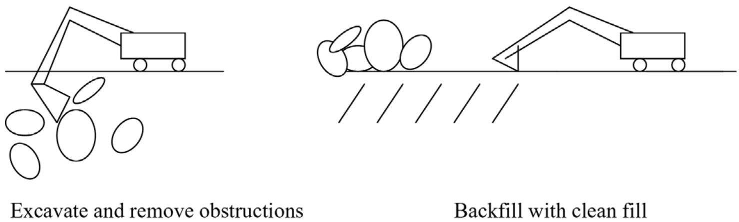


Figure 22.13 Backhoe extravations.

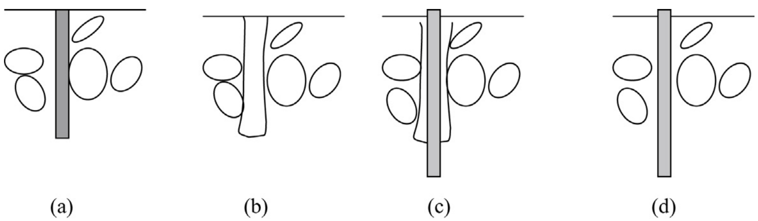


Figure 22.14 Pile driving through obstructions. (a) Drive the stronger pile (special pile) through obstructions. (b) Remove the special pile (the hole is shown here). (c) Insert the permanent pile and drive it to the desired depth. and (d) grout the annulus or fill it with sand.

penetrate obstructions. After penetrating the obstruction, the special pile is removed and the permanent pile is driven. The annulus is later backfilled with sand or grout.

22.13.2 Obstructions occurring at any depth

The second method mentioned earlier can be implemented for deeper obstructions as well, provided the hole remains open. Unfortunately, driving a special pile deep is somewhat equivalent to driving two piles. Other methods exist to tackle obstructions at deeper levels (Fig. 22.14).

22.13.2.1 Spudding

Spudding is the process of lifting and dropping the pile constantly until the obstruction is broken into pieces. Obviously, spudding cannot be done with lighter piles (timber or pipe piles). Usually, concrete piles and steel H-piles are good candidates for spudding.

Extremely high stresses are generated during spudding. The spudding piles should be approved by the engineer prior to be used in the site as spudding can damage the pile.

22.13.3 Augering and drilling

It is possible to auger through some obstructions. Usually, local contractors with experience in the area should be consulted prior to specifying augering or drilling through obstructions.

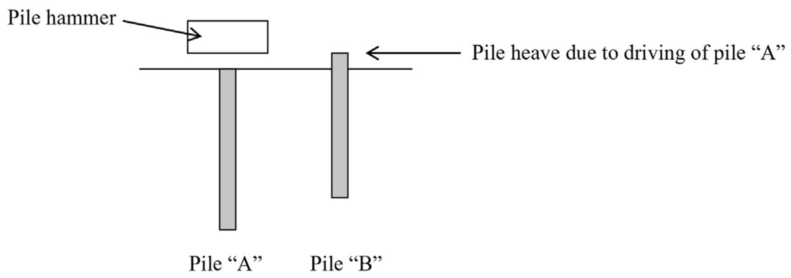


Figure 22.15 Pile heave and redriving.

22.13.4 Toe strengthening of piles

Toe of piles could be strengthened using metal shoes to penetrate obstructions.

22.14 Pile heave and redriving

- When driving piles in a group, nearby piles would heave due to driving (Fig. 22.15).
- It is important to redrive piles that had been heaved by more than 1 in (local codes need to be consulted by engineers with regard to pile heave).

22.15 Case study

- Pile Type: H-piles
- Hammer type: single acting steam hammer (Vulcan 016)
- Hammer energy: 48,750 ft. lbs
- Number of piles in the group = 40

In this case study, a group of piles were driven and pile heave was measured (Koutsoftas, 1982, Fig. 22.16).

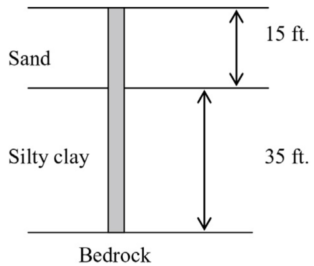


Figure 22.16 Case study: site conditions.

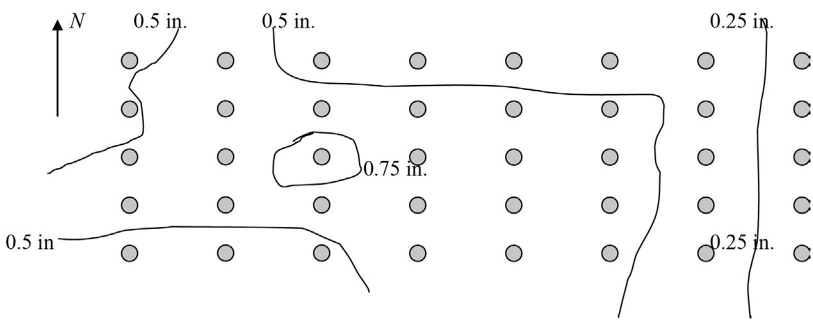


Figure 22.17 Pile driving sequence.

22.15.1 Site conditions

Pile group = 5 × 8 cluster

Pile type = H-piles, HP 14 × 117, grade 50 steel

Pile spacing = 3.5 ft. center to center

22.15.2 Pile driving sequence

One row at a time. Started from the southernmost row and moved north, driving row by row (see Fig. 22.17).

22.15.3 Pile heave contours

Piles at the middle heaved more than the piles at the periphery.

22.16 Soil displacement during pile driving

Volume displacement during pile driving is significant in clayey soils. On the other hand sandy soils tend to be compact. Volume displacement of saturated clayey soils is close to 100%. In other words, if a 10 $ft.^3$ pile is driven, 10 $ft.^3$ of saturated clay soil would need to find another home. Usually, displaced soil would show up as surface heave or displacement of nearby sheetpiling or basement walls (Fig. 22.18).

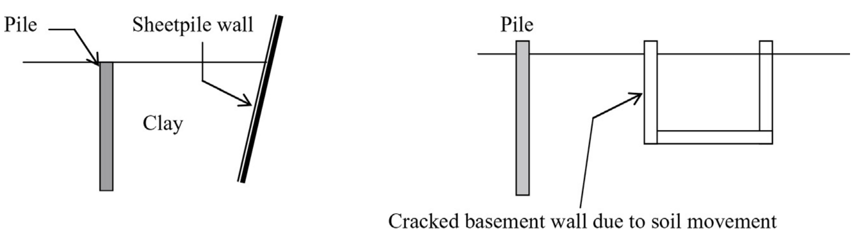


Figure 22.18 Soil displacement during pile driving.

- Failure of a sheetpile wall is shown due to pile driving.
- Sheetpile wall probably would not have failed if the soil had been sand instead of clay.
- Pile driving could damage nearby buildings due to soil displacement or pore pressure increase. If there is a danger of affecting nearby buildings, pore pressure gauges should be installed.
- If pore pressure is increasing during pile driving, the engineer should direct the contractor to stop pile driving till pore pressure comes down to an acceptable level.

References

Feng, Z., Deschamp, R.J., 2000. A study of the factors influencing the penetration and capacity of vibratory driven piles, soils and foundations. Jpn. Geotech. Soc. 40 (3), 43–54.

Koutsoftas, D.C., 1982. H-Pile heave: a field test. ASCE Geotech. Eng. J. 109 (10), 1363–1364.

White, D.J., et al., 1999. Press in Piling: The Influence of Plugging on Driveability. Eighth International Conference on Plugging on Driveability, New York, 1999.

Water jetting

23

Water jetting is the process of providing a high-pressure water jet at the tip of the pile.

Water jetting can ease the pile driving process by:

- loosening the soil
- creating a lubricating effect on the side walls of the pile

Water jetting is required in sandy soils.

23.1 Water jet types

Water jets can be external or internal (Fig. 23.1).

23.2 Ideal water pathway

Fig. 23.1 shows the ideal water pathway. Water should come back to the surface along the gap existing between the pile wall and soil. In some cases water may stray away and fill up nearby basements or resurface at a different location.

23.2.1 Pile slanting toward the water jet

- Piles have a tendency to slant toward the water jet when driving (Fig. 23.2).
- Pile slant toward the water jet can be avoided by providing two water jets symmetrically as shown in Fig. 23.1c and d. Providing the water jet at the middle also (Fig. 23.1a) is a solution to this problem.

23.2.2 Jet pipe movement

Many experienced pile drivers move the jet pipe up and down along the pile to ease driving. This process is conducted especially to keep the plumbness of the pile. This luxury is not available with internal jet pipes.

23.2.3 Water jetting in different soil types

- Sand: water jetting is ideal for dense sandy soils.
- Silt: water jetting can be successful in many silty soil beds. Yet, it is not unusual to have a clogged jet pipe. Clogging occurs when tiny particles block the nozzle of the jet pipe. Furthermore, it is important for the surrounding soil to close up on the pile after jetting is over. In some silty soils this may not happen. That would reduce the skin friction.

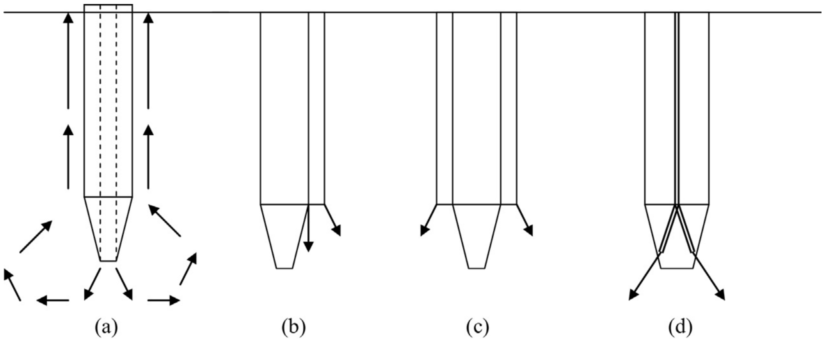


Figure 23.1 Water jetting. (a) Internal water jet pipe is shown here. Internal water jet pipes can be installed only in concrete piles and steel pipe piles. (b) External jet pipe attached to the side of the pile is shown. This is the usual procedure for timber piles. (c) Two jet pipes attached to opposite sides of the pile is shown here. (d) Internal jet pipe with two branches.

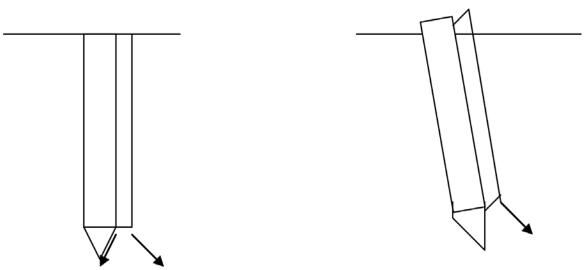


Figure 23.2 Slanting of water jet.

- Clay: water jetting can be used with different success levels in clay soils. Clay soils could clog the jet pipe as in the case of silty soils. Ideally, the water jet is supposed to travel back to the surface along the gap between pile wall and soil. In clay soils this gap may be closed due to adhesion and water may not be able to find a pathway to the surface. In such situations, jetting would not be possible.
- Gravel: water jetting is not recommended in loose gravel. Water would travel through gravel (due to high permeability) and may not provide any loosening effect. Water jetting can be used with different levels of success in dense gravel.

23.2.4 Advantages of water jetting

- Water jetting is a silent process.
- Water jetting could minimize damage to piles.

23.3 Water requirement

23.3.1 *Case 1: pile driving in dry sand (water table is below the pile tip)*

- $\dfrac{Q}{D} = 530\,(d_{50})^{1.3}\ L^{0.5} + 0.1\pi\,L\,K$

Here Q, flow rate of water required for pile jetting (units: m^3/h); D, pile diameter (m); d_{50} = 50% passing sieve size, L, pile length (m), K, permeability of soil (m/day) (Tsinker, 1988) (Fig. 23.3).

Note: when the water table is located above the pile tip, the flow rate required would be less than the computed value shown.

23.3.2 *Case 2: pile driving in dry multilayer sand (water table is below the pile tip)*

- $\dfrac{Q}{D} = 530\,(d_{50})^{1.3}\ L^{0.5} + 0.1\pi L K_m$

$K_m = (K_1 \cdot L_1 + K_2 \cdot L_2 + K_3 \cdot L_3)/(L_1 + L_2 + L_3)$.
K_m, combined permeability of sand layers (Fig. 23.4).

23.3.3 *Determination of required pump capacity*

$$H = \frac{Q^2 L_h}{C}$$

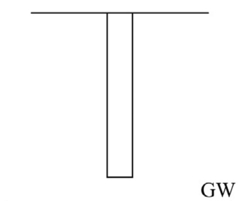


Figure 23.3 Groundwater and pile.

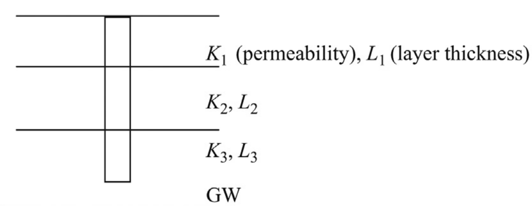


Figure 23.4 Piles in multilayer soil.

Table 23.1 **Head loss parameter for rubber hoses**

Jet pipe internal diameter (mm)	33	50	65	76
C	50	200	850	2000

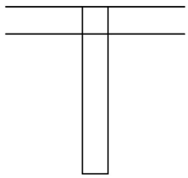


Figure 23.5 Pile driving in dry sand.

Here H, head loss in the hoses;
Q, flow rate (m^3/h);
L_h, total length of water supply hoses (m);
C, head loss parameter (obtained from Table 23.1).

23.3.4 Case 3: pile driving in dry sand (water table is above the pile tip)

- $$\frac{Q}{D} = 530(d_{50})^{1.3} L^{0.5} + 0.017\pi LK$$

Here Q, flow rate of water required for pile jetting (m^3/h);
D, pile diameter (m);
d_{50} = 50% passing sieve size;
L, pile length (m);
K, permeability of soil (m/day) (Tsinker, 1988) (Fig. 23.5).
Note: saturated sand requires much less water than dry sand.

Reference

Tsinker, G.P., 1988. Pile jetting. ASCE Geotech. Eng. J. 114 (3), 326–334.

Pile load testing

24

Pile load tests are conducted to assess the capacity of piles.

24.1 Theory

- Pile load test procedures can change slightly from region to region depending on the local building codes.
- A pile is driven and a load is applied to the pile.
- Fig. 24.1 is a highly simplified diagram of a pile load test. Nevertheless, it shows all the main parts that are needed to conduct a pile load test.
- Generally, to conduct a pile load test, one needs a driven pile, a load (usually steel and timber), a hydraulic jack, deflection gauge, and a load indicator.
- The gauges should be placed to measure the following:
  - the load on the pile-settlement of the pile

24.1.1 Another popular configuration

- In the configuration shown in Fig. 24.2, a steel beam is held to the ground by supporting piles. A hydraulic jack is placed between the pile to be tested and the steel beam.
- The jack is expanded pushing the steel beam upward while pushing the test pile downward into the ground.

24.2 Pile load test procedure

- Compute the ultimate pile capacity based on soil mechanics theory.
- Compute the design load using a suitable factor of safety (*FOS*)

$$\text{Design load}\,(D) = \frac{\text{Ultimate pile capacity}\,(U)}{FOS}$$

- Generally, total test load is twice the design load (this could change depending upon local building codes).

$$\text{Total test load}\,(Q) = 2 \times \text{Design load}\,(D)$$

In some sites negative skin friction is expected. In that case the following formula can be used:

$$\text{Total test load}\,(Q) = 2 \times (\text{Design load} + \text{Negative skin friction})$$

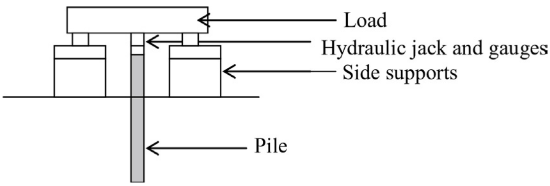


Figure 24.1 Pile load test.

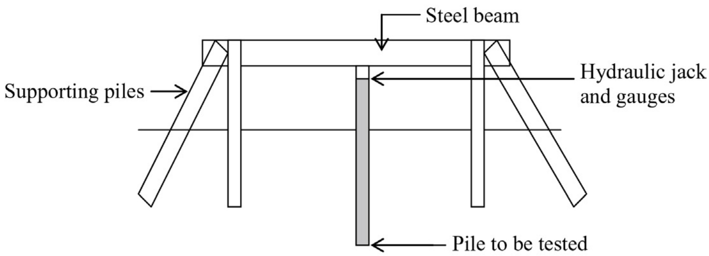


Figure 24.2 Load test configuration.

If negative skin friction is known with good accuracy, one can use the following equation:

$$\text{Total test load}\,(Q) = 2 \times (\text{Design load}) + \text{Negative skin friction}$$

- Apply 12.5% of the test load and record the settlement of the pile every 2 h.
- Readings should be taken until the settlement recorded is less than 0.001 ft. during a period of 2 h. When the pile settlement rate is less than 0.001 ft. per 2 h, add another 12.5% of the test load. Now the total load would be 25% of the test load. The settlement is monitored as previously. When the settlement rate is less than 0.001 ft. per 2 h, the next load is added.
- The next load would be 25% of the test load. Now the total load is 50%. The load is increased to 75% and 100% and settlement readings are taken.
- The load is removed in the same sequence and the settlement readings are recorded. At least 1 h time period should elapse during removal of loads.
- Final settlement reading should be recorded 48 h after removal of the final load (Fig. 24.3).

Q, maximum test load; and maximum test load (Q) is twice the design load. Settlement is marked along the Y-axis. The settlement would increase during application stage of the load and would decrease during removal stage of the load.

Gross settlement = the total settlement at test load (in this case it is 1.55 in.).

Gross settlement = settlement of pile into the soil + pile shortening

- It is obvious that the pile would compress due to the load and as a consequence the pile would shorten.

  Net settlement = settlement at the end after the load is fully removed (in this case it is 0.4 in.).

  Net settlement = settlement of pile into the soil after removal of the load

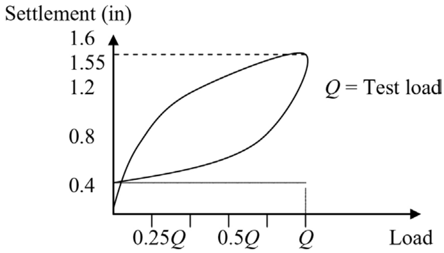


Figure 24.3 Load versus settlement.

- When the load is released the pile would return to its original length. In a strict sense there may be a slight deformation even after the load is removed. In most cases it is assumed that the pile had come to its original length when the load is removed.
- The pile load test is considered to *have failed*, if the settlement into the soil is greater than 1 in., at full test load, or the settlement into the soil is greater than 0.5 in., at the end of the test after removal of the load (Plate 24.1).

Plate 24.1 Pile load test.

24.2.1 Pile load test data form

The pile load test data form is present in

Pile Information

- **Pile type: wood__ H pile__ concrete__ pipe pile__**
- Pile Diameter: (Wood) Tip_______Butt________
- H pile: Section___________ Weight per foot___________
- Pipe Pile: Outer Diameter_______ Inner Diameter_______

The Equipment

Rig Type:______________
Mandrel:______________
Follower_______________

Cap: Type_____Wt:_______

The Hammer

Make:_______________
Model:_______________
Rated Energy:__________
Wt of Ram:___________
Stroke:_______________
Hammer Cushion: Thickness__

Project Name:_________________ Address________________________ Project #______________
Ground Elevation______________ Pile Toe Elevation________________ Length Driven_________
Pile Cutoff Elevation_____________

Pile Splice Data: Depth of the Splice:_____________; Type of the Splice:________________

Contractor Information:

*Contractor's Name:*___________________ *Field Foreman's Name:*__________________
*Contractor's Address:*__________________
*Pile Inspector's Name*________________ *Pile Inspection Company:*_________________
Design Load:
Test Load:

Weather: Temp:_________ Rain_________

Date	Time	Inspector's Name	Load (Tons)	Gauge Pressure (psi)	Settlement Gauge #1	Settlement Gauge #2	Remarks

Pile construction verification

25

After installing piles, they need to be inspected and verified. Typically an inspection firm would inspect the following items:

- straightness of the pile
- damage to the pile plumbness

25.1 Straightness of the pile

During installation, a pile can bend. This is known as dog legging (Fig. 25.1).

A flashlight attached to a rope is lowered to the pile to find out whether the pile is bent (Fig. 25.2, Plate 25.1).

If the flashlight is not visible from the top, it means the pile is bent.

Also, some inspectors drop a 4−5-ft. long rebar attached to a rope into the pile. The length of the rope is the same as the length of the pile. If the rebar goes all the way to the bottom of the pile, then the pile has no bend. If the rebar gets stuck in the pile, then the pile is bent (Fig. 25.3).

In a straight pile, the rebar will go all the way to the bottom of the pile as shown in Fig. 25.3. This can be verified by measuring the length of the rope. In a bent pile, the rebar gets stuck in the middle. More rope would be left on the ground level (Fig. 25.4).

25.2 Damage to the pile

Other than the bends, piles also could be damaged. If a pile is damaged, water would go into the pile (Fig. 25.5).

If water is detected inside the pile, the pile is damaged. Make sure that rainwater had not gone into the pile. Typically, the top of the piles are capped to prevent rainwater from entering the pile.

25.3 Plumpness of piles

Plumpness of a pile can be assessed using a plumb meter. The plumb meter is attached to the side of the pile. It would give the angle to the vertical (Fig. 25.6).

The plumb meter should be attached to the pile on one side and then should be attached to another side to assess the plumbness in the opposite direction (Fig. 25.7).

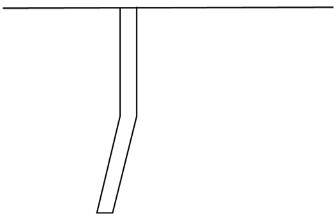


Figure 25.1 Dog legging or bending of piles.

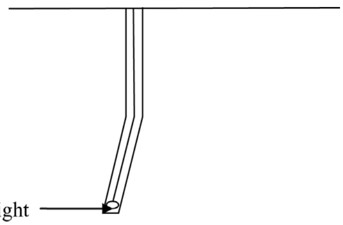


Figure 25.2 Flashlight is lowered to the bottom of the pile.

Plate 25.1 Bent piles identified and removed.

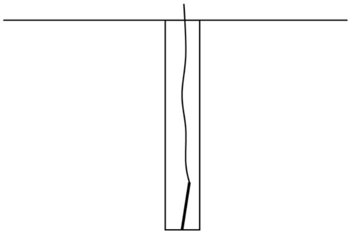


Figure 25.3 In a straight pile, rebar goes all the way to the bottom.

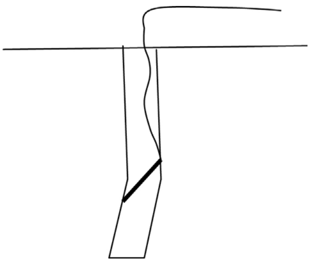


Figure 25.4 In a bent pile, rebar gets stuck in the bent.

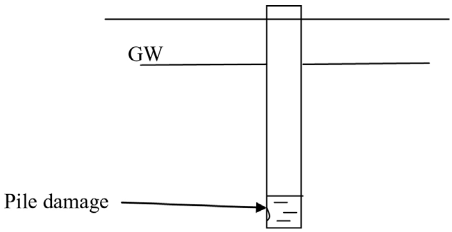


Figure 25.5 Damaged pile with water at the bottom.

Attach the plumb meter to two sides as shown. This way, plumpness on either direction can be found.

25.4 Pile integrity testing

25.4.1 Low-strain methods (ASTM D 5882)

1. Hand-held hammer is used to provide an impact to the pile. The impact would generate two types of waves:longitudinal waves
2. shear waves

Waves generated due to the hammer impact would travel along the pile and return. The sensor will relay the information to the analyzer. The analyzer would make

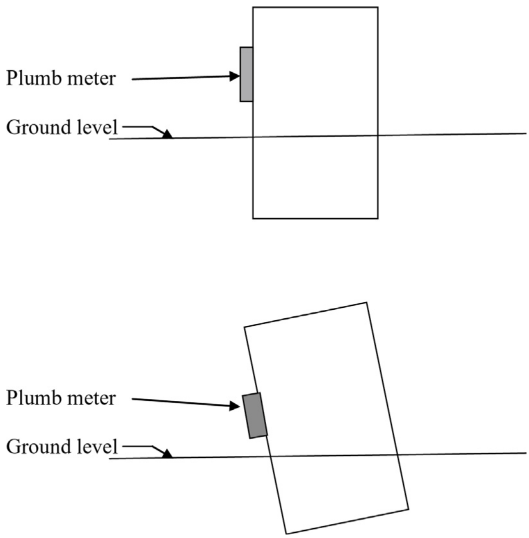


Figure 25.6 A plumb meter would indicate the angle of the pile with regard to the vertical.

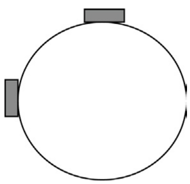


Figure 25.7 Plan view of a pile.

computations and provide information regarding the integrity of the pile. If the pile is in good condition, both waves would return fast. If the pile has a broken section, that would delay the signal. Delay of each wave would give information such as necking, dog legging (bends), and cracked concrete. Depending on the product some sensors are capable of recording the load on the pile, strain in the pile material, and stress relief (Fig. 25.8).

25.4.2 High-strain methods (ASTM D 4945)

Instead of a hand-held hammer, a pile-driving hammer also can be used. This technique can provide the bearing capacity of soil in addition to pile integrity. The wave equation is used to analyze the strain and the load on the pile (Fig. 25.9).

25.4.3 Radar analyzer

In this method, radar signals are sent to the pile hammer (Fig. 25.10). Radar signals would collide with the pile hammer and return back to the antenna. From this information, the sensors in the antenna would be able to detect the velocity of the pile.

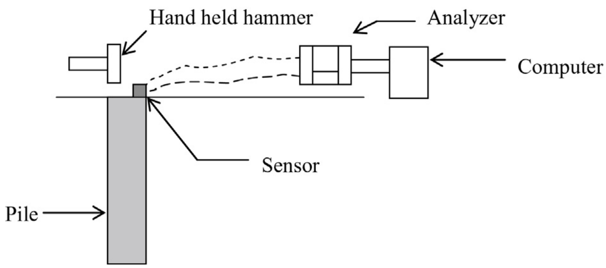


Figure 25.8 Low-strain methods.

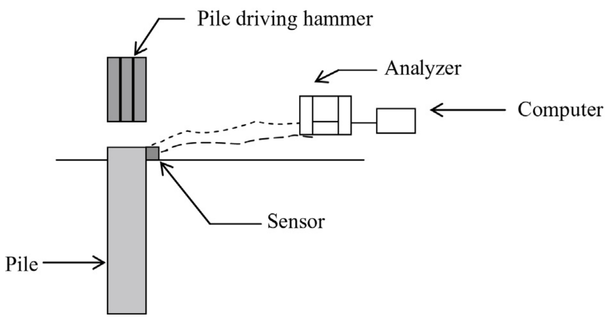


Figure 25.9 Pile and sensor.

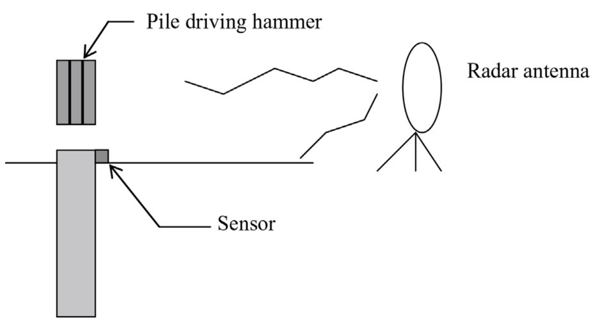


Figure 25.10 Radar analyzer.

From the velocity, it is possible to compute the kinetic energy imparted to the pile. Some radar devices are capable of monitoring the stroke as well.

25.5 Use of existing piles

After demolition of a structure, piles of the old structure may remain. The cost of construction of the new structure can be reduced if old piles can be used. Many building codes allow the use of old piles with proper procedures (Fig. 25.11).

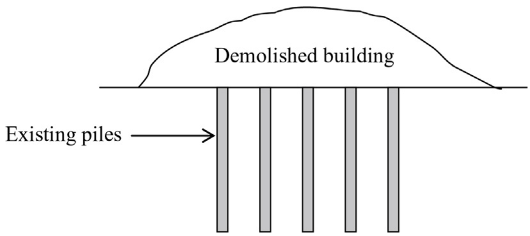


Figure 25.11 Use of existing piles.

The design engineer should conduct a thorough survey of old piles.
The following factors should be taken into consideration during the survey:

1. Capacity of old existing piles: if old geotechnical engineering reports could be obtained, it may be possible to find pile driving logs, pile load test data, and boring logs belonging to the old structure. These data could be used to find the capacity of old piles.
2. Reliability of old information: piles may have been driven decades ago. Reliability of old information needs to be assessed.
3. Present status of old piles: timber piles could be subjected to deterioration. Steel piles could get rusted. Concrete piles are susceptible to chemical attack.
   a. Test pits could be dug near randomly selected piles to observe the existing condition of piles.
   b. A few piles could be extruded to check their present status.
4. Pile load tests: pile load tests could be conducted to verify the capacity.
5. Pile integrity testing: seismic wave techniques are available to check the integrity of existing piles. These techniques are much cheaper than pile load tests.
6. Approval from the building commissioner: usually, the building commissioner needs to approve the use of old piles. Report of findings need to be prepared and submitted to the local building department for approval.

25.6 Environmental issues

25.6.1 Creation of water migrating pathways

- Piles are capable of contaminating drinking water aquifers. When piles are driven through contaminated soil into clean water aquifers, water migration pathways could be created. Water would migrate from contaminated soil layers above to lower aquifers.

As shown in Fig. 25.12, water would seep along the pile into the clean water aquifer below. When a pile is driven or bored, a slight gap between the soil and pile wall is created. Water could seep along this gap into clean water aquifers below. H-piles are more susceptible for creating water migration pathways than circular piles.

25.6.2 Wick effect

- Fig. 25.13 shows water seeping *through* the pile into the lower soil strata. This situation is common to timber piles. Researchers have found that treated timber piles are less likely to

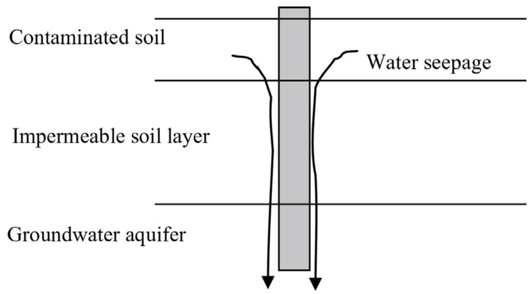


Figure 25.12 Water migration pathways.

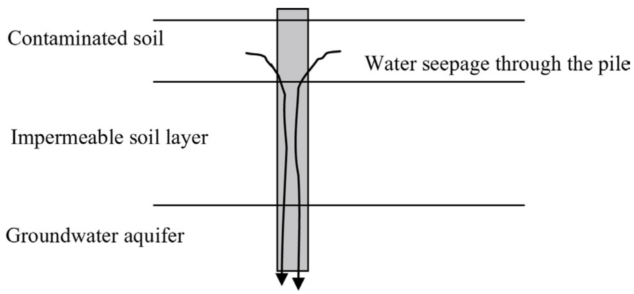


Figure 25.13 Wick effect.

transport water compared to untreated piles. A geotechnical engineer should be aware of this phenomenon when designing piles in contaminated soil conditions.

25.7 Utilities

25.7.1 Summary

Every geotechnical engineer should have a good knowledge of utilities. Utilities are encountered during drilling, excavation, and construction. In this chapter, basic information regarding utilities will be given.

25.7.2 General outline of utilities

In a perfectly planned city, utilities will be located as shown in Fig. 25.14.

Most cities were not built overnight. They were built over many centuries. Utility companies lay down utilities in accordance with the availability of space. Fig. 25.14 shows how utilities would have been laid out, if there were no obstructions.

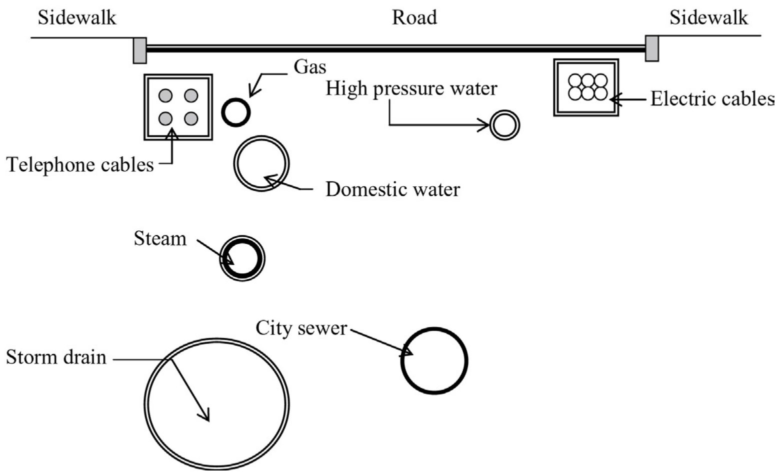


Figure 25.14 Utilities.

25.7.3 Electric, cable TV, and telephone lines

Usually located at shallow depths closer to the edge of the road. These cables are enclosed in wooden or plastic boxes as shown in Fig. 25.14.

25.7.4 Domestic water

Domestic water lines are located relatively deep (approximately 4–6 ft.). Domestic water lines consist of

- mainssubmainsbranch lines supplying buildings and houses

Note: The water main would go underneath a major street (Fig. 25.15). Side roads would get water from this main. Submains are connected to houses through branch lines. Main water lines are made of concrete or steel. Usually, water lines are laid on top of stones and back-filled with clean sand. Prior to a water main, clean fill will be encountered. Water is carried through pipes by pressure.

25.7.5 High-pressure water lines

These lines are needed for fire hydrants. Usually, these lines are smaller in diameter and lies at shallow depths near the edge of roads.

25.7.6 Sewer systems

Sewer lines are located at great depths away from water lines to avoid any contamination in the case of a sewer pipe failure. Usually sewer lines are located 10 ft. or more from the surface. Modern sewer lines are built of concrete. Old lines could be either

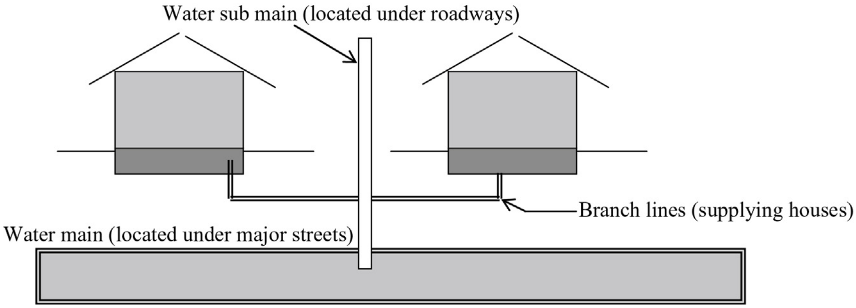


Figure 25.15 Water supply system.

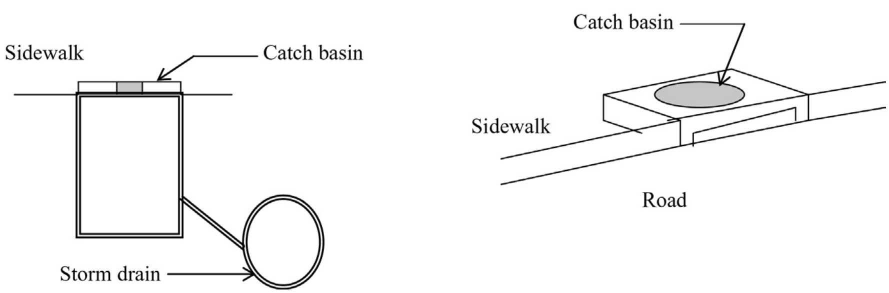


Figure 25.16 Storm drains.

clay or brick. Unlike water lines, sewer lines operate under gravity. Most sewer lines in major cities have been operating for many centuries without any interference from humans; some believe that new creatures have evolved inside them.

25.7.7 Storm drains

Storm drains are much larger than sewer pipes. Usually built using concrete (older systems could be bricks or clay pipes). Storm drains are responsible for removing water accumulated during storm events (Fig. 25.16).

Rainwater is collected at catch basins and carried away by storm drains.

25.7.8 Combined sewer

It is not unusual to connect sewer lines to storm drains. Usually near discharge points storm drains and sewer lines are combined. These lines are known as *combined sewer* lines.

Pile identification plan

26

Prior to driving piles, a pile identification plan should be developed. This plan would tell the pile driving company where to drive the piles and the driving criteria (Fig. 26.1). The following information should be provided in the pile identification plan:

- location of the pile shown with coordinates
- pile diameter and the type
- pile driving criteria
- batter angle (if present)

Table 26.1 is a general sample and can be modified based on the project.

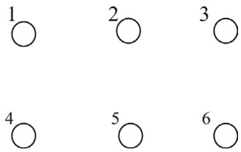


Figure 26.1 Pile locations.

Table 26.1 Pile identification plan

Pile numbers	*X*-coordinate (ft.)	*Y*-coordinate (ft.)	Pile descriptions
1	22.0	50.0	12″ diameter concrete pile to a depth of 100 ft.
2	32.0	50.0	12″ diameter concrete pile to a depth of 110 ft.
3	42.0	55.0	12″ diameter 0.5″ thick steel pipe pile to a driving criteria of 30 blows/in.
4	52.0	60.0	12″ diameter 0.5″ thick steel pipe pile to a driving criteria of 30 blows/in.
5	62.0	60.0	6″ diameter 0.5″ thick steel pipe pile to a driving criteria of 10 blows/in.
6	62.0	60.0	6″ diameter 0.5″ thick steel pipe pile to a driving criteria of 10 blows/in.

As built plans

27

After construction of piles, it is necessary to prepare an as-built pile location plan. Typically, a licensed surveyor would develop the as-built pile location plan.

- Fig. 27.1a shows design locations of a pile group. Fig. 27.1b shows actual locations where the piles were driven. As one can see, the pile group is not symmetrical. Pile "A" would be overloaded beyond the design capacity. Hence, the engineer may decide to add an additional pile to relieve the load on pile "A." Also, deviation of as-built locations can be tabulated (Table 27.1).

27.1 Batter information

In reality, a large number of piles would have a batter, after they have been driven. Piles that have a batter larger than the allowable limit should be identified. Batter angle of piles should be provided to the engineer.

Typically, batter angle (angle measured from the vertical) is measured and provided to the design engineer. If the batter angle is excessive, allowable capacity of the pile needs to be reduced.

If the allowable pile capacity is "q," and the pile has a batter angle of "α," new allowable pile capacity is given by $q \times \cos(\alpha)$

- A pile batter would reduce axial capacity of piles. If piles have excessive batter, then new piles need to be added (Fig. 27.2).

27.2 Use of as-built plans

There are situations where new buildings are built on piles driven for old buildings. It is impossible to determine the length of these piles. If an as-built plan is available, engineers will be able to ascertain the capacity of piles.

Also, there could be situations where some areas need to be excavated. During excavation work some piles may have to be removed. An as-built plan will be very helpful in determining how many piles need to be removed.

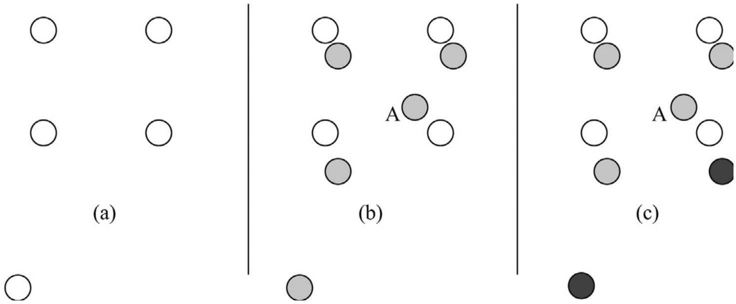


Figure 27.1 As-built and design locations. (a) Design pile location; (b) as-built location; and (c) additional pile.

Table 27.1 Pile as built location table

Pile numbers	Original location of *X*-coordinate (ft.)	Original location of *Y*-coordinate (ft.)	As-built *X*-coordinate (ft.)	As-built *Y*-coordinate (ft.)
1	22.0	50.0	22.05	50.01
2	32.0	50.0	32.02	50.03
3	42.0	55.0	41.96	54.97
4	52.0	60.0	51.08	51.87
5	62.0	60.0	62.03	60.03

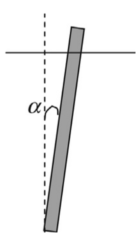


Figure 27.2 Pile batter.

Code issues (Eurocode and other building codes) 28

28.1 Eurocode

An overview of the Eurocode will be provided in this chapter. The following codes are applicable for different types of piles:

- EN 1993-5: Eurocode 3, part 5: design of steel structures: piling
- EN 1536:1999: bored piles
- EN 12063:1999: sheet pile walls
- EN 12699:2000: displacement piles
- EN 14199:2005: micropiles
- EN 12794:2005: precast concrete products. Foundation piles.

Section 7 (1997-1) discusses the following subjects:

7.1 General (3 paragraphs)
7.2 Limit states (1)
7.3 Actions and design situations (18)
7.4 Design methods and design considerations (8)
7.5 Pile load tests (20)
7.6 Axially loaded piles (89)
7.7 Transversely loaded piles (15)
7.8 Structural design of piles (5)
7.9 Supervision of construction (8)

28.1.1 Approaches to design

Eurocode allows three approaches:

Approach 1: design using results of static load tests.

Approach 2: empirical or analytical calculation methods. But the code adds that methods have to be verified through pile load tests.

Approach 3: the results of dynamic load tests. As in approach 2, the code states that the results have to be verified using static load tests.

28.2 Design using static load tests

$$R_{ck} = \min\left[\frac{(R_{cm})_{mean}}{\zeta_1}; \frac{(R_{cm})_{min}}{\zeta_2}\right] \tag{28.1}$$

Table 28.1 **Correlation factors**

n	1	2	3	4	>5
ζ_1	1.4	1.3	1.2	1.1	1.0
ζ_2	1.4	1.2	1.05	1.0	1.0

where R_{ck}, characteristic pile resistance; R_{cm}, measured pile resistance; $(R_{cm})_{mean}$, mean value of pile load tests; $(R_{cm})_{min}$, minimum value of pile load test data; and ζ_1 and ζ_2, correlation factors.

Correlation factors are obtained using Table 28.1

"n" is the number of load tests completed.

Design pile resistance is calculated using characteristic pile resistance (R_{ck}).

Let us see how to compute the characteristic pile resistance.

Step 1: write down all the measured pile resistances using static load tests.
Step 2: find the mean value of load test data. This is $(R_{cm})_{mean}$.
Step 3: find the minimum value of load test data. This is $(R_{cm})_{min}$.
Step 4: find correlation factors using Table 28.1.
Step 5: use Equation (28.1) to find R_{ck}.

Design example

Three static load tests were done and the following values were obtained:

2.4 MN, 2.7 MN, 3.2 MN

Find the characteristic pile resistance.

Solution

Step 1: find the mean value of load test data.

$[R_{cm}]_{mean} = (2.4 + 2.7 + 3.2)/3 = 2.77$ MN

Step 2: find the minimum value of load test data.

$[R_{cm}]_{min} = 2.4$ MN

Step 3: find correlation factors.

For three tests,

$\zeta_1 = 1.2$ and

$\zeta_2 = 1.05$.

Step 4: use Equation (28.1) to find R_{ck}.

Using Equation (28.1) $R_{ck} = \min\left[\frac{(R_{cm})_{mean}}{\zeta_1}; \frac{(R_{cm})_{min}}{\zeta_2}\right]$,

$R_{ck} = \min\ (2.77/1.2; 2.4/1.05)$

$R_{ck} = \min\ (2.31; 2.29)$

The minimum of the two preceding values is selected:

$R_{ck} = 2.29$.

28.3 Compute characteristic axial compression load using ground tests

Ground tests, such as CPT and SPT, can be used to find the characteristic pile resistance.

$$R_{ck} = \min\left[\frac{(R_{c.cal})_{mean}}{\zeta_3};\frac{(R_{c.cal})_{min}}{\zeta_4}\right] \tag{28.2}$$

This equation is similar to Equation (28.1);
where R_{ck}, characteristic pile resistance; $R_{c.cal}$, calculated pile capacity; $(R_{c.cal})_{mean}$, mean value of calculated pile capacity; $(R_{c.cal})_{min}$, minimum value of calculated pile capacity; and ζ_3 and ζ_4 are obtained from Table 28.2.

Let us look at an example.

Design example

Seven CPT cone tests were conducted in seven different holes. CPT test values were converted to calculated pile capacities for seven CPT tests.

2.4 MN, 2.7 MN, 3.2 MN, 2.8 MN, 2.1 MN, 3.0 MN, and 2.9M N

Find the characteristic pile capacity.

Solution

Step 1: find the mean value of the calculated pile capacities.

Mean value = (2.4 + 2.7 + 3.2 + 2.8 + 2.1 + 3.0 + 2.9)/7 = 2.73 MN

$(R_{c.cal})_{mean}$ = mean value of calculated pile capacity = 2.73 MN

Step 2: find the minimum value of calculated capacities.

$[R_{c.cal}]_{min}$ = minimum value of calculated pile capacity = 2.1 MN

Step 3: obtain correlation factors from Table 28.2 for the seven tests.

$\zeta_3 = 1.27$

$\zeta_4 = 1.12$

Step 4: Apply Equation (28.2).

Using Equation (28.2) $R_{ck} = \min\left[\frac{(R_{c.cal})_{mean}}{\zeta_3};\frac{(R_{c.cal})_{min}}{\zeta_4}\right]$,

R_{ck} = min (2.73/1.27; 2.1/1.12)

R_{ck} = min (2.15; 1.875)

The minimum of the two values is 1.875 MN.

Hence, R_{ck} = 1.875.

Characteristic capacity is used to find the design capacity. The procedure is not provided in this book.

Table 28.2 **Correlation factors for ground testing**

n	1	2	3	4	5	7	10
ζ_3	1.4	1.35	1.33	1.31	1.29	1.27	1.25
ζ_4	1.4	1.27	1.23	1.20	1.15	1.12	1.08

28.4 NYC building code

28.4.1 Pile capacity: dynamic equations

Many building codes around the world depend on dynamic formulas for guidance.

Table 28.3, based on the engineering news formula, is adopted by the NYC Building Code.

$$\text{Formula used } P = \frac{2W_h H}{2000(s+0.1)}; \qquad W_h H = E = \text{hammer energy}$$

Table 28.3 Minimum driving resistance and minimum hammer energy for steel H-piles, pipe piles, precast and cast-in-place concrete piles, and composite piles (other than timber)

Pile capacity (tons)	Hammer energy (ft. lb)	Friction piles (blows/ft.)	Piles bearing on cemented sand free of lenses of silt, clay, and soft rock (hardpan, blows/ft.)	Nondisplacement piles bearing on decomposed rock (blows/ft.)	Displacement piles bearing on decomposed rock (blows/ft.)	Piles bearing on rock
Up to 20	15,000	19	19	48	48	Five blows per 1/4 in. minimum hammer energy of 15,000 ft.lb
	19,000	15	15	27	27	
	24,000	11	11	16	16	
30	15,000	30	30	72	72	
	19,000	23	23	40	40	
	24,000	18	18	26	26	
40	15,000	44	50	96	96	
	19,000	32	36	53	53	
	24,000	24	30	34	34	
50	15,000	72	96	120	120	
	19,000	49	54	80	80	
	24,000	35	37	60	60	
	32,000	24	25	40	40	
60	15,000	96		240	240	
	19,000	63		150	150	
	24,000	44		100	100	
	32,000	30		50	50	
70 and 80	19,000		Five blows per 1/4 in.) Minimum hammer energy of 15,000 ft.lb	Five blows per 1/4 in) Minimum hammer energy of 19,000 ft.lb		
	24,000					
	32,000					

Here P, allowable pile load in tons; W_h, weight of striking part of hammer in pounds; H, actual height of fall of striking part of hammer in feet; E, rated energy of the hammer; and s, penetration of pile per blow in inches = 12/blows per foot.

W_p/W_h should not exceed 3 (W_p is the weight of the pile).

For timber piles, allowable pile load is measured using the preceding driving formula.

- For timber piles in soft rock and hardpan (cemented sand free of silt, clay or soft rock lenses):
- Driving resistance given by the formula should be maintained for last 6 in.
- For timber piles in gravel, sand, silt and clay soils.
- Driving resistance given by the formula should be maintained for last 12 in.

Design example

A pile is designed to carry 40 tons. The pile hammer selected has an energy rating of 24,000 ft. lb. Find the number of blows/ft. required to achieve a pile capacity of 40 tons.

Solution: 24 blows/ft.

When the pile reaches 24 blows/ft., the pile could be ended. The engineer needs to make sure that the pile is in the soil type that it is supposed to be in.

28.4.2 Driving criteria (International Building Code)

The International Building Code (IBC) states that the allowable compressive load on any pile should not be more than 40 tons, if driving formulas were used to compute the allowable pile capacity. According to the IBC, if an engineer were to specify an allowable pile capacity of more than 40 tons, wave equation analysis and pile load tests should be conducted.

Economic considerations and costing

29

It is no secret that piles are more expensive than shallow foundations. The cost of piles depends on the following items:

- pile material
- transportation cost
- pile length
- splicing cost
- equipment cost
- labor market
- design requirements

29.1 Pile material

Woodpiles are the cheapest. But woodpiles can be used only for small loads. Steel pipes and H-piles may be expensive. These piles have higher capacities compared to woodpiles. H-piles generate most of its capacity thru skin friction. H-piles have a large contact area with soil compared to other piles. This may not be the case in all instances. Concrete piles can be cheaper in some regions where steel prices are high.

29.2 Transportation cost

Transportation cost of piles can be significant. Special and careful handling is required for concrete piles. Concrete piles are brittle in nature and need to be handled with care. Unloading and loading of concrete piles need special attention. Large tensile stresses can develop in the pile and can damage the pile.

Fig. 29.1 shows lifting of a concrete pile. Due to sagging, the pile will undergo tensile stresses at point A. Concrete can crack under tensile loads. On the other hand, steel piles may not need delicate handling.

29.3 Pile length

The length and diameter of piles will directly affect the cost of piling.

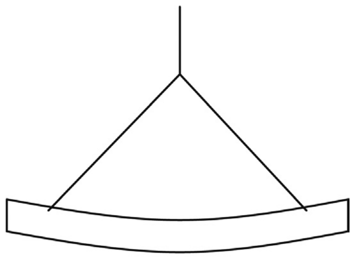


Figure 29.1 Lifting of a concrete pile.

29.4 Splicing cost

Splicing of steel piles needs welding. Welding of piles can be costly. In the case of concrete piles, two piles need to be attached to a groove. This could be time-consuming.

29.5 Equipment cost

A pile driving rig is a must for any pile driving project. Other than the pile driving rig, there are other equipment required for most projects. Some of the other equipment needs are the following:

- welding machine for pile splicing
- trucks to bring the pile to the location
- site clearing prior to pile deriving (some sites may have trees and bushes that need to be cleared)
- cofferdams to keep the pile driving area dry

For augercast piles, an auger rig is needed. Special cutter heads to penetrate boulders are also needed in some sites.

29.6 Labor market

Pile rig operators are rare and highly paid. In some areas these specialty workers may require substantial compensation.

29.7 Cost estimate for pile driving projects

As in any project, pile-driving projects consist of material and labor costs. The following guidelines would help to generate a cost estimate for a piling project:

- Mobilization cost: depends on the location, project size and equipment.
- Material cost: use the following chart.

Pile type	Linear Ft.	Unit cost	Total cost
Wood piles, Class __ *dia.___ treated_Y/N_,*			
Steel H-piles *Size__*			
Pipe piles *dia.___, wall thickness___*			
Precast concrete piles *Size__*			
Concrete filled pipe piles *Size__*			
Cast *in situ* concrete piles *Size__*			

- Pile shoes: No. of shoes required______, Cost of shoes________
- Pile splicing cost: Material cost for splicing_________, Labor cost ____

- ***Equipment cost**

Driving rig make___________, model____________
Estimated driving rate_________, total driving time__________,
Allowance to move from pile to pile______
Rental rate___________, cost of driving rig for the project___________

Mandrel rate________, cost of mandrel for the project__________

- **Labor**

Rig operator rate___________, operator cost for the project__________
Oiler rate_________, oiler cost for the project__________
Fireman rate__________, fireman cost for the project_______
Labor rate_________, labor cost for the project_________

Subject Index

E