Basic Geotechnical Engineering

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BASIC GEOTECHNICAL ENGINEERING
For
NON-GEOTECHNICAL ENGINEERS

By
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Buildings and Bridges and Retaining Walls

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1.0   INTRODUCTION

Geotechnical engineering is an interesting subject.   Unlike many engineering disciplines, it is not a pure science but rather it is an art form that requires both judgment and experience to arrive at a satisfactory solution. Unlike steel or concrete for instance, soil is quite different. Geotechnical engineers can arrive at different but equally satisfactory design values even when given the same set of information. Acceptable solutions are dependent upon many soil variables, methods used to predict the results and the practitioner's experience.

The purpose of this text is to acquaint primarily the non-geotechnical engineer with basic information related to geotechnical engineering in order to enhance his or her understanding of the subject. The topics discussed herein have been simplified and do not provide an exhaustive review of the subject matter. The information has been drawn from reference sources as well as from the author's experience. This publication is subject to the Disclaimers stated in Appendix B.

Topics have been selected for a wide-range audience and all topics have not been included. Searching the internet will reveal much additional information. The interested reader can also consult one of the many textbooks and other publications related to geotechnical engineering for more information on the topics discussed herein as well as those that have not been discussed.

Photograph 1.1 - Foundations for a Medieval Era Cathedral

2.0   EXPLORATIONS

2.1   Introduction

Civil engineering projects such as buildings, bridges, earthen dams, and roadways require detailed subsurface information as part of the design process. The ground below us ultimately supports all structures and to be successful, the ground must not fail under the applied structural load.

The geotechnical engineer's task is to explore the subsurface conditions at a project site, determine the capacity of the soil to carry the load without collapsing or experiencing intolerable movement and to recommend appropriate foundation alternatives. The task might also expand to provide recommendations in other related areas such as groundwater and earthwork. The scope of the soil exploration program including the number of explorations, equipment and testing is usually determined by a registered design professional such as geotechnical engineer.

The geotechnical engineer uses explorations to obtain samples of the soil for classification and testing purposes. Common types of exploration methods include.

- Soil test borings with standard penetration testing.

- Cone penetrometer soundings with cone penetration testing.

- Test pit excavations.

More discussion on these topics is provided in Sections 2.3 to 2.5.

Testing can be conducted in the laboratory with special samples retrieved for testing purposes. These tests might include methods for measuring the soil's shear strength, compressibility or permeability.

The explorations also include in-situ testing. These tests include methods such as the Standard Penetration Test (SPT) or the Cone Penetration Test (CPT) which are taken in soil test borings and cone penetrometer soundings respectively. The information obtained from these tests is used in the process of developing foundation design recommendations.

There is a wealth of published information correlating the test results obtained from the SPT or CPT to certain applicable engineering properties and soil values. The results of field testing and laboratory testing, coupled with the geotechnical engineer's assessment of subsurface conditions, engineering studies and experience are usually sufficient to provide satisfactory recommendations for a successful project.

An example of one correlation is shown in Figure 2.1. In this example, the results of the SPT are used to predict the internal friction angle of the tested soil. There is caution however in using correlations. Published correlations for the same set of values may vary. Therefore, correlations should be used for guidance and by experienced engineers.
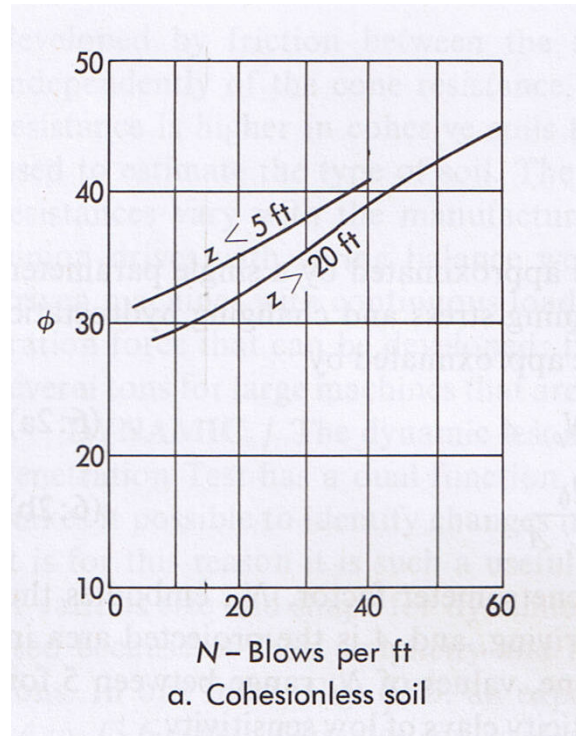

Figure 2.1
[Ref: Sowers and Sowers]

The geotechnical engineer is interested primarily in two pieces of information derived from the exploration program. This information can be used to develop appropriate recommendations for the engineer's task and includes:

- Characteristics of the material encountered, including groundwater.

- Engineering properties of the material and calculated values.

The type of material encountered is important because it provides an indication of how the soil will react under load and whether or not the material is even sufficient to support foundations. For instance clay reacts quite differently from sand. Peat and loose fill lying below a proposed structure are not suitable for supporting the structure. The poor material must be removed or stabilized or the foundations must be supported in firm material lying below the layer(s) of poor material.

Engineering properties of the soil are also important because they provide information on the shear strength of the soil and the ability of the soil to carry the load as well as the

settlement characteristics of the soil. Much of the information that the engineer uses is based on published values, results of past testing, empirical relationships and if necessary, the results of project specific testing.

2.2   Geologic Profile

When explorations are conducted, the information is recorded on a log.   By reviewing all of the logs from a particular site, the geotechnical engineer can formulate a three dimensional picture of the subsurface conditions. Of course this is based on taking individual explorations at specific locations at the site and then interpreting the soil conditions in between the explorations. This is sometimes difficult because it involves interpreting subsurface conditions that have not been explored between the exploration locations.

In short, the purpose of the exploration program is to provide sufficient site-specific information to enable the engineer to develop a picture of the subsurface environment and select appropriate soil values applicable to the soils encountered.   Often, the subsurface conditions are presented in a graphical geologic profile, which shows information from the log, soil strata and soil description. A typical profile is presented in Figure 2.2.
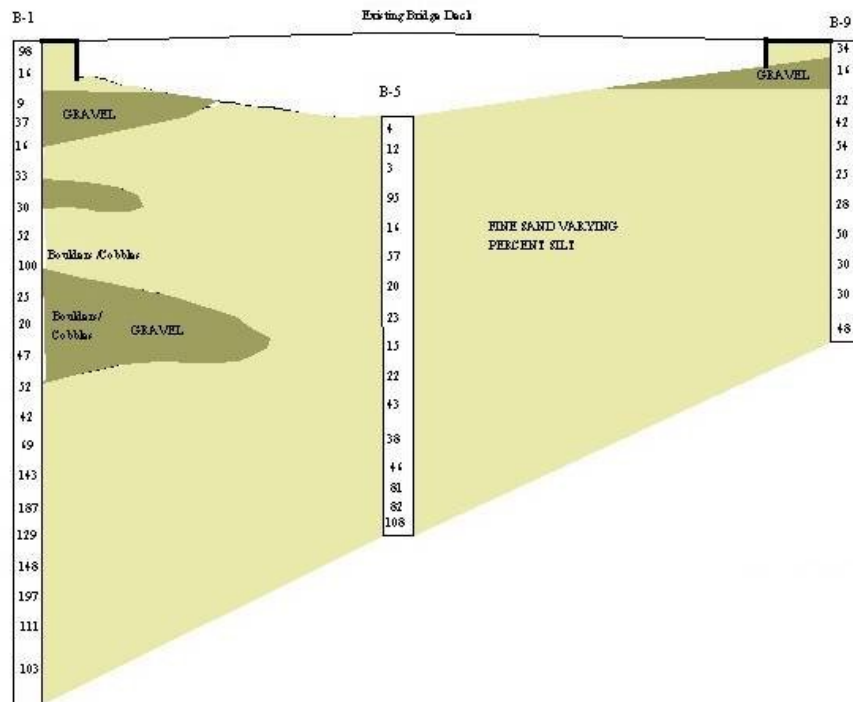

Figure 2.2 - Geologic Profile

2.3   Test Pits

Test pit excavations are useful for viewing large open areas to assess soil type and stratification but they also have drawbacks. Test pits are limited to the depth that the machine can extend, they are impractical to use for explorations below the groundwater level and they produce a large disturbed area; often times within the proposed building footprint. Most importantly, they do not provide penetration test results like the SPT and CPT, which are often used as the basis for making bearing capacity recommendations.

Photograph 2.1 – Test Pit Excavation

Often a subsurface exploration program is taken at locations around the perimeter of a proposed building footprint as well as within the footprint. Usually, the exploration program is conducted well before column lines and footing locations have been located in the field. A well-intentioned program of test pit excavations may place several excavations within or very close to the actual footing locations; and test pits are rarely backfilled with compacted fill suitable for supporting foundations.   There is a chance that a footing could be located directly over a test pit location.

Undoubtedly there are times when test pit excavations are an appropriate means of subsurface exploration. Test pits often allow the engineer to observe "the big picture". Because the size of the excavation is relatively large compared to that of a borehole, the engineer can get a first-hand look at the soil stratigraphy and the interface between varying subsurface materials becomes readily apparent. Test pits have their place in a well-designed program of subsurface explorations, but usually as a supplement to other exploration methods.

Test pits provide adequate material for soil classification but they do not always provide adequate data for assessing bearing capacity and settlement potential of the material. For

instance, a material described as "difficult" to excavate, when excavated using a small rubber-tired backhoe, may be classified as "easily excavated" when using a much larger piece of equipment. An assessment of how well the soil is compact; therefore, is highly subjective and there is no reliable correlation between degree of excavation difficulty noted on the logs and strength of soil, especially in sand and gravel. Test pits excavations, however, can provide some useful design information when verifying that the foundation soil is undisturbed glacial till or bedrock.

2.4   Soil Test Borings

Soil test borings are one way that geotechnical engineers retrieve information about the subsurface environment. Civil engineering projects such as buildings, bridges, earthen dams and roadways require detailed subsurface information as part of the design process. The ground below ultimately supports all structures and to be successful, the ground must not fail under the structural load. Failure can be defined as a sudden, catastrophic movement where the ground below the structure collapses because its resistance to the load is less than the applied load. Failure can also be defined as movement that is too great for the structure to accommodate. For instance, if the structure settles too much, cracks can develop in the frame and floor, windows and doors may not operate and the structure can become unsafe.

Photograph 2.2 – Standard Penetration Test

Soil test borings frequently are used to obtain samples of the soil for classification and testing purposes. Testing can be conducted in the laboratory or in the field at the time the

borings are made. For most projects field testing is sufficient because over the years, there has been a great deal of engineering data collected on the various soils. This, coupled with the geotechnical engineer's experience, is usually sufficient to provide sound advice for a successful project. However there are some projects where laboratory testing is crucial because of the complexity of soil and structure interaction or where the consequences of failure are great.

Soil test borings are simply a means of cleaning out a hole at various depths so that samples of the soil can be collected. The boring is advanced and the sides of the borehole are protected from collapse by using augers, flush joint casing (steel pipes) and in some cohesive materials such as clay, by using drilling mud. When the borehole has been advanced to a specific depth, usually 5-foot intervals, the standard split spoon sampler (a special 30-inch long pipe that spits into two sections) is placed on a steel rod, which is then inserted in the open borehole to the sampling depth. At this point, the driller is ready to collect a sample of the soil while at the same time, conduct the Standard Penetration Test.

The Standard Penetration Test is conducted by driving the split-spoon sampler into the soil at the testing depth using a 140-pound weight dropping 30 inches onto the top of the rods. As the sampler is driven into the ground, the number of hammer blows is counted for each 6-inch interval of movement. Usually the sampler is driven 24 inches into the ground although it need only be driven 18 inches. When this is finished, the sampler is retrieved and the barrel of the sampler is opened to reveal the sample of soil collected.

The geotechnical engineer looks at the sample and classifies the material as sand, silt, clay gravel or any combination based on a specific soil classification system. The description is noted on a log along with the sample depth, sample number, distance the sampler was driven (18 inches or 24 inches), how much soil was retrieved (recovery, not all of the sample may be recovered since some of the material may fall out of the sampler) and the number of hammer blows required to drive the sampler into the ground (blow count). This information is collected and written for each sample taken. The 140-pound weight and 30-inch drop onto the standard split spoon sampler are universal and; thereby, the "standard" method for conducting this test.

The soil test boring log is a collection of the soil and sample information for each sampling interval from the ground surface down to where the boring is terminated. An example of a soil test boring log is shown in Figure 2.3. In general, samples are taken at 5-foot intervals. However, since each sampler is driven 24-inches, there is only a 3-foot gap between soil samples. "Continuous" sampling can be conducted by taking intermediate samples, say from 0 to 2 ft, 2 to 4 ft and 4 to 6 ft, etc. This is particularly useful if the engineer is looking for a demarcation between materials such as the bottom of a peat deposit or the bottom of a fill layer that might be otherwise missed.

TEST BORING LOG — Sheet 1

Geotechnical Associates, Inc. Geotechnical Engineers	Project Description Location	BORING NO. B-1
		DATE: 2/15/03

Groundwater Observations

Ground Elevation: 1167.8
Date Started: 2/14/03
Date Finished: 2/14/03
Driller: Driller Inc.

Date	Depth (ft)	Casing	Stabilization Time
2/14	7	out	At completion

Depth (ft)	Sample No.	Pen / Rec.	Sample Depth	Blows / 6"	Type	Strata	Visual Description	Note
0–2	1	24/10	0-2	9-8-17-48	Ss	Silty Sand	2" topsoil to tan rust fine SAND little Silt	
5–6	2	24/14	5-6	19-30-33-33	Ss		Tan rust fine SAND little Silt trace Gravel	
8'								
10–12	3	24/17	10-12	10-17-23-30	Ss	Glacial Till 12'	Wet gray fine SAND some Silt trace Gravel	
13							Auger Refusal	

Notes:

Sample Type / Field Test	Proportioned Used		Casing	Sampler	Core
ss = split spoon	Trace 0 – 10%				
A = Auger	Little 10 – 20 %	Type	HSA	ss	
U = Undisturbed	Some 20 – 35%	ID	3	1-3/8"	
Tv = Pocket Torvane	And 35-50%				
Pp = Pocket Penetrometer		Hammer		140 lbs.	

Figure 2.3 – Soil Test Boring Log

As the boring is conducted, the geotechnical engineer pays particular attention to the soil classification. Soils having similar characteristics are grouped together into a soil layer. The engineer's interpretation of soil type and thickness is shown on the log as soil strata. By reviewing all of the logs from a particular site, the geotechnical engineer can begin to formulate a three dimensional picture of the subsurface conditions. This is sometimes difficult because it involves interpreting subsurface conditions between the boreholes without seeing the actual soil conditions and sometimes there are surprises. Geotechnical engineering on a project is rarely complete until the Owner receives the key to the door.

The Standard Penetration Resistance (N) determined by blow counts, especially in granular soil such as sand or gravel, is most important. In cohesive soil such as clay, although the blow count is important, often the engineer will conduct a field test using a

small piece of hand-operated equipment such as a Torvane or pocket penetrometer, or will conduct laboratory testing to determine the shear strength of the soil. In granular soil, the blow counts have been correlated to friction angle and unit weight. (This is why the test is conducted using a standard method: 140-pound weight falling 30 inches). The important numbers are the sum of the blow counts for the second and third 6-inch intervals (from 6-inches to 18-inches of penetration). The sum is called the Standard Penetration Resistance.

| Hoisting the hammer to drive the sampler | Drilling with an All Terrain Vehicle |

Photograph 2.3 – Soil Test Boring

The effective overburden pressure affects the soil resistance. Hence a soil having a Standard Penetration Resistance of 15 blows per foot located at a depth of 5 feet may not have the same strength (measured by ϕ) as the same soil having the same penetration resistance but located at a depth of 30 feet. Therefore it is common to correct the blow count (from the SPT test) and sounding (from the CPT test) obtained in the field-testing program. Although this correction is common, it is not universally applied. Various equations and curves are available to make this correction.

2.5   Cone Penetrometer

Some exploration programs use cone penetrometer soundings and the Cone Penetration Test (CPT) to derive foundation design values. The CPT uses a completely different testing device in the field. Instead of driving a sampler into the ground using a hammer, a standardized pointed rod is pushed into the ground using hydraulic pressure. The resistance at the tip of the cone point (cone resistance) and the measured friction along the side of the sampler barrel (friction resistance) provide the geotechnical engineer with information used to determine the classification of the soil and the engineering properties

of the material. Instead of a boring, this method is called a sounding but logs are also prepared to record this information.

Samples are not routinely retrieved as with a soil test boring; thus an interpretation of soil conditions and properties is based upon a correlation using the Friction Ratio. The Friction Ratio is defined as the frictional resistance (f_r) divided by the cone resistance (q_c). One example of a correlation is shown in Figure 2.4.
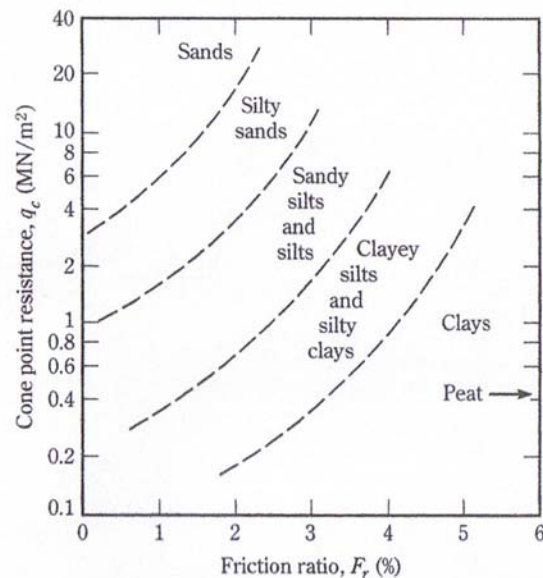

Figure 2.4 – Example of Correlation
[Ref: Das after Robertson and Campanella (1983)]

The discussion in the previous section related to interpreting subsurface conditions between exploration locations applies here as well.

2.6   Depth of Explorations

Explorations should penetrate through all soil layers comprised of unsatisfactory bearing material such as fill, organic deposits, loose sand and soft, compressible clay. At least one exploration should extend to a depth where the increase in vertical stress caused by the structure equals 10 percent of the initial vertical effective overburden stress below the foundation. Consideration must also be given to the depth where liquefaction might be an issue especially in loose granular deposits. Explorations must provide satisfactory information within the critical depth for bearing capacity analysis. This depth is at least twice the minimum width of shallow square foundations and at least 4 times the minimum width of continuous footings or embankments.

2.7   Groundwater Observation Wells

Geotechnical engineers are often called upon to provide recommendations for subsurface drainage. Most often this occurs in connection with design of below ground structures such as basements or below level parking garages. Decisions based on the location of the groundwater level can have a significant impact on design, construction costs and long-term serviceability of the structure. Therefore the engineer must have a reasonable degree of confidence in the groundwater level and its seasonal fluctuation.

Measuring the groundwater level in a borehole at completion does not always provide a satisfactory measurement. Seldom are boreholes left open to allow readings after a 24-hour period. The type of soil and drilling method used influences the credibility of the groundwater level measured at the completion of the boring

Slow draining soils generally do not provide enough time for the groundwater level to stabilize. When hollow flight augers are used to advance the borehole, the water level measured at the completion of the boring can be lower. When wash boring techniques are used, the water level can be higher than the actual groundwater level. These considerations might not be significant if the groundwater level is located well below the proposed bottom (ground floor) of the structure. But if the groundwater lies within a few feet of the structure, then the need for a more accurate groundwater level measurement is warranted. Thus a groundwater observation well can be installed to allow long-term measurement of the groundwater level.

3.0   SOIL PROPERTIES

3.1   Introduction

Soils are sediments and other unconsolidated material comprised of solid particles produced by disintegrations of rock and mixtures of such particles with organic substances. A volume of soil also contains liquid and gasses filling the void between the particles. Hence, a volume of soil is comprised of three phases: solid, liquid and gas.

3.2   Weight-Volume Relationship

Visualize for a moment a shovel full of soil. Likely, you will find solid particles such as sand of various sizes with voids between the particles. The voids are filled with air and quite possibly, some moisture.   Imagine now that this sample is confined within a unit volume and all the solid particles are compressed together without any voids between the particles. Visualize that the water (moisture) contained in the sample collects on top of the solids and the air rides at the very top of the volume. This describes the three-phase diagram shown in Figure 3.1a. The diagram is presented in two dimensions rather than three.

In each of the definitions discussed, refer to the diagram shown in Figure 3.1.

The weight relationship of the phases is shown on the right hand side of the diagram while the volume relationship of the phases is shown on the left hand side of the diagram. It is important to note that each of the three phases, solid, liquid and gas have a volume but only solid and liquid have weight. Amongst geotechnical engineers, the gaseous state (i.e. air) has no weight. We're not picky about the molecular weight of air so in the grand scheme of things, the weight of air is zero.

From the diagram shown in Figure 3.1a, it is evident that:

- W = total weight of the mass while W_s = the weight of the solid phase, and W_w = the weight of the liquid (water) phase. Note that the total weight W is equal to $W_s + W_w$.

- V = total volume of the mass while V_s = the volume of the solid phase, V_w = the volume of the liquid (water) phase and V_a = the volume of the gaseous (air) phase. Note that the total volume V is equal to $V_s + V_w + V_a$. From now on, we'll refer to liquid as water and gas as air.

The volume of water and the volume of air comprise the volume of voids between the soil particles. The volume of the voids can be totally dry in which case there is no water or it can be totally full of water in which case there is no air. Both water and air can also be present in the volume of the voids. Note that the volume of voids $V_v = V_w + V_a$.
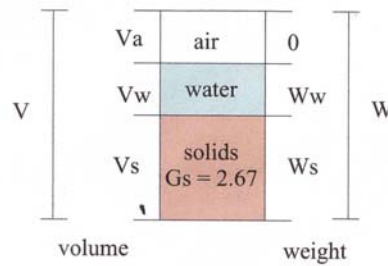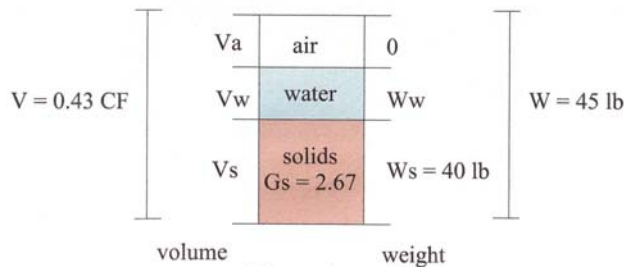
Figure 3.1 – Three Phase Diagram

The following definitions apply to soil.

3.2.1 Void Ratio

$$\text{Void Ratio (e)} = Vv \, / \, Vs \qquad (3.1)$$

Void ratio expresses the relationship between the volume of voids to the volume of solids in a unit volume of material. For a given sample of soil, a dense material has a lower void ratio than a loose material. When material is compacted in the field as part of constructing engineered fill, there is a void ratio reduction. The solid particles are forced closer together thus reducing the volume of the voids. For instance, when an 8-inch thick layer of soil is compacted, it becomes less than 8 inches thick. The volume of the voids is reduced by the compaction.

3.2.2 Porosity

$$\text{Porosity (n)} = (Vv \, / \, V) * 100 \qquad (3.2)$$

Porosity expresses the relationship between the volume of voids and the total volume. The higher the porosity of a material, the more porous the material becomes. Note that a

soil with high porosity may not necessarily be highly pervious. Clay for instance has a high porosity but low permeability. Porosity is expressed as a percent.

3.2.3 Degree of Saturation

$$\text{Degree of Saturation } (S) = (Vw / Vv) * 100 \quad (3.3)$$

Degree of saturation expresses the relationship between the volume of water and the volume of the voids. Saturation is expressed as a percent. As shown earlier, if all the voids were filled with water, then Vw = Vv and S = 100%. The material would be fully saturated.

3.2.4 Water Content

$$\text{Water Content } (Wc) = (Ww / Ws) * 100 \quad (3.4)$$

Water content expresses the relationship between the weight of water in a given volume of material to the weight of the solids contained in that same volume. Water content is expressed as a percent.

3.2.5 Total Unit Weight

$$\text{Unit Weight } (\gamma) = W / V \quad (3.5)$$

The unit weight relationship is the *total* unit weight of soil because it relates total weight (Ws + Ww) with total volume (Vs + Vw + Va). Note, if the material is moist or saturated, this would be the moist or saturated unit weight of soil.

3.2.6 Dry Unit Weight

Sometimes, it is important to know the dry unit weight of soil (γd), especially when calculating the degree of compaction. The dry unit weight of soil is expressed as:

$$\text{Dry Unit Weight } (\gamma d) = Ws / W \quad (3.6)$$

During field control of compaction it is necessary to know the "degree of compaction" attained. The inspector or engineer will calculate the in-place *dry* unit weight of soil retrieved from the compacted fill and compare it to the theoretical maximum *dry* unit weight of the material determined by laboratory testing.

3.2.7 Specific Gravity

Specific gravity is another value used in calculations.

$$\text{The mass or apparent specific gravity } (Gm) = W / (V * \gamma w) \quad (3.7)$$

The specific gravity of the solids (Gs) = Ws / (Vs * γw)   (3.8)

Note that γw is the unit weight of fresh water (i.e. 62.4 lb pcf).

From these relationships it is possible to make other engineering calculations. There are other relationships that are not shown but they are all based on the fundamental relationships discussed herein. Other relationships can be found in publications such as DM-7 (see references).

Example 3.1

Refer to Figure 3.1b for an example calculation using the relationships expressed above. The diagram shows that 43 pounds of material was retrieved from a hole that had a volume of 0.41 cubic feet. The material was dried and reweighed. The dry weight (weight of the solids Ws) is 40 pounds. From this information and using the relationships expressed above or derived from the phase diagram calculate:

Total Unit Weight (γ) = W / V = 43 / 0.41 = 104.9 pcf

Dry Unit Weight (γd) = Ws / V = 40 / 0.41 = 97.6 pcf

Volume of Solids (Vs) = Ws / (γw * Gs) = 40 / (62.4 * 2.67) = 0.24 cf (from Eq. 3.8)

Weight of water (Ww) = W – Ws = 43 – 40 = 3 lb

Volume of Water (Vw) = Ww / δw = 3 / 62.4 = 0.05 cf

Volume of air (Va) = V – Vs – Vw = 0.41 – 0.24 – 0.05 = 0.12 cf

Volume of voids (Vv) = Vw + Va = 0.05 + 0.12 = 0.17 cf

Void ratio (e) = Vv / Vs = 0.17 / 0.24 = 0.71

Porosity (n) = Vv / V = 0.17 / 0.41 = 41.5%

Degree of Saturation (S) = Vw / Vv = .05 / 0.17 = 29.4%

Water content (Wc) = Ww / Ws = 3 / 40 = 7.5%

Example 3.2

Again, refer to Figure 3.1b for the relationships. Assume that the total unit weight of a sample of soil is 117 pcf. The material is 100 percent saturated and the water content is 41 percent. Calculate the void ratio (e).

If the material is 100 percent saturated than all of the voids are filled with water and the volume of air (Va) equals zero. Since the total unit weight is 117 pcf, the total weight (W) = 117 pounds and the total volume (V) equals 1 cubic foot.

The water content is 41 percent; therefore Ww / Ws = 0.41 which becomes Ww = (Ws) (0.41)

The total weight (W) = Ww + Ws and by substitution for Ww, W = (Ws)(0.41) + Ws and W = Ws(1 + 0.41)

Solving for Ws where W = 117 pounds, Ws = 117 / 1.41 or 83 pounds. Therefore, Ww = 117 – 83 = 34 pounds

The soil is 100 percent saturated, Vv = Vw and Vw = Ww / (γw) = 34 / 62.4 = 0.54 cf

Since V = Vs + Vw, then Vs = V – Vw. Therefore, Vs = 1 – 0.54 = 0.46 cf

By definition, Void Ratio (e) = Vv / Vs = 0.54 / 0.46 = 1.17

There types of calculations are used to derive soil values.

3.2.8   Atteberg Limits

Atteberg Limits, most commonly Liquid Limit and Plastic Limit, are an integral part of several engineering classification systems to characterize fine grained soil. Fine grained soil such as silt and clay are finer than the No. 200 sieve (finer than 0.002 mm grain size). These limits along with Plasticity Index can be used with other engineering properties to correlate with engineering behavior such as compressibility and permeability.

As a clayey soil is mixed with excessive water, it flows like a semi-liquid. As the material dries, it passes through a plastic, semisolid and then solid state. There is a reduction in the water content and also the void ratio as the material shrinks. The water content at which the soil changes from a liquid to plastic state is the Liquid Limit (LL) and the water content at which the soil changes from a plastic to semisolid state is the Plastic Limit (PL). Although these limits represent water content, they are expressed without the percent designation.

The Plasticity Index (PI) is the difference between the Liquid Limit and the Plastic Limit and is a measure of plasticity.

$$PI = LL - PL \quad (3.9)$$

A high Plasticity Index indicates that the material has significant clay content, while a low Plasticity Index near 0 indicates that the material is non-plastic such as silt.

Methods for determining the Liquid Limit, Plastic Limit and Plasticity Index of soils are described in ASTM D4318.

3.2.9 Shear Strength

Shear Strength is a fundamental engineering property of soil and it is usually expressed as:

$$S = c + \sigma \tan(\phi) \quad (3.10)$$

Where S = shear strength
  c = cohesion (property of cohesive soil)
  σ = normal stress on shear plane (usually the effective weight of the soil overburden above the shear plane)
  ϕ = angle of internal friction of soil

For clay (cohesive soil) in undrained conditions, f = 0, thus S = c.

For sand (cohesionless soil), c = 0, thus $S = \sigma \tan(\phi)$.

For a soil that exhibits both cohesion and friction, Equation 3.10 expresses the shear strength.

3.2.10 Sensitivity

Most clay loses some of its strength and stiffness when remolded or disturbed. The main cause may be a reorientation of the individual clay particles to a less favorable orientation. Sensitivity is determined in the laboratory as the quotient of the undisturbed strength to the remolded strength. Commonly the unconfined compression test with a value of the unconfined compressive strength (qu) is used to determine strength.

$$\text{Sensitivity} = qu \text{ (undisturbed sample)} / qu \text{ (remolded sample)} \quad (3.11)$$

Insensitive clay that does not lose significant strength when disturbed has sensitivity less than 2. On the other hand, "quick" clay loses significant strength and has a sensitivity that exceeds 16. A common classification is shown in Table 3.1.

Table 3.1 – Sensitivity Classification

Sensitivity	Classification
<2	Insensitive
2-4	Moderately sensitive
4-8	Sensitive
8-16	Very sensitive
16-32	Slightly quick
32-64	Medium quick
>64	Quick

[Ref: Foundation Engineering Handbook,]

Sensitivity can also be described by the type of clay as shown in Table 3.2.

Table 3.2 – Soil Description vs. Sensitivity

Sensitivity	Soil Description
2-8	Clay of medium plasticity, normally consolidated
10-80	Highly flocculent, marine clay
1-4	Clay of low to medium plasticity, overconsolidated
0.5-2	Fissured clay, clay with sand seams

[**Ref: Sowers and Sowers**]

3.2.11 Engineering Properties of the Mass

Solutions to engineering problems requiring an assessment of engineering properties of soil and rock involve determining quantitative information on the mass involved. When a site is explored for instance, a finite number of samples are retrieved from which engineering values are derived for the entire study area. This leads to two questions that must be answered by the engineer undertaking the assignment.

1. Are the samples tested representative of the mass?

2. What are the combined effects of stratification, cracks, planes of weakness and other geometric and structural aspects of the mass?

In most cases, experience and judgment are required to interpret the results so that they can be used to develop a satisfactory engineering solution.

4.0   BEARING CAPACITY OF SHALLOW FOOTINGS

4.1   Introduction

A foundation is that part of a structure which transmits a load directly into the underlying soil. If the soil conditions immediately below the structure are sufficiently strong and capable of supporting the required load, then *shallow spread footings* can be used to transmit the load. On the other hand, if the soil conditions are weak, then piles or piers are used to carry the loads into deeper, more suitable soil.   Shallow footings are foundations where the depth of the footing is generally less than the width (B) of the footing.

Photograph 4.1 – Shallow Spread Footings

Geotechnical engineering is a branch of civil engineering that works with soil properties to establish the allowable bearing capacity of shallow footings. Geotechnical engineers are members of the design team who provide this information to those responsible for design. Often it is stated that geotechnical engineering is an "art form" rather than a science. Much of the geotechnical engineer's guidance results from an interpretation of subsurface conditions based on an economically reasonable number of explorations.

Based on experience and supported by theory, the geotechnical engineer interprets the information in order to predict foundation performance. The prediction usually ends up in a recommendation made by the geotechnical engineer in a report.   Architects and structural engineers are probably most familiar with statements such as "The recommended allowable bearing pressure for shallow spread footings at this site is 4000

psf." Where does this value come from and what was considered when establishing this value?

There are two considerations for determining the allowable soil bearing pressure:

- Calculated theoretical bearing capacity and

- Magnitude of settlement

Thus, the magnitude of settlement that a footing might experience under the design load is an equally important criterion for establishing the allowable soil bearing pressure. In fact for footings wider than 3 feet, settlement consideration often controls the magnitude of pressure applied to the soil.

The ability of soil to safely support a structure is of paramount importance. If the capacity of the soil is not sufficient then failure will occur. Failure can be defined as:

- A sudden, catastrophic movement where the ground below the structure collapses because its resistance to the load is less than the applied load. This relates to the *capacity* of the soil to safely carry the load (Criterion 1)

- *Movement* that is too great for the structure to accommodate. For instance, if the structure settles too much, cracks can develop in the frame and floor, windows and doors may not operate and the structure can become unsafe. This relates to the settlement potential of the soil under the applied load (Criterion 2).

Bearing capacity analysis is a two-part method used to determine the ability of the soil to support the required load in a safe manner without gross distortion resulting from objectionable settlement. The *ultimate bearing capacity* (q_u) is defined as that pressure causing a shear failure of the supporting soil lying immediately below and adjacent to the footing. The geotechnical engineer's task is to explore the subsurface conditions at a project site and determine the allowable capacity that the soil can carry without collapsing or experiencing intolerable movement. These precepts apply equally to deep foundations as well as shallow foundations.

4.2   Modes of Failure

Generally three modes of failure have been identified:

- General Shear Failure: A continuous failure surface develops between the edge of the footing and the ground surface. This type of failure is characterized by heaving at the ground surface accompanied by tilting of the footing. It occurs in soil of low compressibility such as dense sand or stiff clay.

- <u>Local Shear Failure</u>: A condition where significant compression of the soil occurs but only slight heave occurs at the ground surface. Tilting of the foundation is not expected. This type of failure occurs in highly compressible soil and the ultimate bearing capacity is not well defined.

- <u>Punching Shear Failure</u>: A condition that occurs where there is relatively high compression of the soil underlying the footing with neither heaving at the ground surface nor tilting of the foundation. Large settlement is expected without a clearly defined ultimate bearing capacity. Punching will occur in low compressible soil if the foundation is located at a considerable depth below ground surface.

4.3   Bearing Capacity of Continuous Footings

First we will discuss calculating the bearing capacity for continuous footings using the original equation developed for bearing capacity analysis and then we will expand this to discuss other shapes and conditions.

The failure mechanism for a narrow, continuous footing (length is >> than width) assumes that a wedge of soil below the footing is pushed downward by the applied load, thereby displacing soil adjacent to the wedge both laterally and upward.   The ultimate bearing capacity therefore, is a function of the shear strength of the soil and the magnitude of the overlying surcharge due to the depth of footing (D).   The ultimate bearing capacity (qu) of soil underlying a shallow strip footing can be calculated as:

$$qu = 1/2\gamma B N\gamma \ + \ cNc \ + \gamma DNq \qquad (4.1)$$

- $N\gamma$, Nc and Nq are bearing capacity factors that depend only upon the soil friction angle (ϕ) as shown in Figure 4.1. The soil friction angle is commonly assigned by using charts or tables that correlate the penetration resistance obtained during the exploration program to the friction angle.

- The cohesion term "c" is obtained by laboratory or field-testing methods such as using a Torvane. Correlations using SPT results are unreliable for assigning cohesion.

- The unit weight of the soil (γ) is commonly based on a published correlation with soil classification.

- The value "B" is the width of the footing and is the common symbol for the width.

- The value "D" is the depth of the footing below the lowest adjacent backfill. If the footing is backfilled equally on each side, then D is the depth below grade. If

the footing is backfilled unequally on each side as in a basement, then D is the lesser measurement.
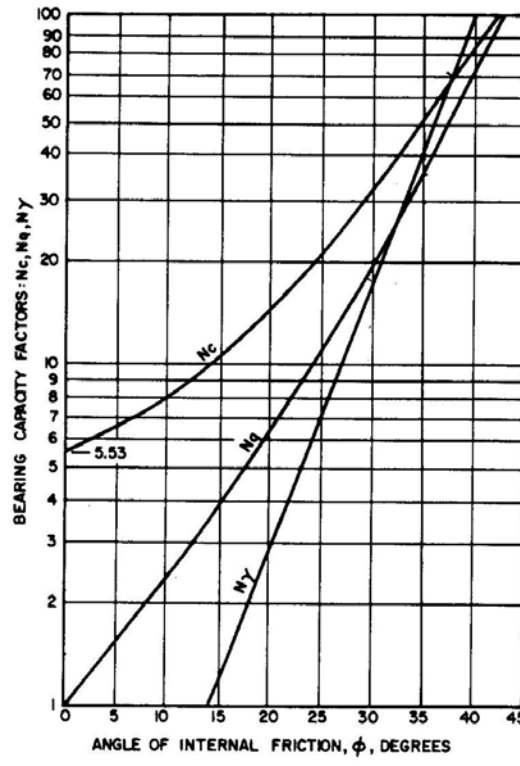

Figure 4.1 – Bearing Capacity Factors
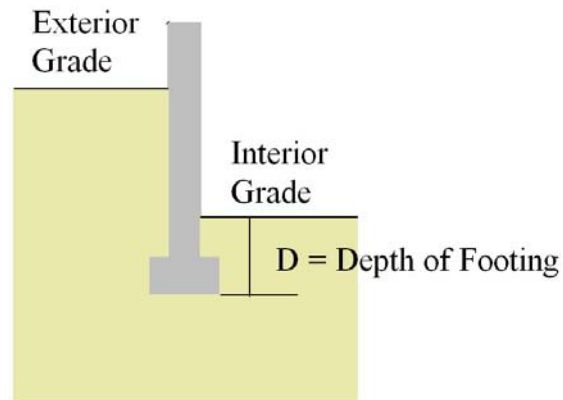**[Ref: NAVFAC DM-7]**

Figure 4.2 – Depth of Footing

Expression (4.1) above shows that there are three components to bearing capacity.

- The first term ($1/2\gamma B N\gamma$) results from the soil unit weight below the footing.

- The second term (cNc) results from the cohesive strength of the soil.

- The third term (γDNq) results from the surcharge pressure, which is the pressure due to the weight of material between the surface and footing depth. This third term has a significant influence on the calculated soil bearing capacity.

4.4   Modification for Shape

The original bearing capacity equation shown in Expression (4.1) applied to continuous footings where the length (L) is very much greater than the width (B). Since many footings however are square, rectangular or circular, the equation for a continuous footing was modified to account for the shape of the footing. Semi-empirical shape factors have been applied to each of the three components of the bearing capacity equation resulting in the following modifications:

- Square Footing: qu = 0.4γBNγ + 1.2cNc + γDNq

- Circular Footing: qu = 0.3γBNγ + 1.2cNc + γDNq

- Rectangular Footing: qu =1/2(1 – 0.2B/L)γBNγ + 1.2cNc +γDNq

In some publications, 1.3 replaces the factor 1.2.

4.5   General Bearing Capacity Equation

Later research improved the simple bearing capacity equations shown above by introducing a correction factor for shape of footing with load eccentricity, depth of footing, and inclination of load. Thus, the *General Bearing Capacity Equation* has evolved as shown in Expression (4.2), which maintains the contribution from the three components identified earlier and incorporates appropriate correction factors for each term.

qu = 1/2γBNγ (FγsFγd Fγi) + cNc(FcsFcdFci) +γDNq(FqsFqd Fqi)   (4.2)

The factors beginning with "F" are the correction factors for depth (d), shape (s) and inclination of load (i) applied to the original terms proposed in Expression (4.1).

Further refinements include correction factors for sloping ground and tilting of the foundation base.

The ultimate bearing capacity obtained when using the General Bearing Capacity Expression (4.2) give bearing pressures that are too large for footings having widths (B) greater than approximately 6 feet. Accordingly, a correction factor can also be applied to the first term of the General Bearing Capacity equation.

The calculation of bearing capacity and correction factors can become quite involved. Since there is no clearly defined universal set of values and equations used by all practitioners, it would not be unusual for the calculated results to vary among practitioners even when given the same set of subsurface conditions.

4.6   Groundwater and Bearing Capacity

The groundwater level affects the bearing capacity of soil. The first and third term of the bearing capacity equation include a factor for the unit weight of soil. Parts of these terms are shown below and identified as (4.3) and (4.4).

$$(1/2\gamma B) \qquad (4.3)$$

$$(\gamma D) \qquad (4.4)$$

When the groundwater level rises to a depth less than B (width of footing) below the footing, then the first term (4.3) changes. The unit weight of soil (γ) becomes affected by the groundwater. As the groundwater level rises, the unit weight below the groundwater level is replaced by the submerged unit weight ($\gamma - 62.4$) and a weighted average is used to express the effective soil unit weight in term (4.3).

When the groundwater level reaches the depth of footing, the value (γ) in term (4.3) is replaced entirely by (γ'), the submerged unit weight of soil. If the groundwater level rises above the depth of the footing, then the submerged unit weight of soil would be used in terms (4.3) and (4.4) as appropriate. Since the submerged unit weight of soil (γ') is always *less* than the total unit weight (γ), the bearing capacity decreases. Note in particular that:

- Term (4.3) can be reduced by up to approximately one-half of its value depending upon the depth of the water below the footing and the assigned value of γ [1/2 (γ-62.4) B].

- When the groundwater level rises *above* the depth of the footing then Term (4.4) is also affected [(γ-62.4) D].

- These conditions reduce the bearing capacity of the soil. Therefore the future highest groundwater level is important.

- If the groundwater level is at an intermediate depth ranging between the bottom of the footing and depth B, a weighted average effective unit weight is used in the bearing capacity equation [Ave $\gamma = \gamma' + d/B (\gamma - \gamma')$] where γ' is the submerged (effective) unit weight of soil, B is the footing width and d is the depth of the groundwater below the footing (i.e. d < B). .

4.7 Factor of Safety

Unlike materials such as steel or concrete, there is no code that specifies the allowable stress or factor of safety used in design. Soil has considerable variability and structures have a multitude of uses and design life. Although the magnitude of the safety factor can vary depending upon uncertainty and risk, a factor of safety of 3 is commonly used in bearing capacity analysis for dead load plus maximum live load. However, when part of the live load is temporary such as earthquake, wind, snow, etc. then the factor of safety can be lower.

The gross allowable bearing pressure used for design is derived by dividing the ultimate bearing capacity (qu) by the assigned factor of safety (FS).

$$q_{all} = q_u \: / \: FS \quad (4.5)$$

Often the surcharge pressure resulting from the depth of footing (soil surcharge) is subtracted yielding the net allowable bearing pressure.

$$q_{all(net)} = (q_u - \gamma D_f) \: / \: FS \quad (4.6)$$

The factor of safety is applied to the bearing capacity at failure as presented in Criterion 1. Footings less than 3 feet wide are most affected by this condition. As the footing becomes larger, the potential settlement of the footing plays a much greater role in establishing the assigned allowable bearing pressure as presented by Criterion 2.

4.8 Presumptive Bearing Capacity

Building Codes provide the maximum allowable pressure on supporting soils under spread footings. The BOCA National Building Code establishes the presumptive load-bearing value of foundation material based solely on material classification. The materials range from the weaker materials such as clay with an allowable bearing pressure of 2000 psf to very strong material such as crystalline bedrock with an allowable bearing pressure of 12,000 psf. The IBC lowers the allowable foundation pressure for clays to 1500 psf.

NAVFAC Design Manual 7.2, "Foundations and Earth Structures", provides a comprehensive tabulation of presumptive bearing pressures and modifications based on size, depth and arrangement of footings as well as the nature of the bearing material. The publication suggests the use of presumptive values for preliminary estimates or when elaborate investigation of soil properties is not justified.

4.9   Other Considerations for Bearing Capacity

There are other considerations that the geotechnical engineer must consider when deriving the bearing capacity of soils. Some of these considerations are outlined below:

- Footings with eccentric rather than concentric loads

- Depth of footings

- Bearing capacity of layered soils where a stronger soil is underlain by a weaker soil

- Seismic bearing capacity of soil

- Bearing capacity of soil supporting machine foundations

- Foundations on or close to slopes

- Footings supported on soils that expand or shrink with changes in the moisture content

4.10   Selection of Engineering Properties

The bearing capacity calculation is very sensitive to the values assumed for the shear strength of soil, namely the friction angle (ϕ) and cohesion. This is especially true at the higher values of friction angle. Therefore, careful consideration should be given to the values selected to define the soil shear strength.

4.11   Important Points

Some important points to consider are:

- The foundation is that part of a structure which transmits the load directly into the underlying soil.

- Shallow spread footings distribute the load over a wide area so that the bearing pressure does not exceed the capacity of the soil to carry the load without objectionable settlement.

- Shallow footings are footings where the depth of the footing is generally less than the width of the footing.

- If the capacity of the soil is insufficient, failure can occur as a sudden, catastrophic movement or movement that is too great for the structure to accommodate.

- Bearing capacity analysis seeks to prevent catastrophic movement and to limit movement to within tolerable ranges for the structure.

- Explorations are conducted in order to present a picture of subsurface conditions, including the nature of the material and the engineering properties. Often correlations are used between test values obtained during the exploration program and published engineering properties of the soil.

- Empirical relationships are often used to predict the bearing capacity of the soil and the settlement potential.

- Given the same set of soil information, different engineers can arrive at different but equally correct values for bearing capacity.

4.12   Deterministic vs. Probabilistic Analysis

The *deterministic method* of analysis is widely practiced in the United States. In the deterministic method, a single set of soil properties such as friction angle, cohesion, and unit weight are selected by the engineer based on some rational method. The ultimate bearing capacity is calculated using these singular values and a selected factor of safety is applied to yield the allowable bearing pressure. The deterministic method however, does not take into consideration the possible (and likely) variability of the assigned soil values. A primary deficiency of the deterministic method is that the parameters (material properties, strength and load) must be assigned single, precise values when in fact the actual (and appropriate) values might be quite uncertain.

Another approach to assessing the bearing capacity of soil is to use a probabilistic method of analysis, which reflects the uncertainty in the assigned values. Probabilistic methods however are not commonly used. The factor of safety concept is extended to incorporate uncertainty in the parameters. The probabilistic approach is more meaningful than the deterministic approach alone since the engineer incorporates uncertainty into the analysis. Both methods of analysis can complement one another since they each have a value that enhances the other method.

Example 4.1

Assume that a 4-foot square shallow spread footing is supported on sand at a depth of 4 feet below ground surface. The friction angle of the sand is 30 degrees, the unit weight of soil is 120 pcf and cohesion is zero. The groundwater level can rise to the depth of the bottom of the footing but no higher.   The cumulative average standard penetration resistance of the sand within a depth of 8 feet (2B) below the footing is 12 blows per foot.

Determine the allowable bearing capacity based on the shear strength of the soil and the ability of the soil to resist the applied pressure.

- Select the ultimate bearing capacity expression for square footings:

$qu = 0.4\gamma BN\gamma + 1.2cNc + \gamma DNq$

since cohesion = 0,

$qu = 0.4\gamma BN\gamma + \gamma DNq$

- For a friction angle of 30 degrees, determine the bearing capacity factors from Figure 4.1. $N\gamma = 16$ and $Nq = 18$

- The unit weight (γ) is given as 120 pcf. However, since the groundwater will rise to the depth of the footing, use the submerged unit weight ($\gamma - 62.4$) in the first term. Thus, $\gamma' = (120 - 62.4) = 57.6$ pcf.

- The ultimate bearing capacity is:

$qu = 0.4\gamma BN\gamma + \gamma DNq$

$qu = (0.4)(57.6)(4)(16) + (120)(4)(18) = 10115$ psf (rounded)

- For a factor of safety of 3, the gross allowable bearing capacity is $q_a = q_u / FS =$ **3372 psf**

- The net allowable bearing capacity is $q_{a(net)} = (q_u - \gamma D_f) / FS =$ **3212 psf**

- Assume the groundwater level never rises above a depth of B below the footing.

$qu = (0.4)(120)(4)(16) + (120)(4)(18) = 11712$ psf (rounded)

$qa = 11712/3 =$ **3904 psf**

This value is 532 psf higher and illustrates the effect of the groundwater on the calculated theoretical bearing capacity. The bearing capacity is higher because the soil is not affected by groundwater, and the total unit weight of soil (120 pcf) is used in term 1 rather than the submerged (buoyant) unit weight (57.6 pcf).

5.0   Settlement of Shallow Footings

5.1   Introduction

Settlement of footings must be considered as part of the foundation design process. For shallow footings, after a bearing capacity analysis has estimated the allowable soil pressure based upon shear strength consideration, settlement must be studied to refine (and possibly further limit) the assigned bearing pressure. The soil design pressure and footing geometry are checked to verify that settlement of the footing under the prescribed load lies within tolerable ranges for the structure. Settlement must also be considered for deep foundations.

The total settlement of a structure is not as much of a concern as the differential settlement that occurs between adjacent columns and structural members. Differential settlement between adjacent footings develops stresses in the structure causing damage. Of course, if the predicted total settlement of a structure would affect underground utilities, entryways, building elevations etc., then total settlement is also a concern. Allowable bearing pressures are designed to limit total settlement and by so doing, differential settlement between adjacent footings is also limited.

Where there is a group of footings supporting a structure, it is common to select the footing that might experience the most settlement for analysis. This could be the largest footing because its stress influence will extend much deeper thereby encompassing more soil or it could be the footing supported over the weakest soil at the site. In practice, it is common to adjust the design bearing pressure so that the footing will experience total settlement of less than 1 inch. Using this criterion, it is generally assumed that if the maximum settlement of footings is limited to 1 inch, then the *differential* settlement between adjacent footings within the group will likely be less than ¾ inches. This magnitude of differential movement is acceptable for most buildings. Tables and charts have been published which set forth tolerable settlement for various types of structures.

Methods of predicting settlement provide only an estimate of the actual expected movement. The calculations used to estimate settlement are based upon assigned soil properties derived from field-testing and laboratory testing methods that are in themselves imperfect. There is wide room for variation of soil properties and error without close attention to detail. Even under the best of circumstances, soil properties can vary. Factors such as water content, freeze-thaw cycles, groundwater level, degree of consolidation, rate of loading, soil stratification, degree of compaction and relative density of the material can change the soil strength and compressibility properties. Settlement can also occur as a result of both static and dynamic loads applied to the foundation soil.

5.2 Components of Settlement

Settlement caused by a loading condition that increases the stress in the underlying soil can be classified into two major components:

- Immediate settlement.

- Consolidation settlement.

Consolidation settlement can be further divided into:

- Primary consolidation

- Secondary consolidation.

5.3 Immediate settlement

Immediate settlement (elastic deformation) takes place during construction or shortly thereafter and results from compression between the soil particles.

5.4 Primary Consolidation

Primary consolidation is a time-dependent phenomenon that occurs as water is squeezed from the voids lying between the individual soil particles. The time required for primary consolidation to occur is a function of how quickly the soil drains.

5.5 Secondary Consolidation

Secondary consolidation occurs after primary consolidation has been completed. Unlike primary consolidation, secondary consolidation does not depend upon drainage. Secondary consolidation is caused by slippage and reorientation of soil particles (creep) under constant load.

Each of the three components of settlement occurs to some degree in both coarse-grained and fine-grained soil such as sand and clay respectively. Immediate settlement is most often associated with granular, coarse-grained soil such as sand. Although consolidation occurs in coarse-grained soil, it takes place very quickly because the material is relatively pervious and drains quickly. Therefore consolidation is not usually distinguishable from immediate settlement. Although secondary consolidations is thought not to occur in coarse-grained soil, some researchers have identified additional movement (creep) that occurs long after the load has been applied.

Primary consolidation and secondary consolidation are most often associated with fine-grained material such as clay and organic soil. Immediate settlement occurs rapidly in fine-grained material much more so than the time-dependent, long-term settlement

associated with primary and secondary consolidation. Primary consolidation is more significant in clays while secondary consolidation is more significant in organic soil.

The total settlement that occurs below a footing is the sum of each of the three components identified above:

S (total) = S (immediate) + S (primary) + S (secondary)

For coarse-grained soil, primary and secondary settlement is ignored.

5.6 Settlement of Footings Underlain by Sand

Settlement that occurs in coarse-grained soil (sand) is normally small and happens relatively quickly. It is generally thought that little additional long-term movement (creep) occurs after loading. However, some researchers propose that this might not be entirely true.

Calculations performed to estimate settlement in coarse-grained material are usually undertaken using empirical methods based on data obtained during the exploration program. Since it is expensive and impractical to obtain "undisturbed" samples of coarse-grained material for laboratory testing, predictions are based on field-testing methods such as the standard penetration test (SPT), cone penetration test (CPT), dilatometer test (DMT) and the pressuremeter test (PMT). Researchers have synthesized information collected from testing programs and studies and have developed a number of empirical relationships to estimate the settlement of footings underlain by granular soil.

Geotechnical engineers have used empirical approaches based on a large number of case studies to estimate the settlement of coarse-grained soil under sustained load. Two widely accepted methods employ the results obtained from the SPT and CPT. Equipment used to make these tests are readily available and relatively inexpensive to employ. These tests are routinely conducted during the site exploration program.

There are numerous empirical relationships available for predicting settlement. Some are apparently better than others in predicting the actual settlement based on the results of full-scale tests conducted on five shallow spread footings under various magnitudes of load. Some of the conclusions derived from a symposium convened during the mid 1900s to evaluate the current industry and academic practice in spread footing design are as follows:

- No participant who provided a calculated prediction of settlement gave a complete set of answers, which consistently fell within plus or minus 20% of the measured footing settlement.

- The load required to produce 1 inch of settlement was underestimated by 27% on average. The predicted load was on the safe side 80% of the time.

- A large variety of methods were used to calculate settlement and it was not possible to identify the most accurate method because most participants used published methods <u>modified</u> by their own experience or used a <u>combination</u> of methods.

- The profession tends to be over-conservative.

<u>One</u> (of many) empirical methods for predicting the settlement of shallow footings underlain by sand is illustrated below as an example. Researchers based this method on a statistical analysis of over 200 settlement records of foundations supported on sand and gravel. The expression shows a relationship between the compressibility of the soil, footing width and the average value of the penetration resistance derived from the SPT and uncorrected for overburden pressure.

The immediate settlement prediction for sand is:

$$Si = qB^{0.7}Ic \qquad (5.1)$$

Where:

- $Ic = 1.71/N^{1.4}$ and N is the Standard Penetration Resistance derived from the soil test boring exploration program.

- Si is expressed in millimeters

- B (footing width) in meters

- q (foundation pressure) in kPa

A modification can be made to this equation if the sand can be established as over consolidated. Although it is normally assumed that settlement will stop after construction and initial loading has been applied, data suggests that settlement can continue. A conservative assumption is that the settlement will ultimately reach 1.5 times the predicted settlement (Si) after 30 years.

5.7   Settlement of Footings Underlain by Clay

The settlement prediction for footings underlain by clay usually ignores immediate settlement. The magnitudes of primary and secondary consolidation are more important in clay and organic soil. Primary consolidation occurs when the pore water in saturated clay is drained (squeezed out) by the superimposed stress increase caused by the footing. As the material drains, settlement occurs.

The phenomenon of primary consolidation can be illustrated as follows: When a footing resting above saturated clay is loaded, there is a stress increase in the underlying material

equal to the amount of the increased foundation pressure. Initially, the pore water held in the voids of the soil between the clay particles supports all of the increased stress. Since the water is incompressible, the water pressure increases an amount equivalent to the increased foundation pressure (excess pore water pressure). With time, the pore water drains from the voids (decreases), thereby transferring the stress from the water to the soil particles. As the pore water drains, settlement occurs. Primary consolidation is complete when all of the excess pore water pressure has dissipated and the soil particles in close contact with one another support all of the pressure.

In order to predict the amount of settlement that will occur in the clay stratum, the engineer must have knowledge of the past history and engineering properties of the clay. This is achieved by retrieving an undisturbed sample of the clay and testing it in laboratory to measure its consolidation characteristics. The results of the laboratory-testing program are presented on a series of semi-log plots. One of these plots shows the decrease in void ratio or strain (vertical axis) in relationship to the increased pressure of load. From this data the engineer obtains important engineering properties of the soil, which are then used to predict the magnitude of settlement.

The settlement for normally consolidated material can be expressed as:

$$S = (Cc\ H/\ 1+e_o) * \log ((\sigma_o' + \Delta\sigma)/\sigma_o') \qquad\qquad (5.2)$$

Where:

- Cc is the compression index derived from laboratory testing

- H is the thickness of the clay layer under consideration

- σ_o' is the effective overburden pressure

- $\Delta\sigma$ is the stress increase resulting from the footing

- e_o is the soil void ratio obtained from laboratory testing

A slight manipulation of this equation will provide the settlement for an over-consolidated material.

- *Normally* consolidated material is material that has <u>not</u> experienced a load greater than the existing (current) load.

- *Over-consolidated* material is material that has experienced a load in the past greater than the existing (current) load.

An example of over-consolidated conditions might be illustrated by a 10-foot high hill that is underlain by clay. If the hill was 20 feet high in the past, then the clay would

already have settled under the weight of the 20-foot high hill. Since over-consolidated material is stronger than the same normally consolidated material, it is less compressible up to the point where the applied pressure is equivalent to the maximum past pressure. Therefore, if an additional 5 feet of fill is placed over the site to a total height of 15 feet, then the underlying clay would experience very little settlement because it has already experienced settlement equivalent to the previous 20-foot high fill.

Since manipulations are made to the equations for calculating settlement based on three possible conditions, the geotechnical engineer must also know the magnitude of the maximum past pressure, which can be obtained from laboratory test results. With this information, the geotechnical engineer can now relate the pressure increase in the underlying compressible soil resulting from the new footing to the existing overburden pressure and the maximum past pressure of the soil. The three possible conditions are:

- Settlement lies entirely within normally consolidated clay.

- Settlement lies entirely within over-consolidated clay where the new foundation pressure plus the existing overburden pressure is less than the maximum past pressure.

- Settlement lies in over-consolidated clay but extend into the normally consolidated zone where the new foundation pressure plus the existing overburden pressure is greater than the maximum past pressure

If secondary consolidation is calculated separately, then the results are added to the predictions for primary consolidation.

5.8   Time Rate of Settlement

Aside from predicting the magnitude of settlement that will most likely occur in fine grained-soil, the engineer must also predict the <u>rate</u> at which the total settlement will occur. There is a significant difference on performance and damage to a structure relating to 2 inches of settlement that occurs over a 1-year period as opposed to 2 inches of settlement that occurs over a 50-year period. The coefficient of consolidation (cv) required to conduct this study is also derived from laboratory test data.

In addition, the engineer must decide whether there is *two-way* or *one-way* drainage.

- Two-way drainage will occur if the clay stratum is located between two more pervious layers of material. The last drop of water to drain from the system is located in the middle of the clay stratum and it only has to travel one-half the thickness of the clay stratum or less until it reaches the pervious layer.

- One-way drainage occurs if the clay is overlain or underlain by a single more pervious stratum. In this case, the last drop of water to drain lies at the bottom or top of the clay stratum furthest from the drainage layer.

The rate of consolidation is expressed in Expression (5.3) below. From this expression, it should be easy to see that two-way drainage occurs more quickly than one-way drainage for the same thickness (H) of clay.

$$\text{Time} = Tv\ H^2/cv \qquad (5.3)$$

Where:

- Tv is a time factor and is obtained from published values.

- cv is the coefficient of consolidation and is obtained from laboratory testing or published values.

Sometimes the compressible material contains thin sand lenses. Since the sand lenses are also drainage pathways, the actual rate of consolidation can be greater than predicted.

5.9   Influence Zone

Whenever a foundation is loaded, a pressure (stress) increase occurs in the underlying soil immediately below the footing. Actually the pressure spreads laterally to a certain degree as well. The intensity of pressure decreases with depth until it eventually becomes too small and is of little concern.

- It is the pressure *increase* that causes settlement to occur in the soil below footings.

The increase in pressure extends to a greater depth below larger footings than smaller footings, hence the depth is influenced by the width of the footing (B). The zone where the pressure increase is significant with respect to settlement varies with the width of the footing. In clay, the zone is also influenced by the intensity of the effective overburden pressure (the pressure due to the effective weight of the soil lying above the point in question).

In granular soils, it is generally assumed that the zone extends to a depth of twice the footing width (2B) below the footing level. Some engineers however, prefer to use a depth equal to three times the width of the footing (3B). When calculating settlement, the average N value or lowest cumulative N value within this zone is used. The values are obtained during a soil test boring program. For compressible soils such as clay however, the pressure increase is considered significant until the pressure increase is less than 10% of the effective overburden pressure. The resulting depth below the footing calculated in this manner defines the height of the compressible layer (H) shown in Expression (5.3).

Example 5.1

If the footing discussed in Example 4.1 of the previous section was loaded to a pressure of 3,372 psf (161.45 kPa), is the settlement within tolerable ranges?

From Expression (5.1) with values expressed in SI units,

$S_i = qB^{0.7}I_c$ and $I_c = 1.71/N^{1.4}$

$I_c = 1.71 / (12)^{1.4}$, $I_c = 0.053$

$S_i = (161.45)(1.219)^{0.7}(0.053) = 9.83$ mm (approx. **3/8-inches**)

Thus the allowable bearing pressure is 3,372 psf. At this pressure approximately ⅜-inch of total settlement is expected, which is less than the 1-inch of total settlement criterion. This value also lies below the typical ¾-inch criterion for differential settlement.

6.0 LATERAL EARTH PRESSURE

6.1 Introduction

Lateral earth pressure represents pressures that are "to the side" (horizontal) rather than vertical. The objective of this section is to familiarize primarily the non-geotechnical engineer such as civil engineers, structural engineers, architects and landscape architects with simple background theory and considerations.

Calculating lateral earth pressure is necessary in order to design structures such as:

- Retaining Walls
- Bridge Abutments
- Bulkheads
- Temporary Earth Support Systems
- Basement Walls

6.2 Categories of Lateral Earth Pressure

There are three categories of lateral earth pressure and each depends upon the movement experienced by the vertical wall on which the pressure is acting. In this section, we will use the word *wall* to mean the vertical plane on which the earth pressure is acting. The wall could be a basement wall, retaining wall, earth support system such as sheet piling or soldier pile and lagging, etc.

The three categories are:

- *At rest* earth pressure
- *Active* earth pressure
- *Passive* earth pressure

The *at rest pressure* develops when the wall experiences no lateral movement. This typically occurs when the wall is restrained from movement such as a basement wall that is supported at the bottom by a slab and at the top by a floor framing system prior to placing soil backfill against the wall.

The *active pressure* develops when the wall is free to move outward such as a typical retaining wall and the soil mass stretches sufficiently to mobilize its shear strength. On the other hand, if the wall moves into the soil, then the soil mass is compressed sufficiently to mobilize its shear strength and the *passive pressure* develops. This situation might occur along the section of wall that is below grade and on the opposite side of the wall from the higher section. Some engineers use the passive pressure that develops along this buried face as additional restraint to lateral movement.
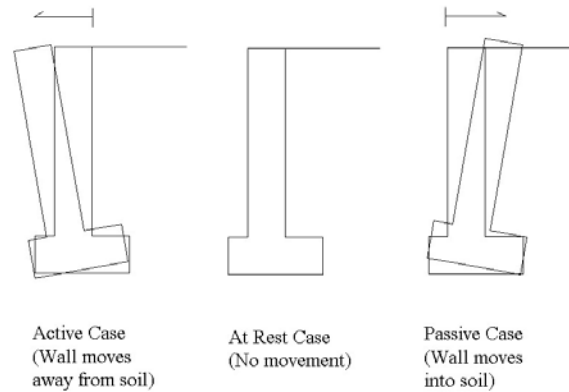
Figure 6.1 - Wall Movement

In order to develop the *full* active pressure or the *full* passive pressure, the wall has to move. If the wall does not move a sufficient amount, then the full pressure will not develop. If the full active pressure does not develop behind a wall, then the pressure will be higher than the expected active pressure. Likewise, significant movement is necessary to mobilize the full passive pressure. This is illustrated in Figure 6.2. Note that the *at rest* condition is shown where the wall rotation is equal to 0, which is the condition for zero lateral strain.
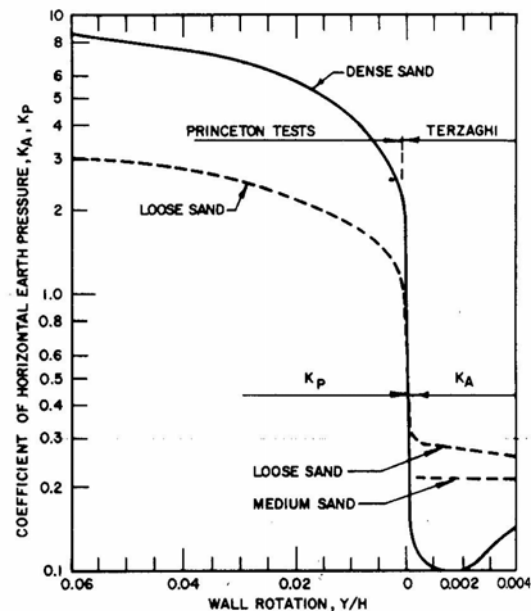

Figure 6.2 Effect of Wall Movement on Wall Pressure
[Ref: NAVFAC DM-7]

This figure shows that:

- As the wall moves away from the soil backfill (left side of Figure 6.1), the active condition develops and the lateral pressure against the wall *decreases* with wall movement until the *minimum* active earth pressure force (Pa) is reached.

- As the wall moves toward (into) the soil backfill (right side of Figure 6.1), the passive condition develops and the lateral pressure against the wall *increases* with wall movement until the *maximum* passive earth pressure (Pp) is reached.

Thus the intensity of the active/passive horizontal pressure, which is a function of the applicable earth pressure coefficient, depends on *wall movement* as the movement controls the degree of shear strength mobilized in the surrounding soil.

6.3   Calculating Lateral Earth Pressure Coefficients

Lateral earth pressure is related to the vertical earth pressure by a coefficient termed the:

- At Rest Earth Pressure Coefficient (Ko), or

- Active Earth Pressure Coefficient (Ka), or

- Passive Earth Pressure Coefficient (Kp)

The lateral earth pressure is equal to vertical earth pressure times the appropriate earth pressure coefficient. There are published relationships, tables and charts for calculating or selecting the appropriate earth pressure coefficient.

Since soil backfill is typically granular material such as sand, silty sand, or sand with gravel, this section assumes that the backfill material against the wall is coarse-grained, non-cohesive material. Thus, cohesive soil such as clay is not discussed. However, there are many textbooks and other publications where this topic is fully discussed.

6.4   At Rest Coefficient

Depending upon whether the soil is loose sand, dense sand, normally consolidated clay or over consolidated clay, there are published relationships that depend upon the soil's engineering values for calculating the at rest earth pressure coefficient. One common earth pressure coefficient for the "at rest" condition used with granular soil is:

$$Ko = 1 - \sin(\phi) \qquad (6.1)$$

Where: Ko is the "at rest" earth pressure coefficient and ϕ is the soil friction value.

6.5   Active and Passive Earth Pressure Coefficients

When discussing active and passive lateral earth pressures, there are two relatively simple classical theories (among others) that are widely used:

- Rankine Earth Pressure

- Coulomb Earth Pressure

The Rankine Theory assumes:

- There is no adhesion or friction between the wall and soil

- Lateral pressure is limited to vertical walls

- Failure (in the backfill) occurs as a sliding wedge along an assumed failure plane defined by ϕ.

- Lateral pressure varies linearly with depth and the resultant pressure is located one-third of the height (H) above the base of the wall.

- The resultant force is parallel to the backfill surface.

The Coulomb Theory is similar to Rankine except that:

- There is friction between the wall and soil and takes this into account by using a soil-wall friction angle of δ. Note that δ ranges from $\phi/2$ to $2\phi/3$ and $\delta = 2\phi/3$ is commonly used.

- Lateral pressure is *not* limited to vertical walls.

- The resultant force is not necessarily parallel to the backfill surface because of the soil-wall friction value δ.

The general cases for calculating the earth pressure coefficients can also be found in published expressions, tables and charts for the various conditions such as wall friction and sloping backfill. The reader should obtain these coefficients for conditions other than those discussed herein.

The Rankine active and passive earth pressure coefficients for the condition of a *horizontal* backfill surface are calculated as follows:

$$\text{(Active)} \quad Ka = (1 - \sin(\phi)) / (1 + \sin(\phi)) \qquad (6.2)$$

$$\text{(Passive)} \quad Kp = (1 + \sin(\phi)) / (1 - \sin(\phi)) \qquad (6.3)$$

Tabulated values based on Expression (6.2) and (6.3) are shown in Table 6.1 below.

Table 6.1 – Tabulated Values

φ (deg)	Rankine Ka	Rankine Kp
28	.361	2.77
30	.333	3.00
32	.313	3.19

The *Coulomb active and passive earth pressure coefficients* are more complicated expressions that depend on the angle of the back of the wall, the soil-wall friction value and the angle of backfill. Although the expressions are not shown, these values are readily obtained in textbook tables or by programmed computers and calculators. Tables 6.2 and 6.3 show some tabulated values of the Coulomb active and passive earth pressure coefficients for the specific case of a *vertical* back of wall angle and *horizontal* backfill surface.

Table 6.2 - Coulomb Active Pressure Coefficient

	δ (deg)				
φ (deg)	0	5	10	15	20
28	.3610	.3448	.3330	.3251	.3203
30	.3333	.3189	.3085	.3014	.2973
32	.3073	.2945	.2853	.2791	.2755

Table 6.3 - Coulomb Passive Pressure Coefficient

	δ (deg)				
φ (deg)	0	5	10	15	20
30	3.000	3.506	4.143	4.977	6.105
35	3.690	4.390	5.310	6.854	8.324

Some points to consider are:

- For the Coulomb case shown above with no soil-wall friction (i.e. $\delta = 0$) and a horizontal backfill surface, both the Coulomb and Rankine methods yield equal results.

- As the soil becomes stronger the friction value (ϕ) increases. The active pressure coefficient decreases resulting in a decrease in the active force, and the passive pressure coefficient increases resulting in an increase in the passive force.

- As the soil increases in strength (i.e. friction value increases), there is less horizontal pressure on the wall in the active case.

6.6   Calculating the Vertical Effective Overburden Pressure

The vertical effective overburden pressure is the effective weight of soil above the point under consideration. The term *effective* means that the submerged unit weight of soil is used when calculating the pressure below the groundwater level. For instance, assume that a soil has a total unit weight (γ) of 120 pcf and the groundwater level is 5 feet below the ground surface. The vertical effective overburden pressure ($\sigma v'$) at a depth of 10 feet below the ground surface (i.e. 5 feet below the groundwater depth) is:

$$\sigma v' = 5(\gamma) + 5(\gamma')$$

Where γ is the total unit weight of the soil and γ' is the effective (or submerged) unit weight of the soil which equals the total unit weight of soil minus the unit weight of water (i.e. 62.4 pcf). Thus:

$$\sigma v' = 5(120) + 5(120\text{-}62.4) = 888 \text{ psf}$$

6.7   Calculating the Lateral Earth Pressure

There is a relationship between the vertical effective overburden pressure and the lateral earth pressure. The lateral earth pressure (σ) is:

$$\sigma a = Ka\ (\sigma v') \text{ Active lateral earth pressure} \qquad (6.4)$$

$$\sigma p = Kp\ (\sigma v') \text{ Passive lateral earth pressure} \qquad (6.5)$$

where ($\sigma v'$) is the vertical effective overburden pressure.

If water pressure is allowed to build up behind a retaining wall, then the total pressure and the resulting total force along the back of the wall are increased considerably. Therefore, it is common for walls to be designed with adequate drainage to prevent water from accumulating behind the wall. Thus, weepholes, lateral drains or blanket drains along with granular soil (freely draining backfill) are commonly used behind retaining walls. In the case of a drained condition, the total unit weight of soil (γ) is used behind the full height of the wall and there is no water pressure contribution.

An example of an earth pressure calculation using the Rankine active earth pressure coefficient is shown later as Example 6.1. A similar calculation can be performed for the Coulomb case by using the Coulomb earth pressure coefficient applicable to the case at hand.

6.8   Calculating the Total Lateral Earth Pressure Force

The total lateral force is the area of the pressure diagram. In the simple example shown later in this course, the area of the earth pressure diagram is the earth pressure at the

bottom of the wall (KaγH) times the height of the wall (H) times one-half (1/2), since the pressure distribution increases linearly with depth creating a triangular shape. Thus the total active earth pressure force (Pa) acting along the back of the wall is the area of the pressure diagram expressed as:

$$Pa = \frac{1}{2} Ka \, \gamma \, H^2 \quad (6.6.1)$$

The total passive earth pressure force is:

$$Pp = \frac{1}{2} Kp \, \gamma \, H^2 \quad (6.6.2)$$

The total force acts along the back of the wall at a height of H/3 from the base of the wall.

In more complicated cases, the earth pressure distribution diagram is drawn and the total force is calculated by determining the area of the pressure diagram. The location of the resultant force is also determined.

6.9   Other Forces Acting on the Wall

Aside from the earth pressure force acting on the wall, other forces might also act on the wall. These forces include:

- Surcharge load

- Earthquake load

- Water Pressure

6.10   Surcharge Load

A surcharge load results from forces that are applied along the surface of the backfill behind the wall. These forces apply an additional lateral force on the back of the wall. Surcharge pressures result from loads such as a line load, strip load, embankment load, traffic (such as a parking lot), floor loads and temporary loads such as construction traffic. Generally, elastic theory is used to determine the lateral pressure due to the surcharge and these methods have been extensively published.

In the case of a uniform surcharge pressure (q) taken over a wide area behind the wall, the lateral pressure due to the uniform surcharge is:

$$K_0 q \qquad (6.7)$$

Where K_0 is the applicable at rest active or passive pressure coefficient. The pressure diagram behind the wall for a uniform surcharge is rectangular and acts at a height of H/2

above the base of the wall. Thus, the additional lateral force (Ps) acting behind the wall resulting from a uniform surcharge is the area of the rectangle, or:

$$Ps = K_0qH \qquad (6.8)$$

Whether the total surcharge load is calculated from elastic theory or as shown in Expression (6.8), the force (pressure) is superimposed onto the calculated lateral earth pressure.

6.11   Earthquake Force

Additional lateral loads resulting from an earthquake are also superimposed onto the lateral earth pressure where required. Publications such as <u>AASHTO Standard Specifications for Highway Bridges</u> and other textbooks provide methods for calculating the earthquake force.

6.12   Water Pressure

Walls are typically designed to prevent hydrostatic pressure from developing behind the wall. Therefore the loads applied to most walls will not include water pressure. In cases where water pressure might develop behind an undrained wall, the additional force resulting from the water pressure must be superimposed onto the lateral earth pressure. Since water pressure is equal in all directions (i.e. coefficient (K) = 1), the water pressure distribution increases linearly with depth at a rate of γ_wz where γ_w is the unit weight of water (62.4 pcf) and z is the depth below the groundwater level. If the surface of water behind a 10-foot high wall (H) were located 5 feet (d) below the backfill surface, then the superimposed total <u>lateral force</u> resulting from groundwater pressure would be:

- W = ½ $(\gamma_w)(H\text{-}d)^2$ = 780 pounds, which is the area of the linearly increasing pressure distribution.

- W acts at a height of (H-d)/3 (or 1.67-ft) above the base of the wall.

- Note that the earth pressure would be calculated using the submerged unit weight of soil γ' below the groundwater level.

If seepage occurs, then the water pressure must be derived from seepage analysis, which is outside the scope of this course.

6.13   Compaction

If heavy rollers are used to compact soil adjacent to walls, then high residual pressures can develop against the wall. Although a reasonable amount of backfill compaction is necessary, excess compaction should be avoided.

6.14 Building Codes

Building codes also provide information related to earth pressure and calculating lateral soil load. The topic *Lateral Soil Loads* is included in The BOCA National Building Code.

6.15 Important Points

Some important points to consider are:

- Lateral earth pressure acts to the side and is a function of the vertical effective soil overburden pressure and the applicable earth pressure coefficient.

- There are three categories of earth pressure; each dependant upon magnitude and direction of wall movement. These categories are: At Rest, Active and Passive.

- Two classical earth pressure theories in common use are Rankine and Coulomb.

- In addition to earth pressure, other common superimposed lateral pressures result from: surcharge, earthquake, and water.

- The total lateral force equals the area of the pressure distribution along the back of the wall.

Example 6.1

Use the Rankine method to calculate the total **active** lateral force and location of the forces behind a 10-foot high vertical wall. Assume that the soil has a total unit weight of 120 pcf and a friction value of 32 degrees. Assume that there is a uniform surcharge of 100 psf located along the surface behind the wall. Groundwater is well below the depth of the foundation so that groundwater pressure does not develop behind the wall.
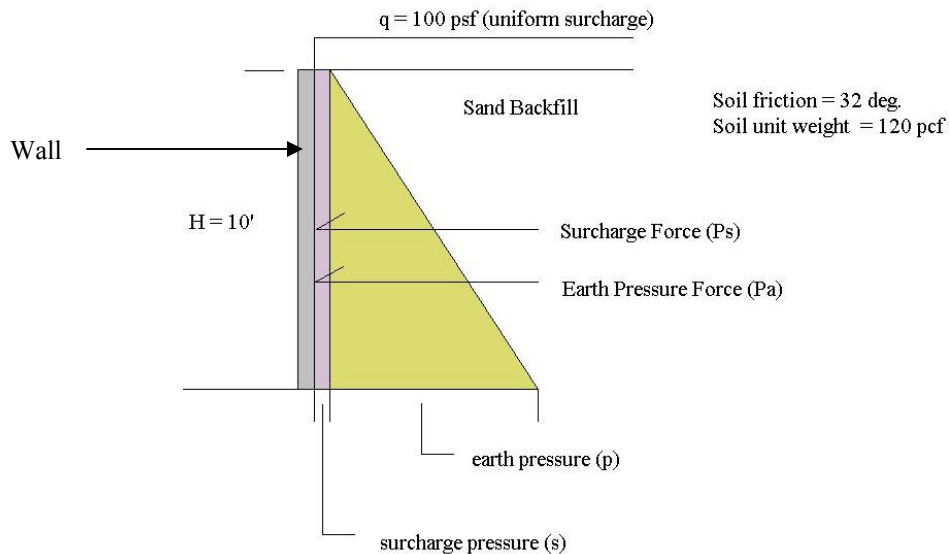
$K_a = 1 - \sin(32) / 1 + \sin(32) = 0.313$ is the Active Earth pressure Coefficient

At bottom of wall (surcharge pressure) $s = K_a(q) = 0.313(100) = 31.3$ psf

At bottom of wall (active lateral earth pressure) $p_a = K_a(\gamma) H = 0.313(120)(10) = 375.6$ psf

Total Surcharge Force: $P_s = K_a(q)H = 313$ pounds and acts at a height of H/2 from the base of the wall.

Total Earth Pressure Force: $P_a = \frac{1}{2} K_a(\gamma) H^2 = \frac{1}{2} (0.313)(120)(10)^2 = 1878$ pounds and act at a height of H/3 from the base of the wall.

Total Active Force $= 1878 + 313 = 2191$ pounds

7.0   RETAINING WALLS

7.1   Introduction

Retaining walls are structures that support backfill and allow for a change of grade. For instance, a retaining wall can be used to retain fill along a slope or it can be used to support a cut into a slope.
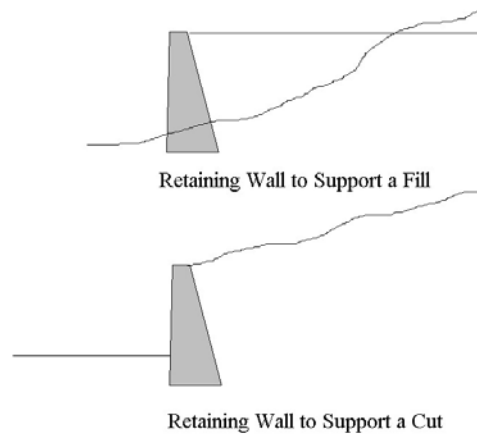

Figure 7.1 – Example of Retaining Walls

Retaining wall structures can be gravity type structures, semi-gravity type structures, cantilever type structures, and counterfort type structures. Walls might be constructed from materials such as fieldstone, reinforced concrete, gabions, reinforced earth, steel and timber. Each of these walls must be designed to resist the *external forces* applied to the wall from earth pressure, surcharge load, water, earthquake etc.

7.2   Calculating the Total Active Earth Pressure Force

The total lateral force is the area of the pressure diagram acting on the wall surface. The examples in this section assume drained conditions and a homogeneous granular soil backfill behind the wall, which results in a simple triangular distribution. Although this is a common case, the pressure diagram can become more complicated depending upon actual soil conditions that might have different values.

With the Coulomb method, the active force acts directly on the wall and friction develops between the soil and wall. With the Rankine method however, wall friction is ignored and the active force acts directly on a vertical face extending through the heel of the wall. If the back of the wall were vertical, then the force acts on the wall. On the other hand, if the back of the wall were sloping, then the force acts on the vertical soil plane as illustrated in Figure 7.2.

In the example shown later in this section, the area of the earth pressure diagram is the earth pressure at the bottom of the wall (KaγH) times the height of the wall (H) times one-half (1/2) since the pressure distribution increases linearly with depth creating a triangular shape. Thus the total active earth pressure force (Pa) acting along the back of the wall is the area of the pressure diagram expressed as:

$$Pa = \frac{1}{2} \ Ka \ \gamma \ H^2 \quad (7.1)$$

The total force acts along the back of the wall at a height of H/3 from the base of the wall. So far we have not stated whether this is the Rankine or Coulomb Case. The calculation for the active earth pressure force (Pa) is the same provided that the appropriate earth pressure coefficient (Ka) is used. Selecting whether the Rankine method or Coulomb method will be used is usually a matter of choice or convention.

The example shown in Figure 7.2 relates specifically to a wall supporting a horizontal backfill. Thus the active earth pressure coefficient (Ka) can be derived directly from Expression (6.2) or Table 6.1 shown in the previous section. For the case of a sloping backfill and other wall geometries, the reader should refer to the published references.

This example assumes that a 9-foot high gravity type retaining structure supports soil backfill having a total unit weight of 125 pcf. Groundwater is well below the structure and the backfill material is freely draining. The backfill soil has an angle of internal friction (φ) of 32 degrees and the backfill surface behind the wall is horizontal. Both the Rankine and Coulomb earth pressure forces are shown.

Note that the location and direction of the active forces follows the assumptions stated above for the Rankine and Coulomb Theory. Although the back of the wall has an angle of 80 degrees, the Rankine force acts along a vertical plane beginning at the heel of the wall while the Coulomb force acts directly along the back of the wall. Since the Rankine Theory assumes that there is no soil-wall friction, the force (Pa) is parallel to the backfill surface. On the other hand, since the Coulomb Theory takes the soil-wall friction into consideration, the force (Pa) acts at an angle of δ from the perpendicular to the wall. This results in both a vertical and horizontal component of the force (Pa). The Rankine method will also produce a vertical and horizontal component of the force (Pa) if the backfill surface has a slope.

In each case, the resultant force Pa acts at a height of H/3 from the base of the wall where H is the height of the wall for the simple case illustrated herein. If the pressure diagram were more complicated due to differing soil conditions, for instance, then the location of the force (Pa) will change. In all cases however, the resultant of the force (Pa) is located at the centroid of the combined mass area.
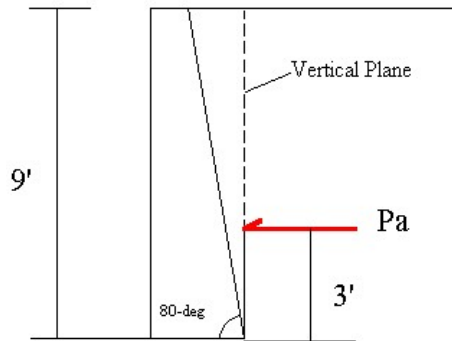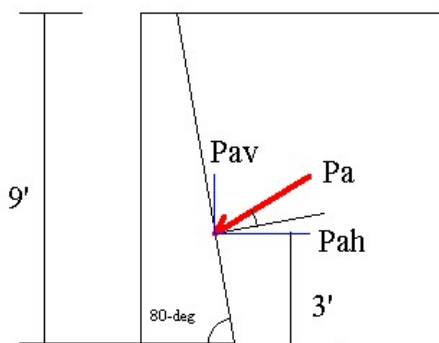
$Ka = (1 − \sin (\phi)) / (1 + \sin (\phi)) = 0.307$

$Pa = \frac{1}{2} \; Ka \; \gamma \; H^2 = (0.5)(0.307)(125)(9^2)$

$Pa = 1554.2$ pounds

Rankine Active Earth Pressure Force

$Ka = 0.3545$ from Table 6.1 for conditions stated

$Pa = \frac{1}{2} \; Ka \; \gamma \; H^2 = (0.5)(0.354)(125)(9^2)$

$Pa = 1792.1$ pounds

Calculate horizontal and vertical components of Pa where Pa acts 31.3 deg from the horizontal.

$Pa_h = Pa \cos (31.3) = 1531.3$ pounds

$Pa_v = Pa \sin (31.3) = 931.0$ pounds

Coulomb Active Earth Pressure Force

Figure 7.2 - Calculation of Earth Pressure Force for a Homogeneous Cohesionless Backfill

7.3 Other Forces Acting on the Wall

Aside from the earth pressure force acting on wall, other forces might also act on the wall. Although these forces are not discussed in this course, they might include:

- Surcharge load

- Earthquake load

- Water Pressure

These additional forces would be superimposed onto the earth pressure force to yield the total lateral force.

7.4 Factors of Safety

Retaining wall design is an iterative process. An initial geometry is assigned to the wall and the appropriate forces are calculated. The actual forces are then checked using

appropriate factors of safety and the geometry is revised until satisfactory factors of safety are reached. There are common dimensions that are available that can be used as a first cut.

7.5 Proportioning Walls

In order to achieve stability, retaining walls are usually proportioned so that the width of the base (B) is equal to approximately 0.5 to 0.7 times the height of the wall (H). Thus, a 9-foot high wall would have a base approximately 4.5 feet to 6.3 feet wide which provides a convenient starting point.

7.6 Sliding

A retaining structure has a tendency to move away from the backfill surface because of the horizontal driving forces resulting from the soil backfill and other forces such as surcharge. Generally, the wall resists sliding by the frictional resistance developed between the foundation of the wall and foundation soil. Although other horizontal forces act opposite to the driving force such as passive soil pressure in the fill in front of the wall, it is often ignored.

The factor of safety with respect to sliding equals the *resisting force* divided by the *driving force* as shown in Expression (7.2). A minimum factor of safety of 1.5 is desirable to resist sliding assuming that passive resistance from any fill in front of the wall is ignored. This is a common assumption and avoids relying on the presence of soil in front of the wall for additional resistance.

$$FS_s = \Sigma V \tan(k\phi_1) / Pa_h \qquad (7.2)$$

ΣV is the total vertical force, Pa_h is the horizontal active earth pressure force and $\tan(k\phi_1)$ is the coefficient of friction between the base of the wall and the soil. The factor "k" ranges from ½ to ⅔ and ϕ_1 is the friction angle of the foundation soil. Friction factors between dissimilar materials can also be found in publications such as <u>NAVFAC Design Manual 7.2</u>. Expression (7.2) assumes that the soil below the wall is a cohesionless material such as sand without any cohesive strength. Therefore, there is no additional resistance due to cohesion.

7.7 Overturning

A retaining structure also has a tendency to rotate outward around the toe of the wall. The moment resulting from the earth pressure force (as well as other lateral forces such as surcharge) must be resisted by the moments resulting from the vertical forces produced by the wall including any vertical component (Pa_v) of the earth pressure force. Thus, the factor of safety with respect to overturning is the *resisting moment* divided by the *overturning moment* as shown in Expression (7.3). A minimum factor of safety of 2 to 3 is desirable to resist overturning.

$$FS_o = \Sigma Mr / \Sigma Mo \quad (7.3)$$

Where ΣMr is the sum of the resisting moments around the toe of the wall and ΣMo is the sum of the overturning moments around the toe of the wall.

7.8   Bearing Capacity below Retaining Walls

As with any structure, the bearing capacity of the soil must be adequate to safely support the structure. The ultimate bearing capacity of the foundation soil (q_u) is calculated using theoretical bearing capacity methods presented in textbooks and other published resources.

The resultant of all forces acting along the base of the wall from earth pressure and the weight of the wall results in a non-uniform pressure below the base of the wall with the greatest pressure below the toe of the base and the least pressure below the heel of the base.

The maximum and minimum pressures below the base of the wall (B) are:

$$q_{max} = (\Sigma V / B)(1 + 6e / B) \quad (7.4.1)$$

$$q_{min} = (\Sigma V / B)(1 - 6e / B) \quad (7.4.2)$$

$$\text{Where } e = \text{eccentricity; } e = (B / 2) - (\Sigma Mr - \Sigma Mo) / \Sigma V \quad (7.5)$$

The factor of safety with respect to bearing capacity is shown in Expression (7.6). Generally, a factor of safety of 3 is required.

$$FS_{bc} = q_u / q_{max} \quad (7.6)$$

Eccentricity is an important consideration when proportioning the walls. Consider the eccentricity (e) in relationship to the minimum pressure (q_{min}). Substituting for (e) in Expression (7.4.2):

$$\text{If } e = B / 6 \text{ then } q_{min} = (\Sigma V / B)(1 - 6e / B) = 0 \quad (7.7.1)$$

$$\text{If } e < B / 6 \text{ then } q_{min} = (\Sigma V / B)(1 - 6e / B) > 0 \quad (7.7.2)$$

$$\text{If } e > B / 6 \text{ then } q_{min} = (\Sigma V / B)(1 - 6e / B) < 0 \quad (7.7.3)$$

Expressions (7.7.1) and (7.7.2) give acceptable results since the pressure at the heel is zero or greater (positive). Thus the entire base lies in contact with the soil. If Expression (7.7.3) was true, then the pressure at the heel is negative indicating the heel of the base is

tending toward lifting off the soil, which is unacceptable. If this condition occurs, then the wall must be re-proportioned.

7.9 Other Considerations

Before a wall design is complete, the settlement of the wall and the global stability of the entire mass on which the wall is supported must be checked. Settlement must lie within tolerable ranges and global stability, such as from slope stability calculations, must be adequate.

7.10 Important Points

Some important points to consider are:

- The Rankine and Coulomb methods are commonly used to calculate the active earth pressure force. The discussion in this course is limited to granular (cohesionless) backfill soil, which is a typical condition relating to retaining walls.

- The active earth pressure force (Pa) is a function of the earth pressure coefficient (Ka), the unit weight of the soil and the height of the wall.

- Wall movement must occur in order to develop the full active earth pressure force.

- Other lateral forces are superimposed on the lateral earth pressure force to derive the total lateral force.

- Retaining wall design is iterative and seeks to provide wall geometry that produces suitable factors of safety for sliding, overturning and bearing capacity.

- Retaining walls must also be checked for tolerable settlement and global stability

Example 7.1

The following example illustrates the discussion presented in this section.

Using the <u>Rankine</u> method of analysis, calculate the factors of safety with respect to sliding, overturning and bearing capacity. Use the values presented in the following Table 7.1 and refer to the figure below. It is inferred that all calculations relate to a unit length of wall.

Table 7.1

Friction angle of soil backfill (ϕ)	32 degrees
Soil Backfill Unit Weight (γ)	125 pcf
Friction angle of the foundation soil (ϕ_1)	33 degrees
Rankine active pressure coefficient (Ka)	0.307
Concrete Unit Weight (γ_c)	150 pcf
Dimensions of the concrete wall section 1	1-ft by 8-ft
Dimensions of the soil backfill section 2	4-ft by 8-ft
Dimensions of the concrete wall section 3	6-ft by 1-ft
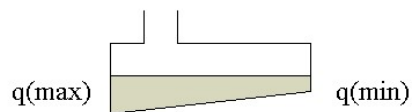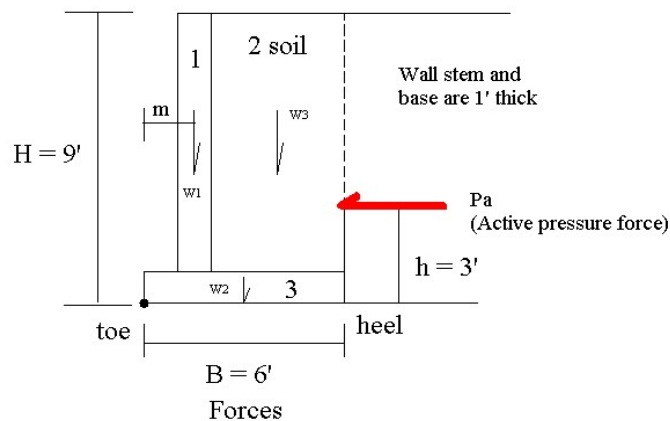
Since Pa is horizontal, there is no vertical component of the force. If the backfill surface was sloping, then Pa would slope at an angle parallel to the backfill slope. In this case there would be both a vertical and horizontal component of Pa. The lateral thrust would be the horizontal component and the vertical component would be an additional vertical force included in ΣV.

Calculate the values shown in Table 7.2. The dimensions for "Area" relate to each of the three sections identified in the above figure. The unit weight (γ) is provided for the concrete wall and soil backfill over the base of the wall. W is the weight of each section and it acts at the centroid of the mass area as shown in the figure above. The value "m"

is the moment arm measured from the toe to the location of the individual W values. M is the resisting moment for each of the individual areas.

Table 7.2 – Table of Values

Section	Area (sf)	γ (pcf)	W (lbs)	m (ft)	M (ft-lb)
1	1 x 8	150	1200	1.5	1800
2	4 x 8	125	4000	4	16000
3	6 x 1	150	900	3	2700
		$\Sigma V = \textbf{6100}$		$\Sigma Mr = \textbf{20500}$	

$Pa = \frac{1}{2} \; Ka \; \gamma H^2 = (0.5) \, (0.307) \, (125) \, (81) = \underline{1554.2 \text{ lbs}}$

$\Sigma Mo = Pa \, (h) = (1554.2) \, (3) = \underline{4662.6 \text{ ft-lbs}}$

Overturning: $Fs_o = \Sigma Mr \, / \, \Sigma Mo = 20500 \, / \, 4662.6 = \textbf{4.4} > 2 \textbf{ OK}$

Sliding: $FS_s = \Sigma V \tan(k\phi_1) \, / \, Pa_h = (6100) \tan (22) \, / \, 1554.2 = \textbf{1.58} > 1.5 \textbf{ OK}$
Where $k = 2/3$

Bearing Capacity:

Assume that the ultimate bearing capacity of the foundation soil is 5000 psf.

$e = (B \, / \, 2) \, - \, (\Sigma Mr - \Sigma Mo) \, / \, \Sigma V = (6 \, / \, 2) - (20500 - 4662.6) \, / \, 6100 = \textbf{0.4}$ (i.e. $e < B \, / \, 6$)

$q_{max} = (\Sigma V \, / \, B) \, (1 + 6e \, / \, B) = (6100 \, / \, 6) \, (\, 1 + 2.4 \, / \, 6) = (1016.6) \, (1.4) = \underline{\textbf{1423.4 psf}}$

$q_{min} = (\Sigma V \, / \, B) \, (1 - 6e \, / \, B) = (6100 \, / \, 6) \, (\, 1 - 2.4 \, / \, 6) = (1016.6) \, (.6) = \underline{\textbf{610 psf}}$ (i.e. base of wall is in full soil contact)

$FS_{bc} = q_u \, / \, q_{max} = 5000 \, / \, 1423.4 = \textbf{3.5} > 3.0 \textbf{ OK}$

Example 7.2

For low retaining walls, a solution using the equivalent fluid pressure might be satisfactory and obtained from a graphical solution. The equivalent fluid pressure is derived from Figure 7.3 and requires knowledge of the soil backfill. This figure is presented for illustration only and the reader should refer to the referenced publication for details.
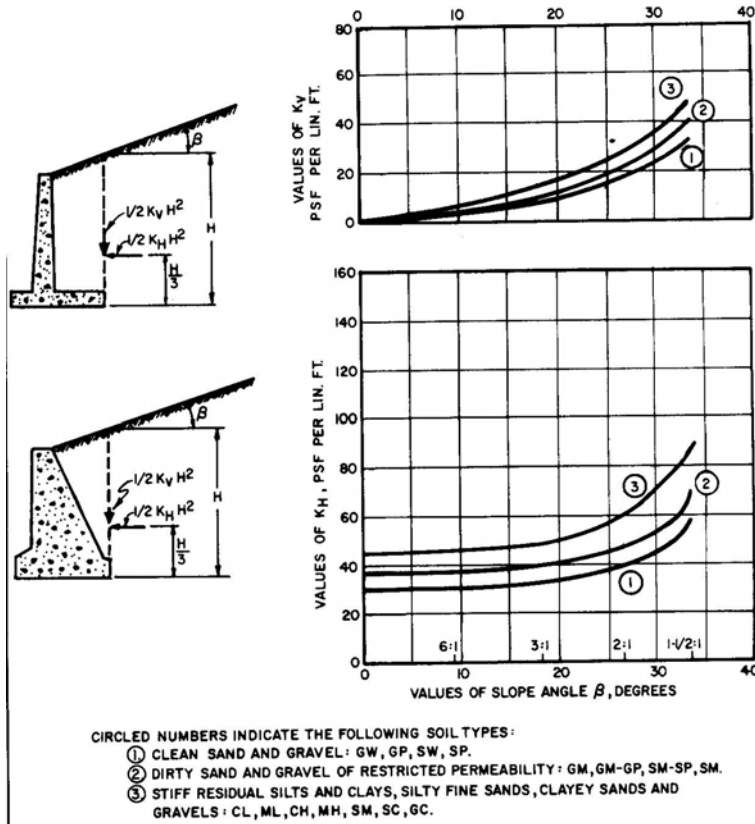

Figure 7.3 – Design Loads for Low Retaining Walls
[Ref: NAVFAC DM 7.2]

8.0 PILE FOUNDATIONS

8.1 Introduction

When the depth to acceptable bearing material lies deep, then deep foundations can be used to support structures. Deep foundations will be used when it becomes more economical to use them rather than lower the footings or remove and replace the unsuitable bearing material. This is a generalization and there might be other factors to consider.

Deep foundations, such as piles, are structural members that carry the design loads through unsuitable soil so that the foundations bear on underlying soil capable of supporting the required load. Piles are long, slender structural members typically constructed of timber, steel, concrete or a combination of steel and concrete. Piles are installed by driving or drilling the member to a required depth or resistance.
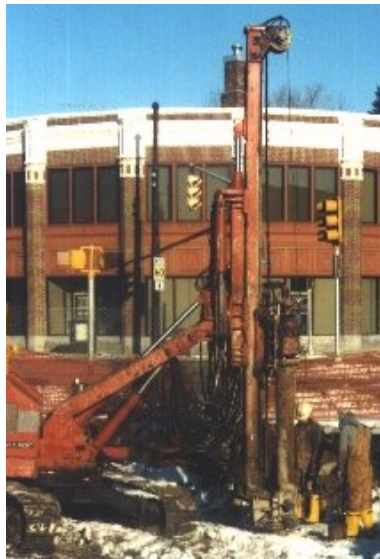
Photograph 8.1 - Drilling to Install a Small Diameter Grouted Pile

8.2 Piles

8.2.1 Types of Piles

Piles come in various shapes and are formed out of various materials. Common pile types are presented below. The information is for guidance and there can be circumstances where the length or load will lie outside the range given.

Timber Pile

Timber piles are a low capacity pile usually limited to a load of 35 tons. The piling material consists of Southern Yellow Pine or Douglas Fir. Timber piles are considered for lengths between 30 feet to 60 feet. The piles are vulnerable to decay unless they are kept below groundwater or treated, and they can be damaged in hard driving.

Steel H Section Pile

Steel H piles are high strength piles considered for loads ranging between 40 tons to 120 tons. The capacity is reduced to allow for corrosion. The piles are generally driven to lengths ranging from 40 feet to 100 feet. They are a low displacement pile and are easily handled, spliced and cut off.

Pipe Pile

Steel pipe piles can be driven open ended or closed ended. These piles are commonly filled with concrete although they can be left unfilled. In some cases, concrete filled piles can achieve a working load capacity of 500 tons using an H pile core. Typically this pile is considered for loads ranging between 80 tons to 120 tons without H pile cores. The pile is a displacement pile, unlike the steel H pile. Corrosion is also a consideration.

Precast Prestressed Concrete Pile

Precast piles are most often prestressed to withstand handling and driving stresses. The piles can reach a capacity of 250 tons and typically the length of pile ranges between 60 feet to 100 feet for the prestressed section. The pile is a displacement pile and splicing is difficult.

Cast-in-Place Mandrel-Driven Pile

Mandrel-driven piles are thin steel shells driven into the ground with a mandrel and then filled with concrete. The pile is driven from the bottom and the steel casing is pulled along. The length of pile is limited by the length of the mandrel and typically in the range of 50 feet to 80 feet. The mandrel-driven pile can achieve a load capacity of up to 100 tons. The pile is a displacement pile.

Pressure Injected Footing

Pressure injected footings (PIFs) are a cast-in-place concrete pile formed by driving a steel casing fitted with a dry concrete plug. At the required pile depth the plug is ejected from the casing by additional driving thus forming a concrete bulb. The casing is withdrawn and a thin steel shell filled with concrete extends from the bulb to pile cut off to form the shaft of the pile. PIFs typically range from 10 feet to 60 feet long and support loads ranging from approximately 60 tons to 120 tons.

Helical Pier

Helical piers are a proprietary item that consists of steel helical plates fitted on a lead section steel shaft. The lead section is screwed into the ground until it achieves a design torque which equates with ultimate bearing capacity. As the lead section advances, additional steel extensions are fastened to achieve depth. Under certain circumstances, according to the manufacturer, the pier can achieve high capacity.

Small Diameter Grouted Pile

Small diameter grouted piles are a low displacement pile installed by drilling to a specified depth and filling the hole with grout. A steel section such as reinforcing steel is inserted in the grout and extends from the tip of the pile to cut off. The typical diameter of this pile ranges from 6 inches to 10 inches. Like the helical pier, the pile is installed by drilling rather than driving.

8.2.2 Point Bearing and Friction Piles

Piles can derive their support through a combination of end-bearing and friction (Figure 8.1a), end bearing (Figure 8.1b) or friction (Figure 8.1c). End bearing piles derive most if not all of their support at the tip of the pile if the pile is driven to bedrock or bear several feet into a strong soil layer. Friction piles derive most if not all of their support from the friction or adhesion between the pile and the material surrounding the shaft of the pile.

Often any contribution to load derived by the weak soil surrounding the shaft is ignored especially if the material is peat or organic silt. However, it is assumed that the weak soil shown surrounding the friction pile is capable of developing the required capacity. The ultimate load capacity of the pile is Qu.
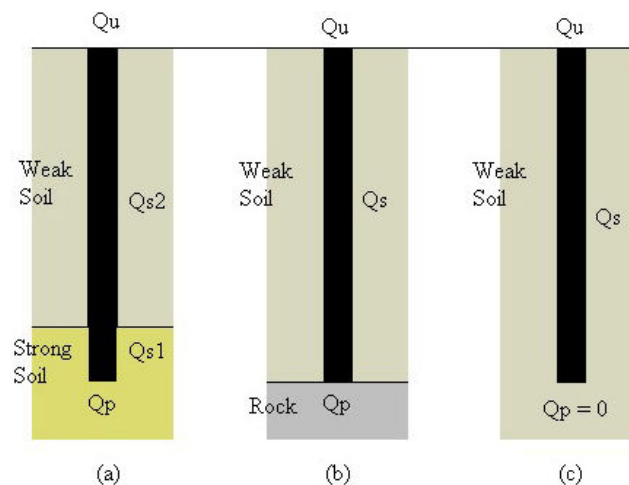

Figure 8.1 – Pile Capacity

8.2.3   Pile Capacity

The allowable capacity of a pile is based on two factors:

- Structural considerations - the allowable capacity of the structural member.

- Geotechnical considerations - the allowable capacity of the bearing material (i.e. soil, rock)

Structural Capacity of Piles

The structural capacity of a pile is determined by applying the applicable allowable stress of the material to the applicable area of the material as shown in Table 8.1.

Table 8.1

Steel Pile:	$Qall = (As)(fs)$
Cased Concrete Pile:	$Qall = (As)(fs) + (Ac)(fc)$
Uncased Concrete Pile:	$Qall = (Ac)(fc)$
Timber Pile:	$Qall = (Ap)(fw)$

Where:

Table 8.2

Qall =	Allowable structural capacity
As =	Area of cross section of steel
Ac =	Area of cross section of concrete
Ap =	Average area of cross section of timber
*f*s =	Allowable stress of steel
*f*c =	Allowable stress of concrete
*f*w =	Allowable stress of timber

Geotechnical Capacity of Piles

In the simplest of terms, the ultimate load-carrying capacity of a pile (Qu) is the sum of the point resistance (Qp) plus the skin friction derived from the soil-pile interface (Qs).

$$Qu = Qp + Qs \qquad (8.1)$$

There are a number of different published methods for estimating the values of Qp and Qs and it is beyond the scope of this text to provide a discussion. The reader is encouraged to consult one of the many text books and other publications for additional detailed information. However, the following rudimentary information is provided.

The point resistance of a pile is similar to the capacity of a shallow footing except that the pile extends much deeper. The ultimate resistance can be expressed in a form similar to the form used for footings although changes will be made for the value of the bearing capacity factors Nc and Nq. Since the "width" of a pile B is relatively small, the term gBNg is dropped without serious error. Therefore, the unit point resistance of a pile is:

$$q_{tip} = cNc* + q'Nq* \qquad (8.2)$$

The pile resistance at the tip (bottom of the pile) is:

$$Qp = A_{tip} (cNc* + q'Nq*) \qquad (8.3)$$

The skin resistance of the pile is the area of the pile shaft times the unit skin resistance.

$$Qs = \Sigma(p)(\Delta L)f \qquad (8.4)$$

Where:

p = perimeter of the pile section
ΔL = incremental pile length over which p and f are taken as constant
f = unit shaft resistance at any depth

Depending upon specific loading conditions, piles might also be required to resist uplift (tension) loads and lateral loads. Design procedures are available for estimating the uplift and lateral capacity of piles.

Again it is emphasized that this course presents a simplified discussion. Important factors to consider include the following:

- The point bearing capacity of pile in sand increases with depth of embedment in the bearing stratum but reaches a maximum value at a critical embedment.

- The unit frictional resistance in sand is difficult to estimate. The value increases with depth but then remains constant after reaching a critical depth.

- The type of pile (displacement, non-displacement) and installation methods (jetted, driven) affect the unit skin friction value.

- The skin resistance of piles in clay is also difficult to estimate.

8.2.4   Pile Settlement

The settlement of a pile under a vertical working load is the sum of three individual components. These components include the elastic movement of the pile (Se), the settlement of the pile caused by the load at the tip (St), and the settlement of the pile caused by the load transmitted along the pile shaft (Ss).

Thus the total expected pile settlement (S) is:

$$S = Se + St + Ss \qquad (8.5)$$

Theoretical methods are available to calculate these terms.

Settlement is calculated for both individual piles and for pile groups. Although structural elements (columns) can be supported by a single pile, they are most commonly supported by a group of piles incorporated in a single pile cap.

8.2.5   Static Pile Load Test

Often a pile load test is conducted to prove the capacity of the pile. The reason for the proof lies in the unreliability of the prediction methods. Building Codes will state whether a load test is required for a selected pile and provide the criteria for establishing the allowable load based on the load test.

An axial load test should conform to the procedures outlined in ASTM D1143. Tests can be conducted to prove the compression capacity, tension capacity and lateral capacity of a pile. In a pile load test, constantly increasing loads are applied to the pile as prescribed in the applicable standard procedures. As the load increases, pile movement is measured and recorded at the butt (top) end of the pile and each increment of load is allowed to remain on the pile for a specified length of time. In some cases, telltales are used to measure movement at the tip of the pile.

At the completion of the test, the pile is unloaded in specific increments and the net settlement of the pile after the entire load has been removed is measured and recorded. The information is presented in the form of a plot of load vs. settlement. The engineer will use this information to verify the load capacity of the pile.

8.2.6 Dynamic Testing

A Pile Driving Analyzer provides a general indication of pile capacity. It measures the hammer and cushion performance and pile stresses during driving from measurements of applied force and acceleration at the pile head. The Analyzer is helpful for establishing the pile driving criterion and can provide quality control when used in combination with a static pile load test.

The Pile Driving Analyzer can also be used in conjunction with theoretical predictions of pile capacity when a pile load test is not economically justified. The instrument can also be used to evaluate the driving hammer efficiency and to evaluate or detect damaged piles.

9.0   DRILLED SHAFTS

Drilled shafts are also referred to as caissons or piers or drilled piers. Drilled shafts typically have a diameter greater than 2.5 feet. Drilled shafts are installed by drilling a hole of the required diameter to the required depth and filling the excavation with concrete. Drilled shafts can have a straight shaft for the entire length or the bottom can be belled to enlarge the base and increase the bearing area.

Drilled shafts are classified as straight shafts or belled shafts. A straight shaft is a shaft that extends nearly at a constant diameter from the cut off end to the tip. In reality however, the shaft might taper inward slightly because of the way they are installed especially if they are deep. To reach depth progressively smaller diameter shafts are inserted in the upper shaft in a telescope fashion. The bearing surface at the tip of the shaft is the same diameter as the diameter of the last shaft section.

On the other hand, belled shafts have an enlarged base. The final shaft length is undercut to form an enlarged base which increases the bearing area of the tip. The success of the bell depends on the type of material in which the bell is cut. Clay is a suitable material because its cohesive strength will support the formation of a bell. Sand on the other hand, or material with sand lenses are poor materials because the bell is likely to collapse.

9.1   Geotechnical Capacity of Drilled Shafts

Methods to estimate the geotechnical capacity of a drilled shaft are similar to the methods described in Section 8.2.3 for piles. The ultimate load-carrying capacity of a drilled shaft (Q_u) is the sum of the point resistance (Q_p) plus the skin friction derived from the soil-pile interface (Q_s).

$$Q_u = Q_p + Q_s \qquad (9.1)$$

Procedures are available to determine the load carrying capacity of drilled shafts supported by various materials, such as sand, clay and rock, and to determine the expected settlement.

The net capacity of the pier at the base $Q_p(net)$ is determined by subtracting the effective stress at the base due to the weight of soil. An appropriate factor of safety is applied to the net ultimate load to obtain the net allowable load carrying capacity of the drilled shaft.

$$Q_{all}(net) = (Q_p(net) + Q_s) / FS \qquad (9.2)$$

The drilled shaft must also be capable of supporting the applied load through the structural strength of the concrete shaft. Depending upon specific loading conditions, drilled shafts must resist uplift (tension) loads and lateral loads. Design procedures are available for estimating the uplift and lateral capacity of drilled shafts.
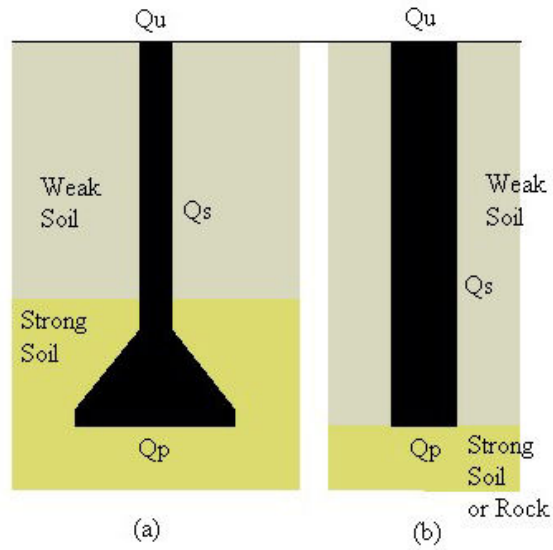
Figure 9.1 – Drilled Shaft Capacity

9.2   Settlement of Drilled Shafts

Like other foundation systems, drilled shafts must also be assessed for the magnitude of settlement under the design load.

10.0   COMPACTED FILL

10.1   Introduction

Often structures are supported on compacted fill. Typically, unsuitable material can be removed from below a structure and replaced with compacted fill to support the structure. The structure can also be built to a new grade using compacted fill. In order for the structure to perform in a satisfactory manner, the fill must be placed and compacted to a specified standard. A standard commonly used is the degree or percent compaction although relative density is also used.

10.2   Percent Compaction

Percent compaction is a measure of the density (unit weight) of soil in place after it has been compacted to a standardized theoretical maximum density determined by laboratory methods. There are several ways of measuring the in-place density of soil in the field as defined by the American Society for Testing and Materials (ASTM). Methods such as the nuclear density gage (ASTM D2922), sand cone (ASTM D1556) and balloon (ASTM D2167) are available. The nuclear density gage is quite common today followed by the sand cone. The balloon method is seldom used. Using one of the specified methods, it is possible to determine the in-place density of the compacted soil.

Since the soil is almost always moist, the in-place density (unit weight) which has moisture in the voids is converted to dry density. This eliminates the weight variable related to moisture and expresses the density in terms of the weight of solids (Ws). The dry unity weight is calculated by determining the weight of solids which were extracted from a hole having a volume (V). The in-place density expressed in terms of dry unit weight becomes the numerator in the calculation for percent compaction.

The denominator is derived in the laboratory using a sample of the same soil that was placed as fill. Methods such as Standard Proctor (ASTM D698) or Modified Proctor (ASTM D1557) are used to determine the *theoretical* maximum dry density of the soil material. Note that the dry density is used to avoid the variability in unit weight resulting from moisture. The term "theoretical" is used because the density is determined in the laboratory based on using a specific amount of energy to compact the material.

The Standard Proctor Test was developed to duplicate in the laboratory, as nearly as possible, the results that could be obtained by compaction equipment working in the 1930s. Since then, compaction equipment has improved and it was possible to achieve higher dry unit weights. For this reason, the modified test was developed in response to the higher compactive efforts being achieved.

The energy used to compact the soil in the laboratory is based on dropping a 5.5-pound weight 12 inches (ASTM D698) or 10-pound weight 18 inches (ASTM D1557) a specified number of times on each layer of soil placed in a standard mold. For instance,

the maximum dry density can be determined using ASTM D1557 Method C when 5 equal height layers of soil are placed in a 6-inch diameter mold and each layer is compacted by making 56 hammer drops of a 10-pound weight falling 18 inches from the surface of the specimen. *The result is that the soil has been compacted using a specific amount of energy from the falling weight.*

In the field, the energy is applied by the compactor. If the contractor is so inclined, he can vary lift thicknesses and the number of passes made over a lift of soil. Depending upon circumstances, this can result in more energy being applied to compact the soil in the field than applied in the laboratory to determine the "theoretical" maximum dry density. In this case, when the soil is over-compacted in the field, the resulting degree or percent compaction can be greater than 100 percent. Therefore, it is possible to achieve greater than 100 percent compaction although the results might seem strange.

10.3 Observation and Testing Compacted Fill

Geotechnical engineers are often called upon to provide construction observation and testing. Aside from observing and logging the installation of pile and drilled pier foundations, the engineer has also been requested to provide observation and testing services on compacted fill. The purpose of these services is to verify that the contractor is supplying material that meets the gradation requirements and that the required degree of compaction has been attained. If the compaction test results are low or the material characteristics change considerably, then the contractor is advised and remedial measures are taken.

On many projects involving compacted fill, the engineer or testing agency is requested to visit the site only after the lift of soil has been placed and compacted. As a practical matter, particularly on large earthwork projects where buildings are constructed on compacted fill, many lifts of fill can be placed and compacted before the engineer arrives. Testing is done at selected locations and the degree of compaction is reported for each of these areas tested. The success of the fill and compactive effort is determined by a few tests. But what about the material lying between the test locations? Normally, it is assumed that the material lying between the test locations is as good as the material at the test location.

What if one or two tests fail the compaction criterion? Does this mean that the entire fill lying between the test locations is also unsatisfactory? Normally, these problem areas are recompacted and retested until they pass the compaction criterion. When there is no qualified engineer on site to witness the placement and compaction of the fill, these questions can remain unanswered. An experienced engineer's participation on site on a daily basis can provide these answers. The qualified engineer can observe the placement and compaction of the fill and can witness how the material reacts when compacted. The engineer can verify that the fill was placed over properly prepared subgrade, that the fill was placed in the correct thickness, that the fill material meets the specifications, and that sufficient moisture content and compactive effort was achieved.

11.0   ARRIVING AT ACCEPTABLE SOLUTIONS

For a moment, consider that the deformation of a steel member under a compressive load is equivalent to the settlement of a foundation under its design load. Calculating the deformation of short sections of steel under an applied load is relatively straightforward and depends upon the applied load, area of the section, original length of the member and the elastic modulus of steel. All of these variables are easily acquired and require no interpretation.

On the other hand, calculating the theoretical settlement of foundations requires a great deal of interpretation and judgment. Factors such as the complexity of the soil profile, the engineering properties of the soil itself, the previous load history of the soil and the variation in groundwater level all play an important role in the outcome.

Is it strange then that geotechnical engineers can arrive at a different set of equally correct solutions to a problem even if they are given identical information? Since judgment is required along virtually every step of geotechnical design, differences in experience, judgment and methods of analysis can affect the conclusion. Soil properties are not specified and the engineer must develop the soil properties by explorations, testing and using the engineer's own experience and judgment. Since it is unlikely that anyone would have all of the information associated with a site, the engineer is faced with choosing simple models based on the limited data that is economically feasible to retrieve in order to predict the outcome.

An interesting study was undertaken in 1988 by Thomas F. Wolff to explore how judgment plays a role in geotechnical engineering design. In this study, a group of experienced practitioners and students were asked to design a shallow foundation (i.e. specify the size of footing required to carry the specified load but not exceed tolerable limits of settlement). Each participant was given identical information regarding loads and subsurface conditions. It was up to each participant to study the data and select appropriate values and methods to derive their conclusion.

The results of the study showed wide variability in the geotechnical values selected. In addition, interesting information was revealed about how the participants formulated their conclusions regarding values that were derived from the same set of subsurface information. Among the findings:

1. N-values, derived from the soil test boring logs, resulted in a range of values used in design. Participants selected values that ranged from 14 blows per foot to 26 blows per foot.

2. The soil friction angle selected by the participants ranged from 30 degrees to 35 degrees. However, no designer used a friction angle greater than 35 degrees even when correlations suggested a greater value.

3. Practitioners tend to be more conservative than students.

The study also showed that the participants recommended a wide range of footing sizes for design. The recommended footings ranged from 5 feet to 9.75 feet wide to support the same given load. The results reflect many factors such as interpretation and selection of soil values, methods of analysis and the participant's experience. Although this study was presented for shallow spread footings, there are likely to be similar results for other aspects of geotechnical engineering such as deep foundations. The important facts are that since there is no clearly defined universal set of values and equations used by all practitioners, it would not be unusual for the calculated results to vary among practitioners even when given the same set of subsurface conditions.

12.0   MEANS AND METHODS

There have certainly been many advancements and solutions directed toward building on less desirable sites. Along with the need comes the solution as many methods have been developed to handle different situations. Many are based on a choice with economics as a principle guiding factor. Searching the internet will reveal many examples.

Walls for instance, or methods to resist lateral forces have evolved from a pile of stones to timber retaining walls, cast-in-place retaining walls, gabion retaining walls and other unique modular products, mechanically stabilized walls, anchors and soil nailing to mention a few. Traditional methods for working on sites underlain by undesirable material have evolved from removing and replacing the material or using deep foundations to pass through the material to a combination of solutions or using proprietary methods to stabilize the unsuitable material. Construction of steep slopes is possible through the use of geotextile fabrics and reinforced earth.

As new challenges emerge, the geotechnical engineer and the geotechnical specialty contractor are ready to develop solutions.

APPENDIX A

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APPENDIX B

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