NOTES ON FOUNDATION ENGINEERING

by

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Approved by:

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Major Professor
First of all, I wish to express my deepest gratitude to LASPAU (Lationo-American Scholarship Program for American Universities) and to the U.NAH (Universidad Nacional Autonoma de Honduras) which are the two organizations that provided the support to make my graduate student possible at Kansas State University.

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Finally, I wish to express my gratitude to those professors who, in some way or another, helped me to better understand the complex field of the Civil Engineering, thank you for giving me your time.
THIS BOOK WAS BOUND WITH TWO PAGES NUMBERED 31. THESE PAGES ARE DIFFERENT. THE BOOK WAS BOUND WITHOUT A PAGE NUMBERED 32.

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Purpose of the study

As a LASPAU scholar, part of my duty will be to teach at a university in Honduras at the time I return there. Therefore, the purpose of the work presented in this report is to prepare a set of notes on Foundation Engineering using recent literature available. Those notes should be used as a guide that may be added to the actual program of study at the University of Honduras.

Scope of the Study

The study basically tends to present methods of analysis on Foundation Engineering. These methods of analysis cover the design of spread footing, mat foundations, piles and retaining walls. In addition to that, a soil mechanics review is included.

The material included here could be used in a three-hour-per-week class during a fourteen-week semester.
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CHAPTER I

INTRODUCTION

It is recognized that soils can be considered as the oldest and the most complex of the construction materials used by civil engineers. In the first place, soils have been available since ancient times and man has used them for different purposes. For example, there are evidences that the Romans built notable engineering structures including an important network of durable and excellent roads where they followed two important principles: a solid foundation and good drainage. In spite of the fact that soils have been used a lot, the progress in learning and understanding their behavior has been slow because of the complexity of their structure, the variability in properties and their behavior in presence of water.

Successful foundation design was a matter of experience until the Terzaghi's work of 1925. Until the middle of the nineteenth century, most footings were built of masonry. These were of two types: a) dimension-stone footings consisting of stone cut and dressed to specific sizes, and b) rubble-stone footings which were made of pieces of random size joined by mortar. In some cases, to increase the size of the footings without increasing their weight, timber grillages were built and conventional masonry footings were
placed on them. Later, the timber grillages were replaced by grillages composed of steel railroad rails embedded in concrete, that improvement saved weight and increased space in the basement. At the beginning of the twentieth century, the steel rails were replaced by steel I beams but, early in 1900, with the advent of reinforced concrete, grillage footings were replaced almost totally by reinforced concrete footings, which are still the common type. Typical grillage foundations are shown in Fig. 1.1.

![Diagrams of foundation types](image)

**Figure 1.1.** Historical development of grillage foundations of (a) timber, (b) railroad rails, (c) steel I-beams, (d) Cantilever footing supporting exterior column of Auditorium Building, Chicago, 1887.
The materials used to build the footings has been discussed and in addition to that, it is important to mention that in earliest times, the area of a footing was chosen based on good guessing and experience. For instance, in some parts of the United States, the width of a continuous footing in feet was made equal to the number of stories in the structure. In the early 1870's, some engineers suggested that the area of the footings should be selected proportional to the loads on the footings and that the centers of gravity of the loads should coincide with the centroids of the footings, so that tilting problems could be avoided. This did not always assure success because the principles of soil consolidation were not understood. To alleviate this, "allowable soil pressures" were recommended in building codes based upon experience within a given area.

Finally, because of the misunderstanding of the soil behavior under load, many failures have occurred and some of the causes are listed below:

a) uncontrolled water flow adjacent to a footing.

b) unexpected settlement of soils.

c) unbalanced soil pressure conditions from differences in elevations.

d) loss of vertical support from the removal of the lateral support of the soil.

e) seismic disturbance.

Some examples of failure foundations are shown in Fig. 1.2.
Figure 1.2. Clay backfill and water collapsed the wall.

Clay backfill which became liquid after a heavy storm provoked lateral pressures greater than for which it was designed.
In 1934, Terzaghi published a paper in which he showed that the settlement and the leaning of the tower are due to the gradual consolidation of the clayey soil located 8 m underneath the footing.

Figure 1.3. Pisa Tower. The most successful foundation failure.

The flow of the soft clay under the gravel bed which formed the support of the buildings may have been the cause
of structural damage occurring during the Anchorage earthquake.

Figure 1.4. 4th Avenue, Anchorage, May 1963.

Figure 1.4. 4th Avenue, Anchorage, April 1, 1964, after earthquake.
Having mentioned that the practice of foundation engineering was based on empirical rules, it is evident that the main purpose of foundation engineering is to replace the empirical rules with scientific methods of analysis that require quantitative soil data such as classification, shearing strength and compressibility of the soil.

Today, the philosophy followed in the design of foundations is that they must have two main characteristics:

1) The foundation should be safe against shear failure in the soil that supports it.

2) The foundation should not undergo excessive settlements.

Hence, the scientific methods mentioned above try to give mathematical relationships which are able to predict the capability of the soil to support loads as well as to predict the settlement that the soil could suffer under those loads.

The main types of foundations used today and their general applications are shown in Table 1.
<table>
<thead>
<tr>
<th>Foundation type</th>
<th>Use</th>
<th>Applicable soil conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spread footing, wall</td>
<td>Individual columns, walls,</td>
<td>Any conditions where bearing capacity is adequate for applied load. May use on single stratum: firm layer over soft layer or soft layer over firm layer. Check immediate, differential, and consolidation settlements</td>
</tr>
<tr>
<td>footings</td>
<td>bridge piers</td>
<td></td>
</tr>
<tr>
<td>Mat foundation</td>
<td>Same as spread and wall footings.</td>
<td>Generally soil bearing value is less than for spread footings: over one-half area of building covered by individual footings. Check settlements</td>
</tr>
<tr>
<td></td>
<td>Very heavy column loads. Usually</td>
<td></td>
</tr>
<tr>
<td></td>
<td>reduces differential settlements and</td>
<td></td>
</tr>
<tr>
<td></td>
<td>total settlements</td>
<td></td>
</tr>
<tr>
<td>Pile foundations</td>
<td>In groups (at least 2) to carry</td>
<td>Poor surface and near surface soils. Soils of high bearing capacity 30-50 m below basement or ground surface, but by distributing load along pile shaft soil strength is adequate. Corrosive soils may require use of timber or concrete pile material</td>
</tr>
<tr>
<td>Floating</td>
<td>heavy column, wall loads: requires</td>
<td></td>
</tr>
<tr>
<td></td>
<td>pile cap</td>
<td></td>
</tr>
<tr>
<td>Bearing</td>
<td>In groups (at least 2) to carry</td>
<td>Poor surface and near-surface soils: soil of high bearing capacity (point bearing on) is 8-50 m below ground surface</td>
</tr>
<tr>
<td></td>
<td>heavy column, wall loads: requires</td>
<td></td>
</tr>
<tr>
<td></td>
<td>pile cap</td>
<td></td>
</tr>
<tr>
<td>Caisson (shafts 75 cm</td>
<td>Larger column loads than for piles</td>
<td>Poor surface and near-surface soils: soil of high bearing capacity (point bearing on) is 8-50 m below ground surface</td>
</tr>
<tr>
<td>or more in diameter)</td>
<td>but eliminates pile cap by using</td>
<td></td>
</tr>
<tr>
<td></td>
<td>caissons as column extension</td>
<td></td>
</tr>
<tr>
<td>Retaining walls, bridge</td>
<td>Permanent retaining structure</td>
<td>Any type of soil, but a specified zone (Chaps. 11, 12) in back of wall usually of controlled backfill</td>
</tr>
<tr>
<td>abutments</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sheet-pile structures</td>
<td>Temporary retaining structures as</td>
<td>Any soil: waterfront structure may require special alloy or corrosion protection. Cofferdams require control of fill material</td>
</tr>
<tr>
<td></td>
<td>excavations, waterfront structures,</td>
<td></td>
</tr>
<tr>
<td></td>
<td>cofferdams</td>
<td></td>
</tr>
</tbody>
</table>

Table 1. Foundation types and typical usage.
CHAPTER II

SOIL MECHANICS REVIEW

a. Soil Deposits

Soil "can be defined as naturally occurring mineral aggregations that can be separated easily into smaller particles and in mass form contains numerous voids." It is also defined as a natural occurring material that can be moved by conventional earth moving equipment. Soil deposits can be divided in three general categories: 1) residual soils, 2) transported soils, and 3) organic soils. Residual soils are those which develop in the place of their formation from the underlying rock. Transported soils are those which have been removed from the place of their formation by wind, water, ice or gravity to the current location. Based on the transporting agent, transported soils can be subdivided into three categories.

1) Aeolian -- deposited by wind action
2) Alluvial -- deposited by running water
3) Glacial -- deposited by glacier action
4) Talus and Creep -- deposited by gravity.

The organic soils are those derived from the decomposition of organic materials and usually are found only in swamps and northern latitudes.

The engineering properties are different for each of
these deposits, in general, residual deposits tend to have better foundation characteristics in terms of strength and deformation than the other two but, when the residual soil above the bedrock is normally consolidated, heavy loads on large foundations could cause serious settlements. In general, alluvial deposits have noticeable variation in physical properties (void ratio, unit weight), altering the load capacity of the soil. The physical characteristics of the unstratified deposits laid down by glaciers when they melt (till), may vary from glacier to glacier and during field exploration programs, erratic values of standard penetration tests can be obtained. Aeolian deposits such as loess can be considered a collapsing soil because it looses its strength when it becomes saturated. Therefore, special precautions should be taken for building foundations on loessial deposits. Finally, the organic deposits have a high natural moisture content and are highly compressible causing a large amount of settlement under load due to secondary consolidation.

The skilled geotechnical engineer recognizes that each of the soil classes vary in a uniform and predictable manner. Most soil strata will have a decreasing void ratio with depth (probably due to the weight of the superimposed overburden) and this alone will increase the shearing resistance and decrease the compressibility.
The water transported alluvium will vary with the velocity of the transporting water since the slower the velocity of the stream, the finer the deposited sediment will be. Thus, most river valleys are aggraded with coarser material at the bottom with finer material at the top. These materials will always be well rounded sand and gravel.

The wind deposited stratum will become finer grained and the thickness of the stratum less in thickness with increasing distance from the source of the soil particles. These stratum are always composed of silt sized particles.

The ice transported materials are always unsorted and will contain particles from boulder to clay size. Any stratification or sorting indicates a reworking by water.

Gravity transported materials are found only at the base of mountains and consist of very large angular blocks or rock with the voids filled with angular granular materials. Although vast amounts of rocks are transported down minor slopes by soil creep mechanisms, these materials retain the characteristics imparted by the original agent of transportation.

b. Physical Characteristics of the Soils.

In nature, the soil is composed of solid particles, water and air (or gas). The mathematical relationships for the three phases can be defined as follows:
Void ratio, e: The ratio of the volume of voids \( v_v \) to the volume of solids \( v_s \) in a given soil mass and can be written:

\[
e = \frac{v_v}{v_s}
\]  

(2.1)

Porosity, \( n \): The ratio of the volume of voids to the volume of the soil specimen \( v \).

\[
n = \frac{v_v}{v}
\]  

(2.2)

Water content, \( w \): The ratio of the weight of water \( w_w \) in a given soil mass to the weight of soil solids \( w_s \) in the same mass.

\[
w(x) = \frac{w_w}{w_s} \times 100
\]  

(2.3)

Unit weight, \( \gamma \): The ratio of the weight of soil to the corresponding volume.

\[
\gamma = \frac{w_t}{v}
\]  

(2.4)

Since water is present, the weight has to be obtained as: unit weight, dry unit weight, submerged unit weight and saturated unit weight. They can be written as

\[
\text{moisture unit weight } \gamma_m = \frac{w_t}{v}
\]

where \( w_t = w_s + w_w \). Assuming the weight of the air is equal to zero.
Dry unit weight \( \gamma_d = \frac{w_s}{v} \)

Saturated unit weight \( \gamma_s = \gamma_m \) if \( v_v = v_w \)

Submerged unit weight \( \gamma_{sub} = \gamma_v - \gamma_w \)

**Degree of saturation, \( S \):** the ratio of the volume of water to the total volume of soil voids.

\[
S(\%) = \frac{v_w}{v_v} \times 100
\]

(2.5)

The soil is completely saturated when all the void volume is occupied by water, usually below the water table.

**Specific gravity, \( G_s \):** The ratio of the unit weight of a material in air to the unit weight of distilled water at 40\(^\circ\)C.

\[
G_s = \frac{w_s/v_s}{w_w/v_w} = \frac{w_s}{w_w} \cdot \frac{v_v}{v_w} 
\]

(2.6)

\( \gamma_w = 62.4 \)pcf or 1.00 g/cm\(^3\)

Representative values of \( G_s, e \) and \( d \) for natural soils are shown in Table 2.1 and 2.2.

**Table 2-1. Specific Gravities of Some Soils.**

<table>
<thead>
<tr>
<th>Soil type</th>
<th>( G_s )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quartz sand</td>
<td>2.64-2.86</td>
</tr>
<tr>
<td>Silt</td>
<td>2.67-2.73</td>
</tr>
<tr>
<td>Clay</td>
<td>2.70-2.90</td>
</tr>
<tr>
<td>Chalk</td>
<td>2.60-2.75</td>
</tr>
<tr>
<td>Loess</td>
<td>2.65-2.73</td>
</tr>
<tr>
<td>Peat</td>
<td>1.30-1.90</td>
</tr>
</tbody>
</table>
Table 2-2. Void Ratio, Moisture Content, and Dry Unit Weight for Some Soils.

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>Void ratio (e)</th>
<th>Natural moisture content in saturated condition (%)</th>
<th>Dry unit weight (γd, kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose uniform sand</td>
<td>0.8</td>
<td>30</td>
<td>14.5</td>
</tr>
<tr>
<td>Dense uniform sand</td>
<td>0.45</td>
<td>16</td>
<td>18</td>
</tr>
<tr>
<td>Loose angular-grained silty sand</td>
<td>0.95</td>
<td>25</td>
<td>16</td>
</tr>
<tr>
<td>Dense angular-grained silty sand</td>
<td>0.4</td>
<td>15</td>
<td>19</td>
</tr>
<tr>
<td>Stiff clay</td>
<td>0.6</td>
<td>21</td>
<td>17</td>
</tr>
<tr>
<td>Soft clay</td>
<td>0.9-1.4</td>
<td>30-50</td>
<td>11.5-14.5</td>
</tr>
<tr>
<td>Loess</td>
<td>0.9</td>
<td>25</td>
<td>13.5</td>
</tr>
<tr>
<td>Soft organic clay</td>
<td>2.5-3.6</td>
<td>90-120</td>
<td>6-8</td>
</tr>
<tr>
<td>Glacial till</td>
<td>0.3</td>
<td>10</td>
<td>21</td>
</tr>
</tbody>
</table>

Example 2.1

- Underlined information in Table is the data given:
- Obtain the required data in each case.

<table>
<thead>
<tr>
<th>Gw (%)</th>
<th>Moisture S (%)</th>
<th>Saturation S %</th>
<th>a</th>
<th>n</th>
<th>Void Ratio</th>
<th>Porosity</th>
<th>γd (pcf)</th>
<th>γsat. (pcf)</th>
<th>γsub. (pcf)</th>
<th>γtotal (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.65</td>
<td>20</td>
<td>84.2</td>
<td>0.6292</td>
<td>0.3862</td>
<td>101.50</td>
<td>125.6</td>
<td>63.2</td>
<td>121.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.65</td>
<td>18</td>
<td>66.2</td>
<td>0.72</td>
<td>0.4186</td>
<td>96.14</td>
<td>122.1</td>
<td>59.8</td>
<td>113.44</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.65</td>
<td>20</td>
<td>100</td>
<td>0.5300</td>
<td>0.3464</td>
<td>100.88</td>
<td>129.6</td>
<td>67.2</td>
<td>129.69</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.65</td>
<td>20</td>
<td>50</td>
<td>1.9600</td>
<td>0.5146</td>
<td>80.27</td>
<td>112.3</td>
<td>49.9</td>
<td>96.32</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Soil 1

\[ w = \frac{w_W}{w_s}, \quad 0.20 = \frac{w_W}{w_s}, \quad 0.20 \ w_s = w_W \quad (1) \]

\[ G = \frac{w_s}{v_s \ 62.4}, \quad 2.65(6.24) \ v_s = w_s \quad (2) \]

\[ T_{sub} = \gamma_{sat} - T_w \quad (3) \]

\[ 63.2 = \gamma_{dry} + 62.4 \ v_v - 62.4 \]
\[ 63.2 = 2.65(62.4) \ v_s + 62.4(v_v) - 62.4 \]

\[ 125.6 = 165.36 \ v_s + 62.4 \ v_v \]

\[ \gamma_d = 2.65(0.6138)62.4 = 101.50 \ pcf \]

\[ \gamma_{sat} = 101.5 + 62.4(0.3862) = 125.6 \ pcf; \quad v_v + v_s = 1 \quad (4) \]

Solving (3) and (4).

\[ \gamma_{tot} = 101.5 + 62.4(0.3253) = 121.8 \ pcf \]

\[ \gamma_{sub} = 125.6 - 62.4/4 = 63.2 \ ok \]

\[ S = \frac{v_w}{v_v} = \frac{0.3253}{0.3862} = 0.842 \]

\[ V_{w} = \frac{0.3253}{62.4} = 0.3253 \quad v_v = 0.3862 \quad \text{(Solving (3) and (4))} \]

\[ w_w = 0.20(101.50) = 20.3; \quad 125.6 + 165.36 (1 - v_v) + 62.4 \ v_v \]

\[ v_w = \frac{20.3}{62.4} = 0.3253 \]

\[ v_v = 0.3862 \]

\[ v_s = 0.6138. \]

\[ e = \frac{v_v}{v_s} = \frac{0.3862}{0.6138}, \quad e = 0.6292 \]

\[ \frac{v_s}{0.6138} \]
Soil 2.

\[ v_v = 0.3862 \]
\[ n = \frac{v_v}{v_t} = \frac{0.3862}{1} = 0.3862. \]

\[ e = \frac{v_v}{v_s} = 0.72 \quad v_s = v_v \quad (1) \]
\[ v_v + v_s = 1 \quad v_v = 1 - v_s \quad (2) \]

Solving (1) and (2).

\[ 0.72 \quad v_s = 1 - v_s \]
\[ v_s = 0.5814 \]
\[ v_v = 0.4186 \]

\[ \gamma_{sat} = 96.14 + 62.4(0.4186) = 122.26 \text{ pcf} \]
\[ w_s = 2.65(0.5814)(62.4) = 96.14 \quad \gamma_c \]

\[ \gamma_{sub} = 122.26 - 62.4 = 59.86 \text{ pcf} \]
\[ w = \frac{w_w}{w_s}, \quad 0.18 = \frac{w_w}{96.14}, \quad w_w = 17.30 \]

\[ \gamma_{tot} = 96.14 + 62.4(0.2773) \]

\[ v_w = \frac{17.30}{62.4} = 0.2773 \]

\[ s = \frac{v_w}{v_v} = \frac{0.2773}{0.4186} = 0.662, \quad 66.27\% = s \]

\[ n = \frac{v_v}{v_t} = \frac{0.4186}{1} = 0.4186 \]
Soil 3.

\[ w = \frac{w_W}{w_S} = 0.20, \quad 0.20 = \frac{w_W}{w_S}, \quad 0.20 \quad w_S = w_W \quad (1) \]

\[ S = \frac{v_W}{v_V} = 1, \quad v_W = v_V \]

From Eq. (1)

\[ 0.20 \quad v_S \quad (2.65) = v_W \]
\[ 0.53 \quad v_S = v_W \quad (2). \]

\[ v_W + v_S = 1 \quad (3) \quad the \quad soil \quad is \quad saturated. \]

Solving (3) and (2)

\[ 0.53 \quad v_S = 1 - v_S \]
\[ v_S = 0.6536 \]
\[ v_W = 0.3464 \]

\[ e = \frac{0.3464}{0.6536} = 0.5300 \]

\[ n = \frac{0.3464}{1} = 0.3464. \]

\[ \gamma_d = 2.65 \quad (62.4) \quad (0.6536) = 108.08 \]
\[ \gamma_V = 108.08 + 62.4 \quad (0.3464) = 129.69 = \gamma_{sat}. \]
\[ \gamma_{sub} = 129.69 - 62.4 = 67.29 \]

Soil 4.

\[ w = \frac{w_W}{w_S} = 0.20, \quad 0.20 \quad w_S = w_W. \quad (1) \]
\[ s = \frac{V_W}{V_V} = 0.5 \quad , \quad V_W = 0.5 \quad V_V. \quad (2) \]

From (1).

\[ V_V + V_S = 1 \]

\[ 0.20(2.65)(62.4)(V_S) = 62.4(0.5 \quad V_V) \]
\[ 0.20(2.65)(62.4)(1-V_V) = 62.4(0.5 \quad V_V) \]

\[ V_V = 0.5146 \]
\[ V_W = 0.2573 \]
\[ V_S = 0.4854 \]

\[ e = \frac{0.5146}{0.4854} = 1.060 \]

\[ n = \frac{0.5146}{1} = 0.5146 \]

\[ \gamma_d = 2.65(62.4)(0.4854) = 80.27 \text{pcf} \]
\[ \gamma_{tot} = 80.27 + 62.4(0.2573) = 96.32 \text{pcf} \]
\[ \gamma_{sat} = 80.27 + 62.4(0.5146) = 112.38 \text{pcf} \]
\[ \gamma_{sub} = 112.38 - 62.4 = 49.98 \text{pcf} \]

c. Soil tests

The following tests are those most commonly used for foundation design work.

d. Soil classification systems

In order to have a common language, which can be understood by anyone, it is necessary to classify the soil based on common engineering properties such as grain size distribution, liquid limit and plastic limit. The two main classifi-
<table>
<thead>
<tr>
<th>Test</th>
<th>Data Obtained</th>
<th>Used For</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Specific Gravity (Gs)</td>
<td>Ratio between density of soil and density of water</td>
<td>Auxiliary factor to compute other soil properties ($n, c$) in consolidation studies of clay, degree of saturation of a soil, etc.</td>
</tr>
<tr>
<td>2. Atterberg Limits</td>
<td>Liquid Limit (LL) Plastic Limit (PL) Plastic Index (PI)</td>
<td>Classification of soils (cohesive) The LL can give some idea about how expansive the soil is</td>
</tr>
<tr>
<td>3. Grain Size Analysis</td>
<td>Grain Size Distribution curve</td>
<td>Classification of soils (size particle bigger than No. 200 sieve)</td>
</tr>
<tr>
<td>4. Hydrometer Analysis</td>
<td>Grain Size Distribution curve</td>
<td>Classification of soils (size particle smaller than No. 200 sieve)</td>
</tr>
<tr>
<td>5. Water Content</td>
<td>Content of Water in Soil</td>
<td>Auxiliary factor to calculate other Soil Properties</td>
</tr>
<tr>
<td>6. Void ratio and porosity</td>
<td>$e$: void ratio $n$: porosity</td>
<td>$e = important$ (maybe the most) characteristic which can give idea about the compressibility of the soil $n$: gives some idea about the degree of soil density</td>
</tr>
<tr>
<td>7. Compaction Test</td>
<td>Optimum moisture content and maximum dry density</td>
<td>Used for determining the soil moisture-density relationship of a soil used for building a fill (road, dam)</td>
</tr>
<tr>
<td>8. Permeability Test</td>
<td>$K$: coefficient of permeability</td>
<td>Studying: quantity of leakage through and under dams Rate of consolidation Stability of slopes, embankments Seepage velocity through the soil</td>
</tr>
<tr>
<td>9. Consolidation test</td>
<td>$i$: compression index $c_i$: coefficient of consolidation</td>
<td>to obtain the amount of settlement to obtain the rate of settlement with time</td>
</tr>
<tr>
<td>10. Direct Shear Test</td>
<td>Shear resistance ($c$) in terms of $c = c + N + g$</td>
<td>Determining the possible bearing capacity of the soil in foundations and earthwork</td>
</tr>
<tr>
<td>11. Triaxial compression test</td>
<td>Shear resistance ($c$) in terms of $c$</td>
<td>Making estimates of the probable bearing capacity, stability calculations of</td>
</tr>
<tr>
<td>Test Type</td>
<td>Description</td>
<td></td>
</tr>
<tr>
<td>----------------------------------</td>
<td>-----------------------------------------------------------------------------</td>
<td></td>
</tr>
<tr>
<td>12. Unconfined Compression Test</td>
<td>Measuring the consistency of cohesive soils giving an idea about rupture of embankments, slopes. Determining bearing capacity of the soil</td>
<td></td>
</tr>
<tr>
<td>Shear resistance (c)</td>
<td>in terms of &quot;c&quot;</td>
<td></td>
</tr>
<tr>
<td>13. Vane shear test</td>
<td>Obtaining the shearing resistance of the soil when it is not possible to run unconfined compression or triaxial test</td>
<td></td>
</tr>
<tr>
<td>Shear strength</td>
<td>of the soil in terms of torsional moments</td>
<td></td>
</tr>
<tr>
<td>14. California Bearing Ratio (CBR)</td>
<td>Designing asphalt pavement structures</td>
<td></td>
</tr>
<tr>
<td>CBR which is a comparative measure of the shearing resistance of the soil</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15. Plate Bearing Test</td>
<td>The design and evaluation of asphalt pavements</td>
<td></td>
</tr>
<tr>
<td>The strength of any elevation in an asphalt pavement</td>
<td></td>
<td></td>
</tr>
<tr>
<td>16. Standard Penetration test (SPT)</td>
<td>Obtaining the bearing capacity of soils directly</td>
<td></td>
</tr>
<tr>
<td>N&quot; values</td>
<td>To obtain the capacity of a pile to support loads</td>
<td></td>
</tr>
<tr>
<td>17. Pile load tests</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pile capacity&quot; or ultimate pile load&quot;, pile head</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
cation systems currently used are (1) the AASHTO (American Association of State Highway and Transportation Officials) system and (2) The Unified System. The AASHTO classification system is mainly used for disturbed soils, for instance, highway subgrades. It is not used in foundation work. Normally, the unified system (Table 2.3) is used in foundation design.

In the Unified System, the following symbols are used for identification.

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>G</td>
<td>Gravel</td>
</tr>
<tr>
<td>S</td>
<td>Sand</td>
</tr>
<tr>
<td>M</td>
<td>Silt</td>
</tr>
<tr>
<td>C</td>
<td>Clay</td>
</tr>
<tr>
<td>O</td>
<td>Organic silts and clay</td>
</tr>
<tr>
<td>P&lt;sub&gt;t&lt;/sub&gt;</td>
<td>Peat and highly organic soils</td>
</tr>
<tr>
<td>H</td>
<td>High plasticity</td>
</tr>
<tr>
<td>L</td>
<td>Low plasticity</td>
</tr>
<tr>
<td>W</td>
<td>Well graded</td>
</tr>
<tr>
<td>P</td>
<td>Poorly graded</td>
</tr>
</tbody>
</table>

Table 2.4 can be used for a better interpretation of the unified system when used in foundation design.

Example 2.2

The following laboratory results were obtained:

percent passing No. 200 sieve : 70%

\[ LL = 35. \]
\[ PI = 20. \]

Classify the soil by the unified system: Refer to Table 2.3,
since more than 50% of the soil passes through #200 sieve, it is a fine-grained soil. From the plasticity chart with LL = 35, PI = 20, the soil is CL, clay of low plasticity and from Table 2.4, it can be said that the soil may be used to support foundation structures when the seepage is important.

e. Shear strength

The shear strength of soil is the resistance to deformation by continuous shear displacement of soil particles upon the action of tangential stress (Jumikis). It can be said that the shear strength is developed due to:

1. interlocking of soil particles
2. frictional resistance between soil particles at their contact points ($\phi$)
3. cohesion between particles ($c$).

Hence, the shear strength is given by Coulomb as

$$s = c + \sigma \tan \phi$$ (in terms of total stresses)

$$x = c' + \sigma' \tan \phi$$ (in terms of effective stresses)

where $s =$ shear strength

$c =$ cohesion

$\phi =$ friction angle

$\sigma' =$ normal stress on shear plane

Shear strength of a soil is heavily dependent on the type of test. Therefore, the test to be used should represent as good as possible the site conditions, either
Table 2.3. Unified System of Classification.

<table>
<thead>
<tr>
<th>Major divisions</th>
<th>Group symbols</th>
<th>Typical names</th>
</tr>
</thead>
<tbody>
<tr>
<td>More than 50% retained on No. 200 sieve</td>
<td>GW</td>
<td>Well-graded gravels and gravel-sand mixtures, little or no fines</td>
</tr>
<tr>
<td>Gravels</td>
<td>GP</td>
<td>Poorly graded gravels and gravel-sand mixtures, little or no fines</td>
</tr>
<tr>
<td>Gravels with Fines</td>
<td>GM</td>
<td>Silty gravels, gravel-sand-silt mixtures</td>
</tr>
<tr>
<td>GC</td>
<td>Clayey gravels, gravel-sand-clay mixtures</td>
<td></td>
</tr>
<tr>
<td>More than 80% of coarse fraction retained on No. 4 sieve</td>
<td>SW</td>
<td>Well-graded sands and gravelly sands, little or no fines</td>
</tr>
<tr>
<td>Sands</td>
<td>SP</td>
<td>Poorly graded sands and gravelly sands, little or no fines</td>
</tr>
<tr>
<td>Sands with Fines</td>
<td>SM</td>
<td>Silty sands, sand-silt mixtures</td>
</tr>
<tr>
<td>SC</td>
<td>Clayey sands, sand-clay mixtures</td>
<td></td>
</tr>
<tr>
<td>Less than 5% pass No. 200 sieve</td>
<td>ML</td>
<td>Inorganic silts, very fine sands, rock flour, silty or clayey fine sands</td>
</tr>
<tr>
<td>Silts and Clays</td>
<td>CL</td>
<td>Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays</td>
</tr>
<tr>
<td>Liquid limit greater than 50%</td>
<td>OL</td>
<td>Organic silts and organic silty clays of low plasticity</td>
</tr>
<tr>
<td>Fine-Grained Soils</td>
<td>MH</td>
<td>Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts</td>
</tr>
<tr>
<td>CH</td>
<td>Inorganic clays of high plasticity, fat clays</td>
<td></td>
</tr>
<tr>
<td>OH</td>
<td>Organic clays of medium to high plasticity</td>
<td></td>
</tr>
<tr>
<td>Highly Organic Soils</td>
<td>PT</td>
<td>Peat, muck, and other highly organic soils</td>
</tr>
</tbody>
</table>

Classification criteria:

- $C_\alpha = \frac{D_{10}}{D_{60}}$ Greater than 4
- $C_\beta = \frac{(D_{60})^2}{D_{10} \times D_{60}}$ Between 1 and 3

Not meeting both criteria for CW:
- Atterberg limits plotting below "A" line or plasticity index less than 4
- Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols

Not meeting both criteria for SW:
- Atterberg limits plotting below "A" line or plasticity index less than 4
- Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols

For classification of fine-grained soils and fine fraction of coarse-grained soils.

Equation of A line:

$PI = 0.73(1 - LL)$
<table>
<thead>
<tr>
<th>Group Symbol</th>
<th>Soil Type</th>
<th>Range of Maximum Dry Unit Weight, pcf</th>
<th>Range of Optimum Moisture, Percent</th>
<th>Typical Value of Compression</th>
<th>Typical Strength Characteristics</th>
<th>Typical Coefficient of Permeability, ft/min.</th>
<th>Range of CBR Values</th>
<th>Range of Subgrade Modulus, lb/in²</th>
</tr>
</thead>
<tbody>
<tr>
<td>CN</td>
<td>Well graded clean gravel, gravel-sand mix.</td>
<td>125 - 150</td>
<td>11 - 14</td>
<td>0.3</td>
<td>0.6</td>
<td>0</td>
<td>0.5</td>
<td>33</td>
</tr>
<tr>
<td>CP</td>
<td>Poorly graded clean gravel, gravel-sand mix.</td>
<td>115 - 125</td>
<td>14 - 11</td>
<td>0.4</td>
<td>0.8</td>
<td>0</td>
<td>0.5</td>
<td>37</td>
</tr>
<tr>
<td>CN</td>
<td>Silty gravel, poorly graded gravel-sand mix.</td>
<td>120 - 135</td>
<td>12 - 8</td>
<td>0.5</td>
<td>1.0</td>
<td>0</td>
<td>0.5</td>
<td>36</td>
</tr>
<tr>
<td>GC</td>
<td>Clayey gravel, poorly graded gravel-sand mix.</td>
<td>115 - 130</td>
<td>14 - 9</td>
<td>0.7</td>
<td>1.2</td>
<td>1</td>
<td>1.3</td>
<td>32</td>
</tr>
<tr>
<td>SW</td>
<td>Well graded clean sands, gravelly sands.</td>
<td>110 - 130</td>
<td>16 - 9</td>
<td>0.6</td>
<td>1.2</td>
<td>0</td>
<td>0.6</td>
<td>30</td>
</tr>
<tr>
<td>SM</td>
<td>Silty sands, poorly graded sand-silt mix.</td>
<td>110 - 125</td>
<td>16 - 11</td>
<td>0.8</td>
<td>1.4</td>
<td>1</td>
<td>1.4</td>
<td>37</td>
</tr>
<tr>
<td>SM-SC</td>
<td>Sand-silt clay mix with slightly plastic fines.</td>
<td>110 - 130</td>
<td>15 - 11</td>
<td>0.8</td>
<td>1.4</td>
<td>1</td>
<td>1.4</td>
<td>30</td>
</tr>
<tr>
<td>SC</td>
<td>Clayey sands, poorly graded sand-clay mix.</td>
<td>105 - 125</td>
<td>19 - 11</td>
<td>1.1</td>
<td>2.2</td>
<td>0</td>
<td>0.6</td>
<td>37</td>
</tr>
<tr>
<td>HL</td>
<td>Inorganic silts and clayey silts.</td>
<td>95 - 120</td>
<td>24 - 12</td>
<td>0.9</td>
<td>1.7</td>
<td>1</td>
<td>1.7</td>
<td>30</td>
</tr>
<tr>
<td>HL-CL</td>
<td>Mixture of inorganic silts and clay.</td>
<td>100 - 120</td>
<td>22 - 12</td>
<td>1.0</td>
<td>2.2</td>
<td>2</td>
<td>2.2</td>
<td>32</td>
</tr>
<tr>
<td>CL</td>
<td>Inorganic clays of low to medium plasticity.</td>
<td>95 - 120</td>
<td>24 - 12</td>
<td>1.3</td>
<td>2.5</td>
<td>0</td>
<td>0.5</td>
<td>30</td>
</tr>
<tr>
<td>OL</td>
<td>Organic silts and organic clays, low plasticity.</td>
<td>80 - 100</td>
<td>33 - 21</td>
<td>1.3</td>
<td>2.3</td>
<td>2</td>
<td>2.3</td>
<td>30</td>
</tr>
<tr>
<td>MO</td>
<td>Inorganic clayey silts, elastic clays.</td>
<td>70 - 95</td>
<td>40 - 24</td>
<td>2.0</td>
<td>3.0</td>
<td>2</td>
<td>2.5</td>
<td>25</td>
</tr>
<tr>
<td>CN</td>
<td>Inorganic clays of high plasticity</td>
<td>75 - 105</td>
<td>36 - 19</td>
<td>2.5</td>
<td>3.9</td>
<td>2</td>
<td>2.5</td>
<td>19</td>
</tr>
<tr>
<td>OH</td>
<td>Organic clays and silty clays</td>
<td>85 - 100</td>
<td>45 - 21</td>
<td>2.5</td>
<td>3.9</td>
<td>2</td>
<td>2.5</td>
<td>19</td>
</tr>
</tbody>
</table>

Notes:
1. All properties are for condition of "Standard Proctor" maximum density, except values of k and CBR which are for "modified Proctor" maximum density.
2. Typical strength characteristics are for effective strength envelopes and are obtained from USBR data.
3. Compression values are for vertical loading with complete lateral confinement.
4. (C) indicates that typical property is greater than the value shown.
   (...) indicates insufficient data available for an estimate.
during construction time or after construction time, whichever is worse. For example, if consolidation is produced by change in stress due to construction or loading, the unconfined compression test gives conservative results but, if swelling is produced, unconfined compression tests give good results for the construction period or immediately thereafter, but when the shearing resistance decreases with time, the results become less conservative.

The Mohr's circles and rupture envelopes for several tests are shown in Fig. 2.1.

![Mohr's Circles and Rupture Envelopes](image)
In the following examples can be noticed the different results that are obtained if the test conditions are changed.

Example 2.2

The following results were obtained from tests on a saturated clay soil:

(a) Undrained triaxial tests:

<table>
<thead>
<tr>
<th>Cell pressure 3 (lb/in²)</th>
<th>15</th>
<th>25</th>
<th>35</th>
</tr>
</thead>
<tbody>
<tr>
<td>Principal stress difference at failure $\sigma_1 - \sigma_3$ (lb/in²)</td>
<td>20</td>
<td>21</td>
<td>19</td>
</tr>
</tbody>
</table>

The inclination of the plane rupture was 52° to the plane of the cross-section.

(b) Shear box tests in which the soil was allowed to consolidate fully under the influence of both the normal and the shear loads:

<table>
<thead>
<tr>
<th>Normal stress (lb/in²)</th>
<th>8-9</th>
<th>17-9</th>
<th>26-8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear stress at failure (lb/in²)</td>
<td>10-5</td>
<td>14-3</td>
<td>18-6</td>
</tr>
</tbody>
</table>

Determine the shear strength properties of the soil which can be deduced from these results.

(a) From the plot of the three Mohr circles (see Fig. below)
the best Coulomb line gives $c = 10 \text{ lb/in}^2$ and $\phi_u = 0^\circ$. $45^\circ + \phi/2 = 52^\circ$, whence $\phi_f = 14^\circ$.

A line drawn from the origin at $52^\circ$ to the horizontal axis represents the plane of rupture for the case of $C_3 = 0$. This line intersects the Coulomb line at A. From A, draw a line inclined at $14^\circ$ to the horizontal. This intersects the shear stress axis at B, and OB scales $8 \text{ lb/in}^2$, which is the true cohesion.

(b) Direct plotting of the shear-box tests gives the parameters for the drained condition:

$c_d = 6.5 \text{ lb/in}^2$ and $\phi_d = 24^\circ$

Example 2.3

The results of undrained triaxial tests (with pore pressure measurement) on compacted soil at failure are as follows:

<table>
<thead>
<tr>
<th>Lateral pressure $\sigma_3$ (lb/in$^2$)</th>
<th>10</th>
<th>50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total vertical pressure $\sigma_1$ (lb/in$^2$)</td>
<td>43-5</td>
<td>128-0</td>
</tr>
<tr>
<td>Pore-water pressure $u$ (lb/in$^2$)</td>
<td>-4-0</td>
<td>13-5</td>
</tr>
</tbody>
</table>

Determine the apparent cohesion and angle of shearing resistance (a) referred to total stress, and (b) referred to effective stress.

For (a) the Mohr circles A and B are plotted in the usual way. From the common tangent, $c = 8 \text{ lb/in}^2$, and $\phi_u = 21^\circ$ (see Fig. below).
For (b) the effective stresses \((\sigma - u)\) are:

\[
\begin{align*}
\sigma_1' & = 43.5 + 4.0 = 47.5 & 128.0 - 13.5 = 114.5 \\
\sigma_3' & = 10.0 + 4.0 = 14.0 & 50.0 - 13.5 = 36.5
\end{align*}
\]

From these figures the effective-stress circles C and D are plotted. Note that their radii are the same as those of the total-stress circles. The tangent gives \(c' = 2 \text{ lb/in}^2\) and \(\phi' = 30^\circ\) (see Fig. below).

As already indicated, the shear strength of any soil obtained in the laboratory is different for each strength test because of the pore water pressure generated during the test, the pore water pressure dissipates with drainage. In the field, the shearing strength is governed by the rate of load application and drainage condition (permeability coefficient). Therefore, granular soils with coefficients of permeability greater than \(10^{-4}\ \text{cm/sec}\) can be expected to have...
full drainage to eliminate pore pressure due to footing loads if those are applied at a moderate rate. Drained conditions can be assumed to obtain shearing strengths and split tube tests are adequate. When $10^{-3} < k < 10^{-4}$ cm/sec and the footing loads are applied very quickly, pore pressures can be developed and drained conditions are not reliable. For normally consolidated cohesive soils have $k \approx 10^{-5}$ cm/sec, the time required to dissipate the pore water pressure is usually too long, therefore undrained conditions must be used to run thetest and unconfined compression tests can be suitable. The reactions of over-consolidated clays ($OCR > 6, PI > 40$) are difficult to predict since such materials usually contain joints which govern the shearing resistance and the results obtained at the laboratory are very deceiving; for soils having a PI < 40, the triaxial test can give acceptable results. The strengths of soils with intermediate permeability $10^{-4} < k < 10^{-6}$ cm/sec are unpredictable since drainage rate is unknown.

f. Consolidation principles

When a soil has low permeability (only cohesive soils), the amount and the rate of settlement due to load depends on time. Therefore, it is necessary to obtain a compression
parameter to calculate the amount of settlement and a consolidation parameter to calculate the rate of settlement. The compression parameter is called "compression index" \( (c_c) \) and the consolidation parameter is called "coefficient of consolidation" \( c_v \). Both \( c_c \) and \( c_v \) are obtained from consolidation tests. The principle of consolidation is shown in Fig. 2.2. Letting \( d \) be a surcharge at the ground surface which causes the increase of total stress at any depth of the clay layer. It is believed that at a time \( t = 0 \), the increase of effective stress is equal to zero \( = 0 \) and the initial stress is taken by the excess pore water pressure, \( p = \). As time passes, the water pressure is dissipated and the stress is transferred to the soil skeleton as effective stress. At \( t = \), all the excess pore water pressure will be dissipated and \( = 0 \) and \( = p \), having caused gradual settlement when the stress was transferred from the water to the soil.

\[
\begin{align*}
\Delta t &= \Delta p \\
\Delta u &= (\Delta h_i) g \gamma_w = \Delta p \\
\Delta \sigma' &= \Delta \sigma - \Delta u = 0 \\
t = 0 & \\
\Delta \sigma &= \Delta p \\
\Delta u &= 0 \\
\Delta \sigma' &= \Delta p - \Delta u \\
t = \infty & \\
\Delta \sigma &= \Delta p \\
\Delta u &= 0 \\
\Delta \sigma' &= \Delta p.
\end{align*}
\]

When a void ratio-stress curve is plotted on semilogarithmic graph paper, three important values can be obtained.

(1) The compression index, \( c_c \): This is the slope of the straight-line portion, and can be obtained as follows:
parameter to calculate the amount of settlement and a consolidation parameter to calculate the rate of settlement. The compression parameter is called "compression index" \( c_C \) and the consolidation parameter is called "coefficient of consolidation" \( c_v \). Both \( c_C \) and \( c_v \) are obtained from consolidation tests. The principle of consolidation is shown in Fig. 2.2. Letting \( d \) be a surcharge at the ground surface which causes the increase of total stress at any depth of the clay layer. It is believed that a time \( t = 0 \), the increase of effective stress is equal to zero \( \Delta \sigma' = 0 \) and the initial stress is taken by the excess pore water pressure, \( \Delta p = \Delta u \). As time passes, the water pressure is dissipated and the stress is transferred to the soil skeleton as effective stress. At \( t = \infty \), all the excess pore water pressure will be dissipated and \( \Delta u = 0 \) and \( \Delta \sigma' = \Delta p \), having caused gradual settlement when the stress was transferred from the water to the soil.

\[
\begin{align*}
t = 0 & \quad \Delta \sigma = \Delta p \\
\Delta u = (\Delta hi) & \gamma_w = \Delta p \\
\Delta \sigma' = \Delta \sigma - \Delta u & = \Delta p
\end{align*}
\]

When a void ratio-stress curve is plotted on semilogarithmic graph paper, three important values can be obtained.

1. The compression index, \( c_C \): This is the slope of the straight-line portion, and can be obtained as follows:
Figure 2.3. (b) Construction of field consolidation curve for over-consolidated clay.

2. Preconsolidation pressure, \( p_c \): Is the maximum past-effective overburden pressure to which the soil has been subjected. It can be obtained by the Casagrande method. The value obtained is compared with the current effective overburden pressure, \( p_0 \).

If

1. \( p_0 = p_c \) († about 10%) soil normally consolidated
2. \( p_0 > p_c \) specimen may have excessive disturbance
3. \( p_0 < p_c \) soil is over consolidated.

As already indicated, it is important to know what the state of consolidation of the soil is because that state plays an important role in the estimation of the shear strength.

3. Swelling Index, \( c_s \). Is the slope of the unloading portion of the e-log p curve. It is important in the estimation of settlement of over consolidated clays.
Consolidation Settlement (Amount)

The one-dimensional settlement, due to external loads can be obtained as follows:

Letting the volume of voids be equal to "e" and the volume of soil equal to 1, that is

\[ e = \frac{\nu_v}{\nu_s} \quad \text{if} \quad \nu_s = 1, \quad e = \nu_v, \]

then the initial volume of the soil \( v_0 \) is

\[ v_0 = \nu_s + \nu_v \]
\[ v_0 = 1 + e_0 \]

where \( e_0 \) = initial void ratio

Then, assuming that the settlement is caused by a change in the void ratio when the load is applied, the unit settlement is given by:

\[ s = \frac{v_f - v_0}{v_0} \quad \text{where} \quad v_f = \text{final volume.} \]

\[ s = \frac{v_f = 1 + e_{\text{final}}}{1 + e_0} \]
\[
\frac{\Delta e}{1 + e_0}
\]

If the stratum of soil has a thickness \( H_c \), the total settlement, \( s \) is equal to

\[
\frac{\Delta e}{1 + e_0}
\]

\[ (2.8) \]

It is apparent that the value to be computed is the \( \Delta e \) value, which is obtained from the \( e - \log p \) curve taking into account if the soil is normally consolidated or over consolidated. That can be observed in the following figure (Fig. 2.4).

When the soil is normally consolidated, an increment of pressure, \( \Delta p \) due to loads, is added to the existing overburden pressure \( p_0 \). Then from equation 2.7, the change in void ratio becomes: \( c = c_c \log \frac{p + \Delta p}{p_0} \) and substituting this value into equation 2.9, the settlement is expressed as:

\[
\frac{c_c H_c}{1 + e_0} \log \frac{p_0 + \Delta p}{p_0}
\]

or

\[
\frac{c_c H_c}{1 + e_0} \log \left(1 + \frac{\Delta p}{p_0}\right)
\]

\[ (2.10) \]

When the soil is over consolidated, two cases have to be considered:

(a) \( p_0 + \Delta p < p_c \) (see Fig. 2.4)
Again, \( \Delta e = c_s \log \frac{p + \Delta p}{p_0} \) \hspace{1cm} (2.11)

Figure 2.4. One-dimensional settlement calculations: (b) is for Eq. (2.10); (c) is for Eqs. (2.13) and (2.14).
and
\[
s = \frac{c_s H_c}{1 + e_o} \log \left(1 + \frac{\Delta p}{P_0}\right)
\]  
(2.12)

(b) \( P_0 + \Delta p > P_c \). Since the \( e-\log p \) curve shows two different slopes (see Fig. 2.4), the total settlement is expressed as the sum of the changes in void ratio called \( e_1 \) and \( e_2 \) -- that is,
\[
\Delta e = \Delta e_1 + \Delta e_2 \text{ or } \Delta e = c_s \log \frac{P_c}{P_0} +
\]
\[
c_c \log \left(1 + \frac{\Delta p}{P_0}\right)
\]
(2.13)

and the total settlement becomes:
\[
s = \frac{c_s H_c}{1 + e_o} \log \frac{P_c}{P_0} + \frac{c_c H_c}{1 + e_o} \log \frac{P_0 + \Delta p}{P_c}
\]  
(2.14)

Rate of Consolidation

As already indicated, the consolidation process is the result of dissipation of excess pore water pressure with time. Therefore, if the rate of that dissipation is known, the percent or degree of consolidation may be known. Terzaghi derived the following equation for a vertical drainage condition:
\[
\frac{\partial^2 (\Delta u)}{\partial t \partial z^2} = \frac{c_v \partial^2 (\Delta u)}{\partial z^2}
\]  
(2.15)
where \( \frac{\partial (\Delta u)}{\partial t} \) = rate of change of the excess pore water pressure

\[ \frac{\partial^2 (\Delta u)}{\partial z^2} \] = change in hydraulic pressure gradient

\[ c_v \frac{\partial^2 (\Delta u)}{\partial z^2} \] = amount of water expelled from the voids of the soil through one square unit area and during a time \( dt \).

The solution of the consolidation equation, Eq. (2.15), is based on the solution of Fourier’s equation in one dimension, a solution which must satisfy the given boundary conditions, that is \( \Delta u = 0 \) at \( z = 0 \), \( \Delta u = 0 \) at \( z = H_c = 2H \) if permeable layers are located at \( z = 0 \) and \( z = H_c \) (see Fig. 2.5).

![Figure 2.5](https://via.placeholder.com/150)

and at time \( t = 0 \), \( \Delta u = \Delta u_0 \) where \( \Delta u_0 \) is the initial pore
water pressure after load application. A solution for Eq. (2.15) is given by

\[ \Delta u = \sum_{m=0}^{\infty} \frac{2(\Delta u_0)}{M} \left[ \frac{M_2}{H} \exp(-M_2 T_v) \right] \]

(2.16)

If the initial pore water pressure is constant with depth, which is the case usually assumed in the conventional laboratory test. Where

\[ M = \frac{(2m + 1)}{2} \]

\[ m = \text{an integer} = 1, 2, \ldots \]

\[ z = \text{depth where the excess pore water pressure is computed} \]

\[ H = \text{length of maximum drainage path (two-way drainage condition)} \]

\[ T_v = \text{nondimensional time factor} = \frac{c_v t}{H^2} \]

The variation of \( u \) with time is shown in Fig. 2.6.

\[ \Delta u_1 = \text{pore water pressure at } t=t_1 \text{ and } z=h \]

\[ \Delta \sigma'_e = \text{effective pressure at } t=t_1 \text{ and } z=h \]

Figure 2.6.
It must be remembered that at any time and at any depth, the total stress is given by \( \Delta \sigma_t = \Delta \sigma_i + \Delta u \), then at
\[
\begin{align*}
t = 0 & \quad \Delta \sigma_t = \sigma + \Delta u = \Delta \rho \\
t = t_1 & \quad \Delta t = \Delta \sigma_i + \Delta u_1 = \Delta \rho \\
t = \infty & \quad \Delta \sigma_t = \Delta \sigma_i + \sigma = \Delta \rho
\end{align*}
\]

The concept of percent, or degree, of consolidation \( U \) is the ratio of settlement of a clay sample in the consolidometer \( s_t \) to its full or 100% settlement \( s_{\text{max}} \), after full consolidation at a given loading has been achieved. That is
\[
U = \frac{s_t}{s_{\text{max}}} \times 100 \quad (2.17)
\]

It is recognized that the pressure distribution area, shown in Fig. 2.6 is, in a certain way, a measure of the compression that the soil undergoes during the consolidation process. Therefore, integrating that area and assuming that the initial pore pressure is constant, is obtained,
\[
s_t = \int_0^{2H} (\Delta u_0) \, dz - \int_0^{2H} (\Delta u) \, dz \quad \text{(change in area in diagram of Fig. 2.6)}
\]
\[
s_{\text{max}} = \int_0^{2H} (\Delta u_0) \, dz \quad \text{(maximum area enclosed in diagram of Fig. 2.6)}
\]
\[
(\Delta u_0) \frac{2H}{(\Delta u_0) \frac{2H}{} - \int_0^{2H} \Delta u \, dz}
\]
\[
U = \frac{(\Delta u_0) \frac{2H}{(\Delta u_0) \frac{2H}{} - \int_0^{2H} \Delta u \, dz}}
\]
or
\[ U = 1 - \frac{\int_0^{2H} \Delta u \, dz}{(\Delta u_0) \, 2H} \]  \hspace{1cm} (2.18)

where \( \Delta u \) is given by Eq. 2.16).

If Eq. 2.16 is plugged into Eq. 2.18, it becomes

\[ U = 1 - \sum_{m=0}^{\infty} \frac{2}{M^2} \exp\left(-\frac{M^2 T_v}{M^2}\right) \]  \hspace{1cm} (2.19)

A curve of the time factor \( T_v \) versus degree of consolidation \( U \) is shown in Fig. 2.7.

Figure 2.7. Plot of time factor against average degree of consolidation (\( \Delta u_0 = \text{constant} \))
The value of $T_v$ is given by the following approximated relationships:

$$T_v = \frac{U}{4} \left( \frac{U}{100} \right)^2 \text{ for } U = 0 - 60\% \quad (2.20)$$

$$T_v = 1.781 - 0.933 \log (100 - U\%) \text{ for } U > 60\% \quad (2.21)$$

Then, knowing the value of $T_v$ for any percent of consolidation and the time $t$ at which that percent occurs, the value of the coefficient of consolidation is obtained as was indicated, that is

$$c_v = \frac{T_v H^2}{t}.$$ 

From the data obtained in the consolidation test, a time-settlement curve can be plotted in a semilogarithmic scale. Then the percent of consolidation can be fitted on that curve if the zero and 100% can be plotted. In order to do that, there are two methods available: the Casagrande’s method and the Taylor’s Square Root method. The Casagrande’s method is shown in Fig. 2.8 and can be explained in the following steps:

1. Select a time $t_1$ in the parabolic portion $(P_1)$

2. Select a time $t = \frac{t_1}{4}$ $(P_2)$

3. Obtain the offset between $t_1$ and $t$ (distance $A P_2 = d$)

4. Plot this distance above point $P_2$ $(P_3)$
5. Plot a horizontal line through P3 to obtain zero percent of consolidation line.

6. The above procedure should be repeated to obtain different zero lines to choose the best one.

7. The 100% line is found by intersecting two tangents (points s). One line is tangent to the primary branch of the curve and the other one to the secondary branch.

8. The distance between the zero and 100% is divided into equal parts, for instance each 10%.

Figure 2.8. Time-percent consolidation curve.

The value of $C_v$ is commonly calculated at the laboratory using the 50% of consolidation, that is
\[ C_v = \frac{0.197 H^2}{t_{50}} \]

Once the value of \( C_v \) has been obtained, the time at which certain percents of consolidation occurring at the field can be obtained by reversing the equation for \( C_v \). That is,

\[ t = \frac{T_v H_f}{C_v} \]

where \( H_f \) is the length of longest drainage path for a particle of water (half layer thickness when drainage is from both faces).

Finally, it must be said that the process of consolidation, due to the pore-water dissipation and the air expulsion, is called primary consolidation and that the soil still continues settling after this primary stage has been completed and the causes of that second, called secondary consolidation, are not known very well yet. Figure 2.9 shows the primary and secondary consolidation.

![Figure 2.9. Elements of time-settlement curve of a cohesive soil at constant load.](image-url)
Elastic Properties of the Soil

The modulus of elasticity $E_s$, the shear modulus $G$, and the Poisson’s ratio $\nu$, are the basic soil properties which are required to calculate elastic foundation settlements (no time dependent) and the modulus of subgrade reaction $k_s$. Although soils undergo deformation under loads like solid deformable bodies, they follow the Hooke’s law only within a narrow load interval, so it is very difficult to define elastic properties for them when they don’t behave as elastic materials. Many values have been suggested and the best that can be done is to establish possible ranges as shown in Table 2.6 and 2.7.

<table>
<thead>
<tr>
<th>Soil</th>
<th>$E_s$ (ksf)</th>
<th>$E_s$ (Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Very soft</td>
<td>50-250</td>
<td>2-15</td>
</tr>
<tr>
<td>Soft</td>
<td>100-500</td>
<td>5-25</td>
</tr>
<tr>
<td>Medium</td>
<td>300-1000</td>
<td>15-50</td>
</tr>
<tr>
<td>Hard</td>
<td>1000-2000</td>
<td>50-100</td>
</tr>
<tr>
<td>Sandy</td>
<td>500-5000</td>
<td>25-250</td>
</tr>
<tr>
<td>Glacial till</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>200-3200</td>
<td>10-153</td>
</tr>
<tr>
<td>Dense</td>
<td>3000-15 000</td>
<td>144-720</td>
</tr>
<tr>
<td>Very dense</td>
<td>10 000-30 000</td>
<td>478-1440</td>
</tr>
<tr>
<td>Loess</td>
<td>300-1200</td>
<td>14-57</td>
</tr>
<tr>
<td>Sand</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silty</td>
<td>150-450</td>
<td>7-21</td>
</tr>
<tr>
<td>Loose</td>
<td>200-500</td>
<td>10-24</td>
</tr>
<tr>
<td>Dense</td>
<td>1000-1700</td>
<td>48-81</td>
</tr>
<tr>
<td>Sand and gravel</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>1000-3000</td>
<td>48-144</td>
</tr>
<tr>
<td>Dense</td>
<td>2000-4000</td>
<td>96-192</td>
</tr>
<tr>
<td>Shale</td>
<td>3000-300 000</td>
<td>144-1400</td>
</tr>
<tr>
<td>Silt</td>
<td>40-400</td>
<td>2-20</td>
</tr>
</tbody>
</table>

Table 2.6. Typical range of values for the static stress-strain modulus $E_s$ for selected soils. Field values depend on stress history, water content, density, etc.
<table>
<thead>
<tr>
<th>Type of soil</th>
<th>$\mu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay, saturated</td>
<td>0.4–0.5</td>
</tr>
<tr>
<td>Clay, unsaturated</td>
<td>0.1–0.3</td>
</tr>
<tr>
<td>Sandy clay</td>
<td>0.2–0.3</td>
</tr>
<tr>
<td>Silt</td>
<td>0.3–0.35</td>
</tr>
<tr>
<td>Sand (dense)</td>
<td>0.2–0.4</td>
</tr>
<tr>
<td>Coarse (void ratio = 0.4–0.7)</td>
<td>0.15</td>
</tr>
<tr>
<td>Fine-grained (void ratio = 0.4–0.7)</td>
<td>0.25</td>
</tr>
<tr>
<td>Rock</td>
<td>0.1–0.4 (depends somewhat on type of rock)</td>
</tr>
<tr>
<td>Loess</td>
<td>0.1–0.3</td>
</tr>
<tr>
<td>Ice</td>
<td>0.36</td>
</tr>
<tr>
<td>Concrete</td>
<td>0.15</td>
</tr>
</tbody>
</table>

Table 2.7. Typical range of values for Poisson’s ratio.

And from *Principles of Mechanics of Materials*, the shear modulus $G$ is obtained as

$$G_s = \frac{E_s}{2(1 + \mu_s)}$$
CHAPTER III

SUBSURFACE EXPLORATION

There are many steps to carry out a soil investigation and they may vary from one project to another because of the familiarity with the site geology, local code requirements, the nature of the structure which is being planned and so on. A general procedure to be followed to perform a soil investigation could be:

1. Obtain as much information as possible about the proposed structure. That is, the magnitude and nature of loads and a general site plan.
2. Review available soil information from:
   a) Soil Conservation Service soil surveys.
   b) Geological maps.
   c) Previous engineering reports
   d) Topographic maps
3. Visit the site carrying a camera.
4. Design a tentative footing for the structure.
5. Lay out number of test holes.
6. Show depth of each bore hole (2B + Df. (minimum)).
7. Enumerate sampling and testing to determine bearing capacity and settlement.

a. Data Required

A soil report should include the following information:

1. Information to select the appropriate type of foundation
2. Information of the shear strength of the soil
3. Data to predict settlements
4. Ground water table location and its possible variations
5. Information regarding bedrock, if encountered
6. The boring logs drawn to a stated scale.

b. Presentation of Data

The boring data obtained from the soil investigation is usually presented on a sheet called "boring log." A typical boring record for one hole at a project is shown in Fig. 3.2.

c. Depth and Number of Borings:

There are not exact rules to be followed in order to know the number and depth of borings required in a project, each project must be considered individually. However, experience has shown that when uniform soil conditions are found, the number of holes should be four, allowing a boring at each corner of the proposed structure. The boring depth should be carried out to below the depth of foundation influence (usually taken where Boussinesq pressure-bulb analysis shows that the induced stress due to structural loads is on the order of 5 to 10% of the contact pressure). In other words, this means a depth of about 2-3 times footing width below the foundation for spread footings. When soils are unpredictable, it is recommended to drill a hole under each loaded point.
Possible boring location for regular stratified soil.

Possible depth of borings.

Figure 3.1
Figure 3.2. Boring log
CHAPTER IV

BEARING CAPACITY OF SHALLOW FOUNDATIONS

Introduction

According to Terzaghi, a footing can be considered shallow if the depth $Z$ (Fig. 4.2) of the foundation is less than or equal to the width $B$ of the foundation. He noticed that when a vertical static or transient load is applied on the footing and results are plotted on a load-settlement curve, the shape of this curve depends mainly on the composition of the supporting material. That is, if the soil is fairly dense or stiff, the footing fails by shear following a well-defined failure pattern consisting of a continuous slip surface from one edge of the footing to the ground surface. Usually, the failure is sudden and catastrophic and the movement is similar to the curve shown in Fig. 4a. This type of failure is called General Shear Failure. If the soil is of medium compaction, an increase of load will also be accompanied by an increase of movement, the settlement curve does not show a well-defined peak-load value (see Fig. 4.5). There is no catastrophic collapse or tilting of the footing and the failure pattern is well defined but only after considerable vertical displacements of the footing $(\frac{Z}{B} \leq \frac{1}{2})$ may be the slip surfaces appearing in the ground.
This type of failure is called Local Shear Failure. Vesic (1963) observed that if the soil is a fairly loose soil, the load-settlement curve is similar to that shown in Fig. 4c, there is now a well-defined failure pattern. Except for sudden small movements of the foundation in the vertical direction, there is neither visible collapse nor substantial tilting. The failure surface will not extend to the ground surface. This type of failure is called Punching Shear Failure. Vesic (1973) developed a relationship to predict the mode of failure of footings resting on sands (see Fig. 4d) where

\[ D_r = \text{relative density} \]

\[ D_f = \text{depth of foundation} \]

\[ B^* = \frac{2BL}{B+L} \]

\[ B = \text{width of footing} \]

\[ L = \text{length of footing} \]

b. Definition

Bearing capacity of the soil can be defined as the maximum contact pressure which can be imposed on a soil without causing shear failure or excessive settlement, therefore, it is called ultimate bearing capacity. Based on the theory of plastic equilibrium, Prandtl developed a theory to explain the process of penetration of hard bodies into another softer, homogeneous, isotopic material and obtained
Nature of bearing capacity in soil:  (a) General shear failure; (b) Local shear failure; (c) Punching shear failure (redrawn after Vesie, 1973).

Figure 4. Modes of foundation failure in sand (after Vesie, 1973).
Figure 4.1. Prandtl's system of study.

an expression for the ultimate compressive stress ($\sigma_u$) that can be imposed on the softer material, that is

$$\sigma_u = \frac{c}{\tan \phi} \left[ \tan^2 (45^\circ + \phi) e^{2\sigma_t \tan \phi} - 1 \right]$$  \hspace{1cm} (4.1)

The system used by Prandl is shown in Fig. 4.1 and can be applied to calculate soil-bearing capacity with some variations, as shown in Fig. 4.1.a.

It can be noted in equation 4.1 that the bearing capacity becomes zero when the cohesion "c" is zero. Terzaghi prevented that introducing a new value which takes into account the weight of the soil and adding to the original c value a new one, $c'$, where $c' = \gamma t \tan \phi$ and $t = \text{equivalent height of surcharge of soil given by}$


\[ t = \frac{\text{area of wedges and sector}}{2} \] (one half of the system) 

\[ T_i \text{ soil unit weight}. \]

\[ \sigma_u = \frac{c + c'}{\tan \phi} \left[ \tan^2(45^\circ + \phi/2) e^{n\tan - 1} \right] \]

Now, if \( c = 0 \), the \( \sigma_u = 0 \).

---

Figure 4.1(a) Prandtl's system for a (\( \phi-c \)) soil.

---

c. Terzaghi's Bearing Capacity Equation

Later, Terzaghi, based on Prandtl's theory, developed new expressions using a different system (see Fig. 4.2). Terzaghi placed the strip footing not on the ground surface but below it and considered a rough base as they really are. Using the free body shown in Fig. 4.2, the sum of the vertical forces must be zero if they are in equilibrium. The
analysis can be made taking into account the weight of the soil within ADB or disregarding it. If the weight mentioned is disregarded, the equilibrium of the strip footing requires that

\[ Q_{\text{crit}} = 2P_p + B c \tan \phi. \]  
(4.3)

where

- \( P_p \) = passive pressure
- \( B \) = footing width
- \( \phi \) = friction angle
- \( c \) = cohesion.

This equation represents the solution of the problem where \( P_p \) is known.

The passive earth pressure required to cause a slip on DEF can be divided into two parts, \( p_1 \) and \( p_2 \). The force \( p_1 \) is the resistance due to the weight of BDEF and its point of
application is the lower third point of BD. The force p₂ is composed of two parts. One is due to cohesion Pₐ and the other is due to surcharge γz, Pq. Pₐ and Pq are uniformly distributed, then its point of application is located at the middle of BD.

Now, the equation 4.3 is arranged as

\[ Q_{\text{crit}} = 2(p₁ + pₐ + pq) + BC \tan \phi \]

where the passive pressures are given by

\[ Nₐ = \frac{2pₐ}{BC} + \tan \phi \]

\[ Nq = \frac{2pq}{γzB} \]

\[ N = \frac{4p₁}{γB^2} \]

plotting that values into the above equation is obtained.

\[ Q_{\text{crit}} = B(cNₐ + γzNq + 1/2 γBNq) \quad (4.4) \]

the values Nₐ, Nq and Nq are called the "bearing capacity factors and they are dimensionless quantities which depend only on the value of \( \phi \). These values are shown in Table 4.1.

where

\[ Nₐ = \cot \phi \left[ \frac{e^{2(3\pi/4 - \phi/2)\tan \phi}}{2 \cos^2(\frac{\pi}{2} + \phi/2)} \right] - 1 \quad (4.5) \]

\[ Nq = \frac{e^{2(3\pi/4 - \phi/2) \tan \phi}}{2 \cos^2(45 + \phi/2)} \quad (4.6) \]

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\[ N_y = \frac{1}{2} \frac{K_{p\gamma}}{(------ - 1) \tan \phi} \]

\( K_{p\gamma} = \text{passive pressure coefficient.} \)

Values of \( N_y \) for \( \phi \) of 34 and 48° are original Terzaghi values and used to back-compute \( K_{p\gamma} \) for forward computations of \( N_y \) by author

<table>
<thead>
<tr>
<th>( \phi, \text{deg} )</th>
<th>( N_x )</th>
<th>( N_z )</th>
<th>( N_y )</th>
<th>( K_{p\gamma} )</th>
</tr>
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<tbody>
<tr>
<td>0</td>
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<td>1.0</td>
<td>0.0</td>
<td>10.8</td>
</tr>
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<td>7.3</td>
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<td>0.5</td>
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</tr>
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<td>2.5</td>
<td>18.6</td>
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<td>7.4</td>
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<tr>
<td>50</td>
<td>347.5</td>
<td>415.1</td>
<td>1153.2</td>
<td>800.0</td>
</tr>
</tbody>
</table>

Table 4.1. Bearing capacity factors for the Terzaghi equations.

It is necessary to mention that Terzaghi developed the above equation assuming a general shear failure and basically he suggested that the failure zone under the foundation be divided into three parts (see Fig. 4.2):

1. A triangular Rankin Active Zone (Zone ABD)
2. Two radial shear zones BDE and ADG, being the curves PE and DG arcs of a logarithmic spiral.
3. Two triangular Rankin passive zones BEF and AGD.

The wedge ABD is pushed downward by the load footing and
shear resistance is developed along planes BD and AD from the soil cohesion and a friction resistance develops along the slip develops along the slip surfaces DEF or DGI to produce the passive force pp mentioned above. When the footing is not an infinite strip, the plane wedge becomes a cone for circular footing and pyramid-shape for square and rectangular footings. Terzaghi, based on experiments, suggested the following semiempirical values

\[
\begin{align*}
\text{Square } q_u &= 1.3 N_c + \gamma D_f + 0.4 \gamma B N_f \\
\text{Round } q_u &= 1.3C N_c + \gamma D_f + 0.3 \gamma B N_f
\end{align*}
\]

(4.8) (4.9)

d. Meyerhof's Bearing Capacity Equation

The ultimate bearing capacity equations shown in Eqs. (4.4), (4.8) and (4.9) are used for continuous, square and round footings. Meyerhof and Hansen suggested factors which can be applied to the Terzaghi equations to take into account depth and shape when the footing is not infinitely long and also to take into account inclined load effect. The equation suggested by Meyerhof is:

\[
q = C N_c F_{cs} F_{cd} F_{ci} + \gamma D_f N_f F_{qd} F_{q_i} F_{qs} + 1/2 \gamma B N_f F_{fs} F_{fd} F_{pi}
\]

(4.10)

where

\[c = \text{cohesion}\]

\[\gamma D_f = q = \text{effective stress at the level of the bottom of foundation}\]

\[\gamma = \text{unit weight of the soil}\]
B = width of foundation (diameter for a circular footing)

\( F_{cs}, F_{qs}, F_{ys} = \) shape factors

\( F_{cd}, F_{qd}, F_{yd} = \) depth factors

\( F_{ci}, F_{qi}, F_{yi} = \) load inclination factors

If the Meyerhof equation is used, the bearing capacity factors must be modified because the angle between AB and AD which was considered equal to \( \phi \) by Terzaghi is considered equal to \( 45 + \frac{\phi}{2} \) by Meyerhof and those factors become:

\[
N_q = e^{\pi \tan \phi \tan^2 \left( 45^\circ + \frac{\phi}{2} \right)} \tag{4.11}
\]

\[
N_c = (N_q - 1) \cot \phi \tag{4.12}
\]

\[
N = (N_q - 1) \tan (1.4\phi) \tag{4.13}
\]

Modified bearing capacity factors are shown in Table 4.2.

---

Table 4.2 Bearing-capacity factors for the Meyerhof bearing-capacity equations

Note that \( N_c \) and \( N_q \) are same for both equations

<table>
<thead>
<tr>
<th>( \phi ), deg</th>
<th>( N_c )</th>
<th>( N_q )</th>
<th>( N_r )</th>
<th>( 2 \tan (1 - \sin \phi)^2 )</th>
<th>( N_r )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>5.14</td>
<td>1.0</td>
<td>0.19</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>6.50</td>
<td>1.6</td>
<td>0.24</td>
<td>0.15</td>
<td>0.1</td>
</tr>
<tr>
<td>10</td>
<td>8.30</td>
<td>2.5</td>
<td>0.30</td>
<td>0.24</td>
<td>0.4</td>
</tr>
<tr>
<td>15</td>
<td>11.80</td>
<td>3.9</td>
<td>0.36</td>
<td>0.29</td>
<td>1.1</td>
</tr>
<tr>
<td>20</td>
<td>14.80</td>
<td>6.4</td>
<td>0.43</td>
<td>0.32</td>
<td>2.9</td>
</tr>
<tr>
<td>25</td>
<td>20.70</td>
<td>10.7</td>
<td>0.51</td>
<td>0.31</td>
<td>5.8</td>
</tr>
<tr>
<td>30</td>
<td>30.10</td>
<td>18.4</td>
<td>0.61</td>
<td>0.29</td>
<td>15.7</td>
</tr>
<tr>
<td>35</td>
<td>46.10</td>
<td>33.3</td>
<td>0.72</td>
<td>0.25</td>
<td>37.1</td>
</tr>
<tr>
<td>40</td>
<td>75.30</td>
<td>64.2</td>
<td>0.85</td>
<td>0.21</td>
<td>33.7</td>
</tr>
<tr>
<td>45</td>
<td>133.90</td>
<td>134.9</td>
<td>1.01</td>
<td>0.17</td>
<td>252.7</td>
</tr>
<tr>
<td>50</td>
<td>266.90</td>
<td>319.3</td>
<td>1.20</td>
<td>0.13</td>
<td>873.7</td>
</tr>
</tbody>
</table>
Shape factors can be obtained as

\[ F_{cs} = 1 + \frac{B}{L} \frac{Nq}{N_c} \]  \hspace{1cm} (4.14)

\[ F_{qs} = 1 + \frac{B}{L} \tan \]  \hspace{1cm} (4.15)

\[ F_{rs} = 1 - 0.4 \frac{B}{L} \]  \hspace{1cm} (4.16)

L = length of foundation \((L > B)\)

Depth factors can be obtained as:

\[ F_{cd} = 1 + 0.4 \left(\frac{D_f}{B}\right) \quad \text{for } D_f/B < 1 \]  \hspace{1cm} (4.17)

\[ F_{qd} = 1 + 2 \tan \left(1 - \sin \phi\right)^2 \frac{D_f}{B} \quad \text{for } D_f/B < 1 \]  \hspace{1cm} (4.18)

\[ F_{rd} = 1 \]  \hspace{1cm} (4.19)

for \(D_f/B > 1\)

\[ F_{cd} = 1 + 0.4 \tan^{-1} \left(\frac{D_f}{B}\right) \]  \hspace{1cm} (4.20)

\[ F_{qd} = 1 + 2 \tan \phi \left(1 - \sin \phi\right)^2 \tan^{-1} \left(\frac{D_f}{B}\right) \]  \hspace{1cm} (4.21)

\[ F_{rd} = 1 \]  \hspace{1cm} (4.22)

the factor \(\tan^{-1} \left(\frac{D_f}{B}\right)\) is in radians.

And, the inclination factors can be obtained as

\[ F_{ci} = F_{qi} = (1 - \frac{\phi}{90})^2 \]  \hspace{1cm} (4.23)
\[ F_{y_i} = (1 - \frac{B^0}{\phi})^2 \]  \hspace{1cm} (4.24)

where \( B^0 \) is the inclination of the load on the foundation with respect to the vertical.

e. Modification of Bearing Capacity Equations for Water Table

Normally, the bearing capacity equations are developed assuming that the water table is located well below the foundation. But, when it is close to the surface, some modifications must be made to the bearing capacity equation so that the surcharge pressure can be an effective value as the original equations are developed.

According to the location, three possible cases can be analyzed (see Fig. 4.3):

a) Water table located at \( 0 \leq D_1 \leq D_f \)

The factor \( q \) must be changed to:

\[ q = D_1 \gamma + D_2 (\gamma_{sat} - \gamma_w) \]  \hspace{1cm} (4.25)

where

\( \gamma_{sat} = \) saturated unit weight of the soil

\( \gamma_w = \) unit weight of water

The value of \( \gamma \) in the last term of the bearing capacity equation must be changed to:

\[ \gamma' = \gamma_{sat} - \gamma_w = \gamma_{submerged} \]  \hspace{1cm} (4.26)

b) Water table at \( 0 \leq d \leq B \).

The factor \( q \) remains the same, that is
\[ q = \gamma D_f \]  \hspace{1cm} (4.27)

The factor \( \gamma \) in the last term of the equation must be changed to

\[ \gamma = \gamma' + \frac{d}{B} (\gamma - \gamma') \]  \hspace{1cm} (4.28)

c) Water table at \( d \geq B \).

The factors remain the same.

Figure 4.3. Modification of bearing capacity equations for water table.

The above modifications do not take into account seepage forces.
f. Some Special Cases of Bearing Capacity

f.1. Eccentrically Loaded Footings

When footings are submitted to moments which cause eccentricity either with respect to one axis or both, the pressures beneath those footings are no longer uniform and their distribution can be calculated by the mechanics of material expression.

\[
q = P\left(\frac{e_x x}{A} \right) - \frac{e_y y}{I_y} \quad (4.29)
\]

where

- \( P \) = vertical load
- \( A \) = area of footing
- \( e_y, e_x \) = eccentricity of \( P \) with respect to \( y \) and \( x \)
- \( I_x, I_y \) = moment of inertia of plan area with respect to \( x \) and \( y \)
- \( x, y \) = points where soil pressure is calculated.

Meyerhof (1953) suggested a procedure to modify the bearing capacity equation for those types of footing. Basically, he reduced the actual length and width of the footing as follows (see Fig. 4.4):

- \( L' = L - 2e_x \)
- \( B' = B - 2e_y \)

where \( L' \) and \( B' \) are the effective length and the effective width of the footing respectively.

The ultimate bearing capacity equation, using the Meyerhof equation, is obtained using the effective dimensions
$L'$, $B'$ to evaluate the shape factors and to replace $B'$ by $B$ in the $\gamma BN_x$ terms. To obtain the depth factors, the actual value of $B$ must be used.

The ultimate load is obtained as

$$P_{ult} = q_u B' L'$$

(4.30)

Figure 4.4. Method of computing effective footing dimensions when footing is eccentrically loaded.

**f.2 Foundations in Layered Soils**

The ultimate bearing capacity equation might be modified when footings are placed on layered soils and the thickness of the stratum where the footings are resting do not enclose the rupture zone (zone DEF, Fig. 4.2). Reddy and Srinivasan (1967) developed an expression for bearing capacity on layered clay soils. They assumed $\phi = 0$ (undrained conditions), becoming the curve sector of Fig. 4.2, circular (see Fig. 4.5) and defined the anisotropy of the soil by
means of a coefficient $K$, which was the ratio of the vertical shear strength to the horizontal shear strength of the soil. A value of $K < 1$ means overconsolidation, $K = 1$ means that the soil is isotropic, and $K > 1$ is a normally consolidated soil. Charts are available to compute the value of bearing capacity factor, $N_c$, which is no longer constant, those charts are plotted for different values of $K$. However, Meyerhof and Brown (1967) suggest that the chart for $K = 1$ is enough to obtain the value of $N_c$, that chart is shown in Fig. 4.6.

The bearing capacity for the layered clay soil conditions of Fig. 4.5 is obtained from the Meyerhof equation (4.10):

$$q_u = C_{N_c} F_{cs} F_{cd} F_{ci} + \gamma D_f N_q F_{qs} F_{qd} F_{qi}$$

$$+ \frac{1}{2} \gamma B N_f F_{rs} F_{rd} F_{ri}$$

If the shape, depth, inclination and bearing capacity factors are obtained, as indicated before, the above equation becomes

$$q_u = C_1 N_c F_{cs} F_{cd} + \gamma D_f$$  \hspace{1cm} (4.31)

If $\phi_1 = 0$. 

![Figure 4.5](image-url)

Figure 4.5.
g. Safety Factor in Foundation Engineering

The complexity of the soil behavior, the difficulty to determine either in the field or in the laboratory exact values for soil properties, the change in those properties before, during and after construction, the variability of the soil and so on, are among others, some of the problems to evaluate the bearing capacity of the soil. It must be remembered that the bearing capacity expressions developed above are based on mathematical models which are supported by experimental data. Therefore, the value obtained for ultimate bearing capacity of the soil must be reduced by a factor called "safety factor." This is not a constant value, it may vary from foundation type to expected failure mode. As a manner of guide, common values are shown in Table 4.3.
In Table 4.4 is shown the loads that can be expected to act on a structure. The codes suggest what combination of these loads can be critical.

<table>
<thead>
<tr>
<th>Failure mode</th>
<th>Foundation type</th>
<th>( F )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear</td>
<td>Earthworks</td>
<td>1.2-1.6</td>
</tr>
<tr>
<td></td>
<td>Dams, fills, etc.</td>
<td></td>
</tr>
<tr>
<td>Shear</td>
<td>Retaining structure</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Walls</td>
<td>1.5-2.0</td>
</tr>
<tr>
<td></td>
<td>Sheetpiling, cofferdams</td>
<td>1.2-1.6</td>
</tr>
<tr>
<td></td>
<td>Braced excavations</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(temporary)</td>
<td>1.2-1.5</td>
</tr>
<tr>
<td>Shear</td>
<td>Footings</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Spread</td>
<td>2-3</td>
</tr>
<tr>
<td></td>
<td>Mat</td>
<td>1.7-2.5</td>
</tr>
<tr>
<td></td>
<td>Uplift</td>
<td>1.7-2.5</td>
</tr>
<tr>
<td>Seepage</td>
<td>Uplife, Heaving</td>
<td>1.5-2.5</td>
</tr>
<tr>
<td></td>
<td>Piping</td>
<td>3-5</td>
</tr>
<tr>
<td>Load</td>
<td>Includes</td>
<td></td>
</tr>
<tr>
<td>----------------------</td>
<td>--------------------------------------------------------------------------</td>
<td></td>
</tr>
<tr>
<td>Dead load (DL)</td>
<td>Weight of structure and all permanently attached material</td>
<td></td>
</tr>
<tr>
<td>Live load (LL)</td>
<td>Any load not permanently attached to the structure, but to which the structure may be subjected</td>
<td></td>
</tr>
<tr>
<td>Snow load (S)</td>
<td>Acts on roofs; value to be used generally stipulated by codes</td>
<td></td>
</tr>
<tr>
<td>Wind loads (W)</td>
<td>Acts on exposed parts of structure</td>
<td></td>
</tr>
<tr>
<td>Earthquake (E)</td>
<td>A lateral force (usually) which acts on the structure</td>
<td></td>
</tr>
<tr>
<td>Hydrostatic (HS)</td>
<td>Any loads due to water pressure and may be either (+) or (-)</td>
<td></td>
</tr>
<tr>
<td>Earth pressure (EP)</td>
<td>Any loads due to earth pressures—commonly lateral but may be in other directions</td>
<td></td>
</tr>
</tbody>
</table>
h. Bearing Capacity from Standard Penetration Test

Being the standard penetration test commonly used, many efforts have been made to evaluate bearing capacity of soil using penetration record (N). Experience has proved that this is adequate only for cohesionless soils.

Teng (1962) proposed the following equation to evaluate the allowable bearing capacity

\[
(K_{sf}) q_a = 720(N-3)\left(\frac{B+1}{2B}\right)^2 w' K_d
\]

(4.32)

where

- \( q_a \) = allowable bearing pressure
- \( N \) = standard penetration number
- \( B \) = width of foundation (least dimension)
- \( w' \) = water correction factor (0.5 if ground water table is within \( B \) below the base of the footing; 1.0 otherwise)
- \( K_d = 1 + \frac{D}{2.0 B} \)
- \( D \) = depth of foundation.

Meyerhof suggested less conservative values given as

\[
(K_{sf}) q_a = \frac{N}{2.5} w' K_d \text{ for } B \leq 4'
\]

(4.33)

\[
(K_{sf}) q_a = \frac{N}{4} \left(\frac{B+1}{B}\right)^2 w' K_d \text{ for } B > 4'
\]

(4.34)

Gibbs and Holtz proposed a set of curves to obtain the relative density of a soil given the standard penetration
resistance "N" and the vertical effective stress. These curves are shown in Fig. 4.7. The average effective pressure should be computed at the depth \( D_f + \frac{B}{2} \).

Figure 4.7. Correlations Between Relative Density and Standard Penetration Resistance in Accordance with Gibbs and Holz.
Once the relative density \( D_r \) is known, the angle of friction of the soil can be determined from Fig. 4.8 when the dry unit weight \( \bar{\gamma}_d \) is given and the soil is classified according to the unified system. The value of the friction angle \( \phi \) obtained above can be used in the Terzaghi equations to obtain the allowable bearing capacity.

Figure 4.8. Correlations of Strength Characteristics for Granular Soils.
CHAPTER V

SETTLEMENTS OF SHALLOW FOUNDATIONS

a. Types of Foundation Settlements

Foundation settlements are difficult to predict. It is understood that the soil settlements are provoked by the increase of stress in soil due to the imposed loads, but those loads act on a material which is not homogeneous, and its properties are variable at any direction. Therefore, only reasonable values can be expected when soil settlements are estimated.

In general, the total settlement $S$ of a foundation can be expressed as the sum of an immediate settlement $S_e$, a primary consolidation settlement $S_c$, and a secondary consolidation settlement $S_s$, that is

$$S = S_e + S_c + S_s$$  \hfill (5.1)

The immediate settlement or elastic settlement is that which occurs as soon as the load is applied (0-7 days). In granular soils, which have a large coefficient of permeability, it is the predominant part of the settlement. This type of settlement may be obtained by using theory of elasticity, by empirical correlations and by using the method called "strain influence factor."

The primary consolidation settlement is that which is time-dependent and takes place as a result of dissipation of
the pore water from the void spaces. It is the type of settlement which predominates in all saturated or nearly saturated fine-grained soils which have a \( K \) on the order of \( 10^{-6} \) or less. It can be obtained using the consolidation principles mentioned in Chapter II. The secondary consolidation settlement which continues after the water has been expelled. The reason for that settlement is not clear yet, some authors believe that it is because a colloid-chemical process which becomes active and others believe that it is due to a continued readjustment of clay particles. It may be calculated by

\[
S_s = C \frac{H_{ts} \log \frac{t}{t_p}}{\Delta H_t/H_t} \tag{5.2}
\]

where \( C = \frac{\Delta H_t/H_t}{\log t_2 - \log t_1} \), coefficient of secondary consolidation (slope of the curve between \( t_1 \) and \( t_2 \), see Fig. 5.1).

\( H_{ts} = \) thickness of clay layer at the beginning of secondary consolidation, \( H_{ts} = H_t - S_c \)

\( t = \) time at which secondary consolidation is required

\( t_p = \) time at end of primary consolidation

\( H_t = \) thickness of clay layer

\( \Delta H_t = \) change in thickness of clay layer during secondary consolidation
b. Elastic Settlement

The elastic settlement calculated by means of the theory of elasticity is expressed as

\[ S_{e1} = q B \frac{1 - \mu^2}{E_s} I_w \]  \hspace{1cm} (5.3)

where

- \( S_{e1} \) = elastic settlement
- \( q \) = intensity of contact pressure, in units of \( E_s \)
- \( B \) = width of foundation
- \( I_w \) = Influence factor which depends on shape and rigidity of the footing (see Table 5.1)
- \( E_s, \mu \) = elastic properties of the soil (see Table 2.6 and Table 2.7)

Equation 5.3 can be applied when the footing is placed on the ground surface of a semi-infinite half-space, \( D_f = 0 \). When \( D_f = 0 \) (common practice) and the soil depth to the bedrock is finite, the equation 5.3 should be modified as
\[ S_e = q \beta' \frac{1 - \mu^2}{E_s} \left( F_1 + \frac{1 - 2\mu}{1 - \mu} F_2 \right) \] (5.4)

where

\[ F_1 = \frac{1}{\pi} \left[ M \ln \frac{(1 + \sqrt{M^2 + 1})(\sqrt{M^2 + N^2})}{M(1 + \sqrt{M^2 + N^2} + 1)} \right. \]
\[ + \ln \frac{(M + \sqrt{M^2 + 1}) \sqrt{1 + N^2}}{M + \sqrt{M^2 + N^2} + 1} \]

\[ F_2 = \frac{N}{2\pi} \tan^{-1} \left( \frac{M}{N \sqrt{M^2 + N^2} + 1} \right), \tan^{-1} \text{ in rads} \]

\[ M = \frac{L'}{B'}, \quad N = \frac{H}{B'}, \quad B' = \frac{B}{2} \quad \text{for center} \]

\[ B' = B \text{ for center} \]

\[ L' = \frac{L}{2} \quad \text{for center} \]

\[ L' = L \text{ for corner}. \]

Fox (1948) proposed a correction for depth that can be applied either to equation 5.3 or equation 5.4. That is

\[ S_e = S_1 \text{ or } 2 \ (F_3) \]

The values of \( F_3 \) can be obtained from Figure 5.2.

It can be noted in Table 5.1 that the influence factors are given for flexible and rigid foundations. It is generally assumed that footings are flexible. In practice, no footing is perfectly flexible, nor is it infinitely rigid. Borowicka (1936, 1938) found that the distribution of the
contact pressure underneath the footing depended on the elastic properties of both the footing and the soil on which the footing is placed. He found that such distribution depended on a non-dimensional factor for $K_r$ which is expressed as

$$
K_r = \frac{1}{5} \left( \frac{1 - \mu_s^2}{1 - \mu_f^2} \right) \frac{E_f}{E_s} \frac{T}{b^3}
$$

where $\mu_s$, $\mu_f$ are Poisson's ratio of soil and footing, respectively.

$E_s$, $E_f$ are Modulus of elasticity of soil and footing, respectively.

$T$ = thickness of the footing.

$b = \frac{B}{2}$ for strip and square footings, radius for circular footing.
Table 5.1 Influence factors $I_w$ for various-shaped members and for flexible and rigid footings

<table>
<thead>
<tr>
<th>Shape</th>
<th>Flexible Center</th>
<th>Flexible Corner</th>
<th>Average</th>
<th>Rigid $I_w$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Circle</td>
<td>1.00</td>
<td>0.64 (edge)</td>
<td>0.85</td>
<td>0.88</td>
</tr>
<tr>
<td>Square</td>
<td>1.12</td>
<td>0.56</td>
<td>0.95</td>
<td>0.82</td>
</tr>
<tr>
<td>Rectangle</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>L/B = 0.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td>1.35</td>
<td>0.68</td>
<td>1.15</td>
<td>1.06</td>
</tr>
<tr>
<td>2.0</td>
<td>1.53</td>
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<td>10.0</td>
<td>2.54</td>
<td>1.27</td>
<td>2.25</td>
<td>2.10</td>
</tr>
<tr>
<td>100.0</td>
<td>4.01</td>
<td>2.00</td>
<td>3.69</td>
<td>3.40</td>
</tr>
</tbody>
</table>

Figure 5.2. Influence factor for footing at a depth $D$. Use actual footing width and depth dimension for $D/B$ ratio.
The model used by Borowicka was footings resting on a semi-infinite elastic mass. In Fig. 5.3 the distribution of contact stress that he obtained can be observed. Being the soil a nonperfect elastic material and so variable in its characteristics, that distribution will be somewhat different.

Figure 5.3. Contact stress over rigid foundations resting on an elastic medium. (a) Circular foundation. (b) Strip foundation.
depending on the type of material. For instance, a flexible footing resting on saturated clay, \( \phi = 0 \) will perform as shown in Fig. 5.4a. A rigid footing resting on the same clay will perform as shown in Fig. 5.4b. If a flexible footing is placed on sand \( (c = 0) \), the pressure distribution and settlement profile will look as shown in Fig. 5.4c and, finally, if a rigid footing is placed in the same sand, it will behave as shown in Fig. 5.4c.

![Figure 5.4](image)

**Figure 5.4.** Flexible (a) and rigid (b) foundations on clay; flexible (a) and rigid (b) foundations on sand.

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Example 5.1

A square tank is shown in Fig. 5.5. Assuming flexible loading conditions, find the average elastic settlement of the tank for the following conditions.

a) $D_f = 0$  \hspace{1cm} H = \\
b) $D_f = 4.5'$  \hspace{1cm} H = 30'

Solution

a) Since $D_f = 0$ and $H =$ , equation 5.3 can be applied.

$$Se_1 = q \frac{B}{E_s} \frac{1 - 2}{I_w}$$

$I_w$ average = 0.95 from Table 5.1

$$Se_1 = 2(10) \frac{1 - 0.3}{440} = 0.039 \text{ ft} = 0.47''$$

b) $D_f = 4.5'$  \hspace{1cm} H = 30'$
Using Eq. 5.4

\[ S_{e2} = q \frac{1 - 2}{B'} \left( \frac{F_1 + 1 - 2}{E_s} \right) \]

\[ B' = \frac{B}{2} \text{ (center), } \frac{B'}{2} = 5' , \frac{M}{B'} = \frac{5'}{5} = 1 \]

\[ L' = \frac{L}{2} \text{ (center), } \frac{L'}{2} = 5' , \frac{N}{L'} = \frac{30}{5} = 6 \]

\[ F_1 = \frac{1}{\ln \left( \frac{1 + \sqrt{12} + 1}{1(1 + \sqrt{12} + 6^2 + 1)} \right) \left( \frac{1 + \sqrt{12} + 1}{1 + \sqrt{12} + 6^2 + 1} \right) + \ln \left( \frac{1 + \sqrt{12} + 1}{1 + \sqrt{12} + 6^2 + 1} \right) } = 0.46 \]

\[ F_2 = \frac{6}{2} \tan^{-1} \left( \frac{1}{6\sqrt{1 + 6^2 + 1}} \right) = \]

\[ F_2 = \frac{6}{2} \times 0.027 \text{ rad} = 0.026 \]

Then

\[ S_{e2} = 2(5)(2) \frac{1 - 0.3}{440} \left( \frac{0.46 + 1 - 2(0.3)}{1-0.3} \right) = 0.0196 \text{ (two times } F_1 \text{ and } F_2 \text{ because of four corner distributions)} \]

and

\[ S_{e3} = 0.24'' \text{ (from Fig. 5.2) } \frac{F_3 = 0.24'' \times 0.77 = 0.19''}{center} \]

\[ F_3 = 0.77 \text{ D/B = 0.45} \]

\[ L/B = 1, = 0.3 \]
c. Consolidation Settlement

The principles of consolidation shown in Chapter II can be used to obtain settlements due to consolidation.

Example 5.2

Calculate the settlement of the 10' thick clay layer shown in Fig. 5.5 caused by the load carried by a square footing of size 5 x 5'.

![Diagram of soil layers with G.W.T., 200 k., 5' x 5', Dry sand, Y_d = 100 pcf., Sat = 120 pcf., Normally consolidated clay, Y_sat = 110 pcf., e_0 = 1.0, c_c = 0.3'].

Figure 5.6

The settlement can be obtained using the equation 2.10

\[ S = \frac{C_c H_c}{1 + e_0} \log \left(1 + \frac{\Delta P}{P_0}\right) \]

Better results can be obtained if the clay layer is divided into several layers (1 or 2' in thickness). Then the induced stress (\(\Delta P\)) and the effective stress (\(P_0\)) are calculated at
the middle of each layer. The total consolidation settlement is the sum of the settlement of each layer.

Assuming two-foot layers:

Calculation of the effective stress:

\[ P_{01} = 100 \times 10 + (120 - 62.4) \times 10 + (110 - 62.4) \times 1 \]
\[ = 1623.6 \text{ Psf.} \]

\[ P_{02} = 100 \times 10 + (120 - 62.4) \times 10 + (110 - 62.4) \times 3 \]
\[ = 1718.8 \text{ Psf.} \]

\[ P_{03} = 110 \times 10 + (120 - 62.4) \times 10 + (110 - 62.4) \times 5 \]
\[ = 1814 \text{ Psf.} \]

\[ P_{04} = 110 \times 10 + (120 - 62.4) \times 10 + (110 - 62.4) \times 7 \]
\[ = 1909.2 \text{ Psf.} \]

\[ P_{05} = 110 \times 10 + (120 - 62.4) \times 10 + (110 - 62.4) \times 9 \]
\[ = 2004.4 \text{ Psf.} \]

Calculation of the induced stress (P).

There are several methods available, the 2:1 method will be used for simplicity.
\[ \Delta P = \frac{P}{(B+z)^2} \]

\[ P_1 = \frac{2000000}{(5+18)^2} = 378.07 \text{ Psf} \]

\[ P_2 = \frac{200000}{(5+20)^2} = 320 \text{ Psf} \]

\[ P_3 = \frac{200000}{(5+22)^2} = 274.35 \text{ Psf} \]

\[ P_4 = \frac{200000}{(5+24)^2} = 237.81 \text{ Psf} \]

\[ P_4 = \frac{200000}{(5+26)^2} = 208.12 \text{ Psf} \]

The settlement of each layer is given by:

\[ S_1 = 0.0241 \]

\[ S_2 = 0.020 \]

\[ S_3 = 0.017 \]

\[ S_4 = 0.013 \]

\[ S_5 = 0.012 \]

\[ S_{-} = 0.0861 = 1.03" \]

Total =
(a) Chart for use in determining vertical stresses below corners of loaded rectangular surface areas on elastic, isotropic material. Chart gives $f(m, n)$. (b) At point $A$, $\Delta \sigma = \Delta q_s \times f(m, n)$. (From Newmark, 1942)
If the induced stress are obtained by Newmark chart (see Fig. 5.7), is obtained

<table>
<thead>
<tr>
<th>Point</th>
<th>m</th>
<th>n</th>
<th>z</th>
<th>m/z</th>
<th>n/z</th>
<th>If</th>
<th>p</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.5</td>
<td>2.5</td>
<td>18</td>
<td>0.138</td>
<td>0.138</td>
<td>0.0095</td>
<td>304</td>
</tr>
<tr>
<td>2</td>
<td>2.5</td>
<td>2.5</td>
<td>20</td>
<td>0.125</td>
<td>0.125</td>
<td>0.007</td>
<td>224</td>
</tr>
<tr>
<td>3</td>
<td>2.5</td>
<td>2.5</td>
<td>22</td>
<td>0.113</td>
<td>0.113</td>
<td>0.005</td>
<td>160</td>
</tr>
<tr>
<td>4</td>
<td>2.5</td>
<td>2.5</td>
<td>24</td>
<td>0.104</td>
<td>0.104</td>
<td>0.0045</td>
<td>144</td>
</tr>
<tr>
<td>5</td>
<td>2.5</td>
<td>2.5</td>
<td>26</td>
<td>0.096</td>
<td>0.096</td>
<td>0.004</td>
<td>128</td>
</tr>
</tbody>
</table>

\[ \Delta P = \frac{200000}{5 \times 5} \times 4 \times \text{If} \]

The settlement obtained by using the Newmark's induced stresses is equal to \( \text{Stotal} = 0.060'' = 0.72'' \)

d. Structure Tolerance to Settlements of Building

Tolerable settlement can be defined as the value that a structure could undergo without affecting its function. The tolerable settlement may be different for different types of structures of different types of soils. For instance, some authors suggest a range from 1/4'' to 1'' depending on if the structures are statically indeterminate or statically
determinate. Values recommended by MacDonald and Skempton (1955) are shown in Table 5.2 and values recommended by the U.S.S.R. code are shown in Table 5.3.

Bjerrum (1963) proposed the following table that provides some guidelines to evaluate the tolerable settlement for a building (see Table 5.4).

Table 5.2 Tolerable differential settlements of buildings, in inches,* recommended maximum values in parentheses

<table>
<thead>
<tr>
<th>Criterion</th>
<th>Isolated foundations</th>
<th>Rafts</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angular distortion (cracking)</td>
<td>1.300</td>
<td></td>
</tr>
<tr>
<td>Greatest differential settlement:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clays</td>
<td>1 1/4 (1 1/2)</td>
<td></td>
</tr>
<tr>
<td>Sands</td>
<td>1 1/4(1)</td>
<td></td>
</tr>
<tr>
<td>Maximum settlement:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clays</td>
<td>3(2 1/2)</td>
<td>3-5(2 1/2-4)</td>
</tr>
<tr>
<td>Sands</td>
<td>2(1 1/2)</td>
<td>2-3(1 1/2-2 1/2)</td>
</tr>
</tbody>
</table>
Table 5.3 Permissive differential building slopes by
the U.S.S.R. code on both unfrozen and frozen ground

All values to be multiplied by \( L = \text{length between two}
adjacent points under consideration. } H = \text{height of}
wall above foundation. *

<table>
<thead>
<tr>
<th>Structure</th>
<th>On sand or hard clay</th>
<th>On plastic clay</th>
<th>Average max. settlement, cm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crane runway</td>
<td>0.003</td>
<td>0.003</td>
<td></td>
</tr>
<tr>
<td>Steel and concrete frames</td>
<td>0.0010</td>
<td>0.0013</td>
<td>10</td>
</tr>
<tr>
<td>End rows of brick-clad frame</td>
<td>0.007</td>
<td>0.001</td>
<td>15</td>
</tr>
<tr>
<td>Where strain does occur</td>
<td>0.005</td>
<td>0.005</td>
<td></td>
</tr>
<tr>
<td>Multistory brick wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>L/H to 3</td>
<td>0.005</td>
<td>0.004</td>
<td>8 ( L/H ) 2.5</td>
</tr>
<tr>
<td>Multistory brick wall</td>
<td></td>
<td></td>
<td>10 ( L/H ) 1.5</td>
</tr>
<tr>
<td>L/H over 3</td>
<td>0.003</td>
<td>0.007</td>
<td></td>
</tr>
<tr>
<td>One-story mill buildings</td>
<td>0.001</td>
<td>0.001</td>
<td></td>
</tr>
<tr>
<td>Smokestacks, water towers,</td>
<td>0.004</td>
<td>0.004</td>
<td>30</td>
</tr>
<tr>
<td>Ring foundations</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Structures on permafrost</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reinforced concrete</td>
<td>0.002-0.0015</td>
<td></td>
<td>15 at 4 cm/year</td>
</tr>
<tr>
<td>Masonary, precast concrete</td>
<td>0.003-0.002</td>
<td></td>
<td>20 at 6 cm/year</td>
</tr>
<tr>
<td>Steel frames</td>
<td>0.004-0.0025</td>
<td></td>
<td>25 at 8 cm/year</td>
</tr>
<tr>
<td>Timber</td>
<td>0.007-0.005</td>
<td></td>
<td>40 at 12 cm/year</td>
</tr>
</tbody>
</table>

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e. Elastic Settlement from Standard Penetration Test Data

In Fig. 5.8 is shown the method to obtain the elastic settlement in cohesionless soils when the relative density $D_r$ is known. It must be remembered that the value of $D_r$ can be obtained from standard penetration data as explained in Chapter IV.
Figure 5.8. Instantaneous settlement of isolated footings on coarse-grained soils.
CHAPTER VI

FACTORS TO BE CONSIDERED IN FOUNDATION DESIGN

a. Footing Depth and Spacing

There are three basic points to take into account to establish the depth at which a building foundation should be placed. These are:

1. The foundation must be placed on, or in, a bearing stratum of adequate capacity to support the expected loads without causing failure of the soil mass and without causing excessive settlement. That means, place the footings below topsoil or organic material, peat, muck and unconsolidated material such as garbage dumps, filled areas, etc.

2. The stratum underlying the bearing stratum also must be of adequate capacity to be able to resist the pressures imposed by the bearing stratum.

3. The footing should be placed in stable ground below the influences of erosion, frost heave, or volumetric changes due to moisture fluctuations.

In addition to the depth of the footing, it is also important to establish the spacing of the proposed footings as well as the relative elevations and spacing between the proposed footings and any nearby existing footings. Basically, when placing a new footing adjacent to an existing one, three cases can be observed:
a) The new footing is placed higher than the existing footing (see Fig. 6.1).

b) The new footing is placed lower than the existing footing (see Fig. 6.1).

c) Both the new footing and the existing footing are placed at the same level (see Fig. 6.2).

In the first case, it is a common practice to avoid interference between new and old footings, making the distance "m" greater than the distance "zf" so that conservative pressures can be transferred in the zone between footings and problems as shown in Fig. 6.3 avoided. Normally, a rule of thumb is followed to fix the distance m equal to two times the depth zf. That is, \( m = 2zf \) and the influence of adjacent footings neglected. The above assumptions are conservative but neglect the possibility of settlement of a deeper compressible stratum as shown in Fig. 6.4.

In the second case, the soil could flow laterally from beneath the existing footing causing settlement to the old structure. It is difficult to establish a safe depth zf. Generally, the problem is solved building a wall to retain the soil in rest conditions (see Fig. 6.5).

Similarly, in the third case, the loss of overburden pressure may cause settlements to the existing footing.

Finally, when the clear distance between adjacent
footings is greater than the least footing dimension, each footing may be analyzed independently because the pressure bulbs of adjacent footings will not overlap. For closely spaced footings, the pressure bulbs may overlap as shown in Fig. 6.6.

![Diagram of footings](image)

**Figure 6.1.** Location considerations for spread footings. (a) An approximation for spacing footings to avoid interference between old and new footings. If "new" footing is in relative position of "existing" footing, interchange words "existing" and "new". Make \( m > z \). (b) Possible settlement of "existing" footing because of loss of lateral support of soil wedge beneath existing footing.

![Diagram of settlement](image)

**Figure 6.2.** Potential settlement due to loss of overburden pressure.
Figure 6.3. Adjacent Footings

Figure 6.4. Influence of Construction of New Foundation on Performance of Existing Construction.
Figure 6.5. Common methods for bracing sides of shallow foundations.

Figure 6.6. Effect of overlapping pressure bulbs.
b. Steps on Choosing the Type of Foundation

Preliminary selection of foundation type is made without any formal design calculations based on three main considerations -- the type of structure, the subsurface conditions and the cost of the foundation as compared with the cost of the superstructure.

Peck suggested the following steps in order to determine the foundation type.

1. Obtain information about the nature of the superstructure and the loads to be transmitted to the foundation. For instance, for structures with a low tolerance to differential movement, rigid foundations should offer the solution. On the other hand, for warehouses whose superstructures are less rigid, more flexible foundations should be used. Heavy loads usually suggest mat or deep foundation.

2. Obtain information about the soil conditions. Here, some simple ideas could be kept in mind: If a thin layer of very soft clay is found over a firm layer or rock, the soft soil could be removed and a basement built or the soft soil could be replaced by a better material. If a thick layer of soft clay is found over firm clay, dense sand or rock, piles could be a solution. If stiff clay is over rock, shallow foundations could be good for light loads and mat or piles for heavy loads. For stiff clay over soft clay over rock,
shallow foundations resting on the stiff clay should be used for light and flexible structures. Piles resting on the rock should be used for heavy structures.

3. Consider briefly each of the customary types of foundations. Eliminate unsuitable types either because they cannot be constructed or because they could suffer intolerable settlements.

4. Make more detailed analysis of the types chosen above. Make more refined analysis of load-carrying capacity and settlement.

5. Estimate the cost of each of the possible type of foundation preselected and select the type which shows the best balance between performance and cost.
CHAPTER VII

SPREAD FOOTING DESIGN

a. Individual Columns and Wall Footing

A footing supporting a single column is called a "spread footing." A footing supporting a wall is called a "continuous footing" and serves a similar purpose as the spread footing, that is, transmitting the loads which come from the superstructure to the soil in a safe manner without causing distress and excessive settlement to it.

Spread footings can be uniform in thickness or variable in thickness as shown in Fig. 7.1, the variable thickness is used to save material. A pedestal may be used between metal columns and spread footings to prevent corrosion of metal when it is in contact with the soil.

The soil pressure distribution is assumed uniform beneath the footings if the load is concentric; it has been mentioned in Chapter V a more realistic distribution but the assumption of uniform pressure distribution is conservative and simplifies the analysis. Spread footing design is based on the work of Richart (1948) who located the critical sections for bending and the work of Moe (1961) who located the critical sections for shearing. That work has been summarized in the American Concrete Institute Code ACI which
has been periodically revised, the recent major revisions were made in 1963, 1971, 1977 and 1983.

![Footings Diagram]

Figure 7.1

Procedures for the design of spread footings were based first on elastic behavior of the materials (working-stress method or WSD) and later on plastic behavior (strength method or USD).

The critical sections for bending and shear are shown in Fig. 7.2.

The steps to be following in order to design a spread footing concentrically loaded are:

1. Obtain the allowable bearing capacity $q_a$, of the soil from the soil data and structural loads by means of the equations given in Chapter IV. Obtain by trial and error a
tentative value of the footing plan dimensions $B \times L$ or $B$.

![Critical Sections for wide-beam shear.](image)

![Critical Sections for punching shear.](image)

![Wall column or pedestal except masonry wall.](image)

![Critical Sections for Bending.](image)

Figure 7.2. Critical section for bending and shearing.

2. Check the settlement (elastic or consolidation) of the footing and readjust the dimensions to produce the tolerable settlement.

3. If desired, refine by computing an effective pressure $q_e$, to take into account the weight of the soil and the concrete and the soil removed when the footing is built.

$$q_{\text{effective}} = q_a - \rho D_f$$
where
\[
\sigma = \frac{\text{soil - concrete}}{2}
\]
The footing dimensions are readjusted to
\[
\frac{P_{\text{total}}}{q_{\text{effective}}} = A_{\text{required}}
\]

4. Convert the \(q_a\) value or the \(q_e\) value to an ultimate value \(q_u\)
\[
q_u = \frac{P_u}{B_L}
\]
where \(P_u = \text{factored load} = 1.4 P_d + 1.7 P_L\).

5. Obtain the thickness of the footing by checking for wide-beam shear and punching shear. Since it is required to solve a quadratic equation to obtain the thickness, the curves shown in Fig. 7.3 should be used.

6. Obtain the steel required for bending, computing the bending moments at the critical sections shown in Fig. 7.2.

7. Check column bearing and use dowels if needed. Check the development length \((L_d)\) of the steel.

8. Detail the design.

Table 7.1 shows a summary of the requirements of the ACI code for foundation design.

Example 7.1

Design a spread footing and pedestal for the following conditions:
<table>
<thead>
<tr>
<th>Design factor</th>
<th>ACI Code</th>
<th>General requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing of reinforcement</td>
<td>7.6</td>
<td>Not less than $D$ or 1 in or 1.33 $\times$ max. aggregate size; not more than 3 $\times$ depth of footing or 18 in</td>
</tr>
<tr>
<td>Lap splices</td>
<td>12.15.2</td>
<td>Not for bars $&gt; No. 11$</td>
</tr>
<tr>
<td>In tension</td>
<td>12.16</td>
<td>See section in Code</td>
</tr>
<tr>
<td>In compression</td>
<td>12.17</td>
<td>See section in Code</td>
</tr>
<tr>
<td>Temperature and shrinkage</td>
<td>7.12</td>
<td>$p = 0.002$ for $f_c = 275$ to 345 MPa $\Rightarrow$ 0.0018 for $f_c = 415$ MPa; welded-wire fabric</td>
</tr>
<tr>
<td>Minimum reinforcement cover</td>
<td>7.7</td>
<td>3 in against earth</td>
</tr>
<tr>
<td>Design-methods flexure</td>
<td>10.2</td>
<td>$M_d = 0.4, 0.6d - 0.21$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$v = A_d, 0.65f_y$</td>
</tr>
<tr>
<td>Maximum reinforcement</td>
<td>10.3.3</td>
<td>$p_d = 0.25 + Eq. 14.3-31a (in textbooks)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$p = A_d, b_d \leq p$</td>
</tr>
<tr>
<td>Minimum reinforcement</td>
<td>10.5.1</td>
<td>$p \geq 200f_y$, if footing of variable thickness; for slabs of uniform thickness use shrinkage and temperature percentage</td>
</tr>
<tr>
<td>$f_1$ factor</td>
<td>10.27</td>
<td>$f_1 = 0.85$ for $f_c \leq 28$ MPa</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$f_1 = 0.85 - 0.05$ for each 7 MPa over 28 MPa</td>
</tr>
<tr>
<td>Limits of compression reinforcement</td>
<td>10.9</td>
<td>$0.00 \leq A_{ps} - A_{ps} \leq 0.08$</td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td>8.5</td>
<td>$E = 3.0 \times 10^4, f_y = MPa</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$14.1 \leq E &lt; 24.4 \text{ kN/m}^2$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$4730, f_y = \text{MPa}</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$22 \leq E &lt; 23.6 \text{ kN/m}^2$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$= 57,000, f_y = \text{psi}</td>
</tr>
<tr>
<td>Load factors $\phi$</td>
<td>9.3.2</td>
<td>Flexure = 0.96; shear = 0.85; bearing = 0.70; ensure plate concrete = 0.65</td>
</tr>
<tr>
<td>Load</td>
<td>9.2</td>
<td>$1.4 \times$ dead load; 1.7 $\times$ live load</td>
</tr>
<tr>
<td>Bearing on concrete</td>
<td>10.16</td>
<td>$f_{cu} \leq 0.85f_y$, $v \leq 2$</td>
</tr>
<tr>
<td>Shear wide-beam</td>
<td>11.11</td>
<td>$r_s = V_n, b_d$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$r_s = 2b_d, \sqrt{f_y}$</td>
</tr>
<tr>
<td>Diagonal punching tension</td>
<td>11.11.2</td>
<td>$r_s = V_n, b_d$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$r_s = 2 + 4 b_d, \sqrt{f_y}$, $s \leq 4 b_d, \sqrt{f_y}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\beta = \frac{col. length}{col. width}$</td>
</tr>
<tr>
<td>Shear reinforcement in footings</td>
<td>11.11.3</td>
<td>See equations in text or Code</td>
</tr>
<tr>
<td>Development length of reinforcement</td>
<td>12.2.12.6</td>
<td>See equations in text or Code</td>
</tr>
<tr>
<td>Grade beams</td>
<td>14.3</td>
<td>General footing considerations</td>
</tr>
<tr>
<td>Footings</td>
<td>15.2</td>
<td>See Fig. 8-5</td>
</tr>
<tr>
<td>Location of bending moments</td>
<td>15.4.2</td>
<td>Percent in zone of width $= 2 \times \frac{L}{B + 1}$</td>
</tr>
<tr>
<td>Distribution of reinforcing</td>
<td>15.4.4</td>
<td>See Fig. 8-4</td>
</tr>
<tr>
<td>in rectangular footings</td>
<td>15.5</td>
<td>At least 4 dowels with total $A_d \geq 0.005A_p$</td>
</tr>
<tr>
<td>Shear location</td>
<td>15.8</td>
<td>$f_y = 0.85f_y$, $\phi = 0.70$</td>
</tr>
<tr>
<td>Transfer of stress at</td>
<td>15.11</td>
<td>$f_y = 0.85f_y$, $\phi = 0.65$</td>
</tr>
<tr>
<td>base of column</td>
<td>15.11.4</td>
<td>$w = \sqrt{A_d}$</td>
</tr>
<tr>
<td>Unreinforced pedestals and</td>
<td>15.3</td>
<td>Equivalent square column side.</td>
</tr>
<tr>
<td>footings</td>
<td>15.7</td>
<td>$8$ in unreinforced footing; $8$ in above reinforcement; $12$ in on piles</td>
</tr>
<tr>
<td>Round columns</td>
<td>15.3</td>
<td>$f_y = 0.85f_y$, $\phi = 0.65$</td>
</tr>
<tr>
<td>Minimum edge thickness</td>
<td>15.7</td>
<td>$8$ in unreinforced footing; $8$ in above reinforcement; $12$ in on piles</td>
</tr>
<tr>
<td>Maximum tensile stress in</td>
<td>14.2.7</td>
<td>$\phi = 0.65$</td>
</tr>
<tr>
<td>unreinforced footings</td>
<td></td>
<td>$8$ in unreinforced footing; $8$ in above reinforcement; $12$ in on piles</td>
</tr>
</tbody>
</table>

* Author recommends using 0.002 for all grades of steel.

Table 7.1 Summary of foundation-member requirements ACI Code.
Soil data:

\[ \gamma_d = 94 \text{ Plf} \]

\[ w_{nat} = 12\% \]

\[ \phi = 10^\circ \quad c = 4 \text{ psi} \quad c_c = 0.032 \]

Allowable settlement = 1"

A. Physical properties of the soil

\[ V_T = 1 \]

\[ V_w(62.4) \]

\[ V_s \]

\[ G_s = \frac{w_s}{w} \]

\[ w_s = \frac{v_v(62.4)}{0.5684} \]

\[ 0.12 = \frac{v_w}{0.5684(2.65)} \]

\[ v_s = \frac{w_s}{G_s w} \]

\[ w_s = v_s G_s w \]

\[ s = \frac{v_w}{v_v} = 0.1808 \times 100 \]

\[ s = 41.92 \quad v_v = 1 - v_s = 0.4316 \]

\[ e = \frac{v_v(62.4)}{v_s(0.5684)} \times 0.7 \]

Since this value is a little high, we must check the settlement.
\[ \gamma_{\text{tot}} = 94 \text{pcf} + 0.1808 (62.4) = 105.28 \text{pcf.} \]
\[ \gamma_{\text{sat}} = 94 + 0.4316 (62.4) = 120.93 \text{pcf.} \]
\[ \gamma_{\text{sub}} = \gamma_{\text{sat}} - \gamma_w = 120.93 - 62.4 = 58.53 \text{pcf.} \]

for
\[ \phi = 100 \quad N_c = 9.6 \quad N_q = 2.7 \quad N = 1.2 \quad \text{(from Table 4.1)} \]

B. Bearing Capacity

B.1 \[ Q_u = cN_c + \gamma D_f N_q + 0.5 B T N_y \]
\[ Q_u = 4(144) \quad 9.6 + 105.28 (3) \quad 2.7 + 0.5 B (105.28) \quad 1.2 \]
\[ Q_u = 6382.37 + 63.17 B \]
\[ q_a = \frac{Q_u}{3} = 2127.45 + 21.06 B \]

Assuming
\[ q_a = 2300 \text{ psf, for } P = L.L + D.L = 190 \text{ k.} \]
\[ B^2 = \frac{P}{q_a} = \frac{190000}{2300} \]
\[ B = 9.09' \]

Trying \( B = 9.25' \)
\[ q_a = 2127.45 + 21.06 (9.25) = 2322.25 \text{ psf} \]
\[ P_a = 2322.25 (9.25)^2 \quad 198694.52 \quad 190000 \quad \text{ok} \]

Trying \( B = 9' \)
\[ q_a = 2316.99 \]
\[ P_a = 2316.99 (9)^2 = 187576.19 \quad 190000 \]

\( \frac{1}{2} \) use 9.25' = B to support P = 190 Kips
B.2 Checking the settlement

\[ S_{\text{allow}} = 1'' \]

\[ S = \frac{H \cdot C_{\text{o}}}{1 + e_0} \log (1 + \frac{T}{\sigma_{\text{body}}}) \]

Obtained by:

\[ \sigma_1 = 2:1 \text{ method} \]

\[ \sigma_{\text{body}} = \gamma H \]

\[ H = 24'' \quad C_{\text{o}} = 0.032 \]

\[ e_0 = 0.7593 \]

\[ S_1 = 0.316'' \]

\[ S_2 = 0.203'' \]

\[ S_3 = 0.138'' \]

\[ S_4 = 0.099'' \]

\[ S_5 = 0.071'' \]

\[ S_6 = 0.052'' \]

\[ S_7 = 0.039'' \]

\[ S_8 = 0.030'' \]

\[ S_9 = 0.023'' \]

\[ S_t = 0.971 < 1'' \]

We don't know the thickness of the clay stratum but even if we check settlement up to the depth = 2B, assuming that we have clays up to this depth, the settlement is smaller than 1''.

A footing not smaller than 9.25' x 9.25' should be used.

And \( q_s = 2.32 \text{ ksf} \)
c. Design of the footing

\[ L.L = 90K \quad q_a = 2.32 \]

\[ D.L = 100K \]

\[ \frac{\gamma_s + \gamma_c}{2} = \gamma_o = 125 \text{ pcf} \]

\[ q_{\text{effective}} = q_a - \gamma_o D_f \]

\[ q_{\text{effective}} = 2.32 - (0.125)3 \]

\[ = 1.945 \text{ ksf} = q_e \]

\[ A_{\text{required}} + \frac{P}{q_e} + \frac{190K}{1.945 \text{ ksf}} = 97.69 \text{ sf}, B = 9.88' \]

Use \( B = 10' \)

c.1 Design for Bending

\[ P_u = 1.4(100) + 1.7(90) \]

\[ = 293 \text{ k} \]

\[ q_u = \frac{293 \text{ k}}{10 \times 10} = 2.93 \text{ ksf} \]

\[ M_{a-a} = \frac{2.93(10)(4.25)^2}{2} = 264.62 \text{ k-foot} \]

\[ q = \frac{P \text{ fy}}{f_{1c}} \quad \text{fy} = 60 \text{ ksi} \]
\[ \phi = 0.9 \quad f_{1c} = 3 \text{ ksi} \]
\[ f = 0.016 \]

(according to ACI code Art 10.3.3)

\[ q = \frac{0.016(60)}{3} \]
\[ q = 0.32 \]

\[ M_u = bd^2 f_{1c} q (1 - 0.59q) \]
\[ M_u = 0.9(10 \times 12 \text{ in}) 3 x (0.032) (1 - 0.59(0.32)) d^2 \]
\[ M_u = 8.41 d^2 \]

\[ M_u = M_r \]
\[ 8.41 d^2 = 264.62 \times 12 \text{ k-inch} \]
\[ d = 19.43'' \text{ (value required for bending)} \]

c.2 Design for simple shear

\[ q_u = 2.93 \text{ksf} \]
\[ V_{bb} = (4.25-d)10(2.93) = 124.525 - 29.3 d \]
\[ V_c = v_c bd \]
\[ v_c = 0.85(2) \sqrt{f'c} \]

Art 11.3.1.1 ACI
\[ V_c = 0.85(2) \sqrt{3000} \]
\[ V_C = 93.11 \text{ psi} = 13.4 \text{ ksf} \]
\[ V_C = 13.4(10)d = 134d \]
\[ V_{bb} = V_C \]
\[ 124.525 - 29.3d = 134d \]
\[ d = 0.762' = 9.15'' \text{ (required for simple shear)} \]

### c.3 Design for Diagonal Tension

\[ V_C = 4\phi \sqrt{f_{ic}} \]

\[ \text{Art 11.11.1 ACI} \]
\[ V_C = 4\phi \sqrt{f_{ic}} A, \]
\[ V_C = 4(0.85)(\sqrt{3000}) \]
\[ = 186.22 \text{ psi} = 26.82 \text{ ksf} \]
\[ V_C = 160.92d + 107.28d^2 \]

\[ V_u = 2.93 \left[ 10 \times 10 \right. \]
\[ - \left( 1.5 + d \right)^2 \]
\[ V_u = 2.93 \left( 100 - \left( 2.25 \right. \]
\[ + 3d + d^2 \right) \]
\[ V_u = 293 - 6.5925 - 8.79d \]
\[ - 2.93d^2 \]
\[ V_u = 286.4075 - 8.79d \]
\[ - 2.93d^2 \]
\[ V_u = V_C \]
\[160.92d + 107.28d^2 = 286.4075 - 8.79d - 293d^2\]
\[110.85d^2 + 169.71d - 286.4075 = 0\]

\[d^2 + 1.53d - 2.58 = 0\]

\[d = \frac{-1.53 \pm \sqrt{(1.53)^2 - 4(1)(-2.58)}}{2(1)}\]

\[d = 1.014'\]
\[d = 12.17''\]

We should use
\[d = 19.43'' \text{ (value required for bonding).}\]
\[h = 19.43'' + 3'' = 22.43'' \text{, use } h = 22.5''\]

As required
\[M_u = \phi As \frac{fy}{2} (d - \frac{a}{b}) a = \frac{Asfy}{0.85f'c} b\]

\[M_u = 0.9 \text{ As}(60)(19.43 - \frac{As(60)}{0.85(3)(10 \times 12)})\]

\[M_u = 54 \text{ As}(19.43 - 0.196 \text{ As})\]

\[M_u = 1049.22 \text{ As} - 10.59 \text{ As}^2 = 264.62 \times 12\]

\[-10.59 \text{ As}^2 + 1049.22 \text{ As} - 3175.44 = 0\]

\[\text{As}^2 - 99.08 \text{ As} + 299.85 = 0\]
\[ 99.08 \pm \sqrt{99.08^2 - 4(1)(299.85)} \]

\[ As = \frac{3.12}{2} \]

\[ f = \frac{As}{bd} = \frac{3.12}{10 \times 12 \times 19.43} = 0.0013 \]

\[ f_{min} = \frac{200}{60000} = 0.0033 \]

We have to use \( f_{min} = 0.0033 \); \( A_{min} = 0.0033 \) (120 x 19.43) = 7.77 in².

If we use bars #8, \( A_{#8} = 0.79 \) in²

\[ \# \text{bars} = \frac{7.77}{0.79} = 9.83 \approx 10 \text{ bars #8} \]

\[ \text{spacing} = \frac{120'' - (2 \times 3'')}{2} : 12.67'' \]

10 #8 each 12.5" C.C both ways

22.5"
Checking $L_d$

\[
L_{d,req} = \frac{0.04 \, A_{bfy}}{\sqrt{f'c}} = \frac{0.04(0.79) \, 60000}{\sqrt{3000}} = 34.62''
\]

or

\[
L_{d,req} = 0.0004 \, d_{bfy} = 0.0004(1)(60000) = 24''
\]

\[
L_{d,available} = 60'' - \frac{18'' - 3''}{2} = 48'' > 34.62'' \text{ ok}
\]

If the water table rises to the surface of the ground, the bearing capacity equation will take the form

\[
\text{effective} = \gamma_{df} = (\gamma_{sat} - \gamma_w)Df
\]

And, the value of $\gamma$ in the last term of the equation has to be replaced by $\gamma' = (\gamma_{sat} - \gamma_w)$

The preceding modifications are based on the assumption that there is not seepage forces in the soil, then

\[
Q_u = C\gamma_{c} + (\gamma_{sat} - \gamma_w)DfNQ + 0.58 (\gamma_{sat} - \gamma_w)N_T
\]

- We can notice that the bearing capacity is reduced by the effect of the water table at the surface of the ground.
- Also, we can notice that the water table above the base of the footing causes construction problems.

**Example 7.2**

**Spread Footing Design**

Given: Soil information: $\phi = 30^\circ$ $c = 0$ $D_f = 3$

\[
\gamma_{meas} = 120 \, \text{pcf}
\]

water table @ 50'
Structural data: 12" x 12" column with 4 # 8

\[ D_L = 20K \]
\[ L_L = 90K \]

\[ f_{1c} = 3000 \text{ psi} \]
\[ f_s = 60 \text{ grade} \]

**Bearing Capacity**

\[ Q_u = C N_{ct} + \gamma D f N_q + 0.5 \beta \gamma N_r \]

Bearing capacity factors (Terzaghi equations)

<table>
<thead>
<tr>
<th>( \phi ), deg</th>
<th>( N_c )</th>
<th>( N_q )</th>
<th>( N )</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>37.2</td>
<td>22.5</td>
<td>19.7</td>
</tr>
</tbody>
</table>
\[ Q_u = 3(120)(22.5) + 0.5(B)(120)(19.7) \]
\[ Q_u = 8100 + 1182B \]
\[ q_a = \frac{Q_u}{3} = 2700 + 394 B \]

Assuming \( q_a = 3000 \) psf

\[ A_{req} = \frac{P}{q_a} = \frac{(20 + 90) \text{ kips}}{3 \text{ ksf}} = 36.67 \text{ sf}, \quad B = 6.06' \]
\[ q_a = 2700 + 394(6.06) = 5087.79 \neq 3000 \text{ assumed} \]

Assuming \( 5090 \) psf

\[ A_{req} = \frac{110}{5.09} = 21.61 \text{ square feet}, \quad B = 4.65' \]
\[ q_a = 2700 + 394(4.65) = 4532.1 \neq 5090 \]

Assuming \( q_a = 4.5 \) ksf

\[ A_{req} = \frac{110}{4.5} = 24.44 \quad , \quad B = 4.94' \]
\[ q_a = 2700 + 394(4.94) = 4646.36 \text{ psf} = 4.65 \text{ ksf} \geq 4.5 \text{ assumed} \]

Assuming \( q_a = 4.6 \) ksf

\[ A_{req} = \frac{110}{4.6} = 23.91 \quad B = 4.89 \]
\[ q_a = 2700 + 394(4.89) = 4626.7 = 4.63 \text{ ksf} = 4.6 \]
\[ P_a = 4.62 \times 4.89 = 110.47 \geq 110 \text{ ok} \]

We can use \( B = 4.9' \) or values bigger than that.

Using \( B = 5' \), \( q_a = 4.67 \) ksf

\[
\begin{array}{ccc}
\text{B} & \text{q}_a & \text{P}_{\text{allowable}} \\
5 & 4.67 & 116.75 \geq 100 \text{ ok} \\
\end{array}
\]
<table>
<thead>
<tr>
<th></th>
<th>5.06</th>
<th>182.16</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>5.46</td>
<td>267.54</td>
</tr>
<tr>
<td>8</td>
<td>5.85</td>
<td>373.76</td>
</tr>
<tr>
<td>9</td>
<td>6.25</td>
<td>506.25</td>
</tr>
<tr>
<td>10</td>
<td>6.64</td>
<td>664.</td>
</tr>
</tbody>
</table>

If we need to check settlement, the curve shown above could give values of "qa" for any "B".

**Design of the footing**

\[
qa = 4.67
\]

\[
\gamma_0 = \frac{120 + 150}{2} = 135 \text{ pc}^f
\]
q_{\text{effective}} = q_a - f_{\text{c}} D_f
= 4.67 - 3(0.135)
= 4.265

\text{Area} = \frac{90 + 20}{4.265} = 25.8
= 5.08'

Use B = 5.25'

\text{Designing for Bending}

P_u = 1.4(20) + 1.7(90) = 181
\frac{P_u}{q_u} = \frac{181}{(5.25)^2} = \frac{181}{5.25^2} = 6.57 \text{ ksf}

M_{\text{aa}} = \frac{6.57(5.25)(2.125)^2}{2}
= 77.87 \text{ k-foot}

M_u = \phi b d^2 f'c (1 - 0.59 q)
q = \frac{f_y}{f'c}, \quad \phi = 0.9 \quad f_y = 60 \text{ ksi}
\quad f'c = 3 \text{ ksf}
\quad \phi = 0.016

(\text{Art 10.33 ACI Code})
\[ q = \frac{0.016(60)}{3} \]

\[ q = 0.32 \]

\[ M_u = 0.9(5.25 \times 12)d^2 (3)(0.32)(1 - 0.59(0.32)) \]

\[ M_u = 44.16 \text{ d}^2 \]

\[ M_u = M_r \]

\[ 44.16 \text{ d}^2 = 77.89 \times 12 \]

\[ dflexion = 4.6'' \]

**Design for Diagonal Tension**

Using Fig. 7.3 "Foundation Engineering" by Peck, Hanson, Thornburn

\[ a = \frac{1'' \text{ (column width)}}{5.25} = 0.19 \]

\[ qu = \frac{6570 \text{ Psf}}{186.22 \text{ Psi}} = 35.28 \]

- Using these values from Fig. 7.3 is obtained:

\[ d = 0.16 \]

\[ B \]

\[ d = 0.16 \times 5.25' \]

\[ d = 10.08'' \]

\[ \text{ArtACI 11.11.2 } V_c = 4(0.85)\sqrt{3000} = 186.22 \text{ psi diagonal} \]
Table 7.3. Curves for selecting depth of footing as determined by shear, provided by two-way behavior prevails.
We should assume $d = 11''$, $t = 11 + 3'' = 14''$

\[
\begin{align*}
V_u &= \frac{6.57 \text{ Ksf} \times 1000/144 \text{ Psi}}{(5.25' \times 12 - 23)} \quad \frac{(23)}{(11)} \\
V_u &= 155.09 \text{ psi} < V_c = 186.22 \text{ psi} \quad \text{ok}
\end{align*}
\]

\[d_{\text{required}} = 11''\]

\text{for diagonal tension}

\[
\text{Design for beam shear}
\]

\[
\begin{align*}
qu &= 6.57 \text{ Ksf} = 45.63 \text{ psi} \\
V_c &= 0.85(2) f_{1c} \\
\text{Art 11.3.1.1 ACI} \\
V_c &= 0.85(2) \sqrt{3000} = 93.11 \text{ psi} \\
V_u &= \frac{45.63 \text{ Psi} \ (14.5'' \times 63'')}{63''(d)} \\
&= \frac{661.64}{d} \\
V_c &= V_u \\
93.11 &= \frac{661.64}{d}, \ d = 7.11'' \quad \text{beam shear}
\end{align*}
\]
Having calculated three values of the "d", we can notice that the critical one is: \( d = 11'' \), therefore the footing should have the following dimensions:

![Footing Diagram](image)

**Steel Area Required**

\[
M_u = 77.89 \text{ K-foot} = 934.68 \text{ K-in}
\]

\[
M_u = \phi A_s f_y (d - \frac{a}{2}) \quad \text{where} \quad a = \frac{A_s f_y}{0.85 f_{cb}}
\]

\[
M_u = 0.9 A_s (60) \left[ 11 - \frac{A_s (60)}{2 (0.85) (3.53)} \right]
\]

\[
M_u = 594 A_s - 10.08 A_s^2
\]

\[
934.68 = 594 A_s - 10.08 A_s^2 \quad \text{or} \quad 10.08 A_s^2 - 594 A_s + 934.68 = 0
\]

\[
A_s = \frac{-(-594) + \sqrt{(-594)^2 - 4(10.08)(934.68)}}{2(10.08)}
\]

\[
A_s = \frac{+594 \pm 561.4}{2(10.08)}
\]

\[
A_s = 1.62 \text{ in}^2
\]
\[
\frac{f}{bd} = \frac{1.62}{63 \times 11} = 0.0023 \text{; use } f_{\text{min}} = \frac{200}{60000} = 0.0033
\]

\[A_{s_{\text{min}}} = 0.0033 (63 \times 11) = 2.31 \text{ in}^2\]

If we use #6, \[A_s \#6 = 0.44 \text{ in}^2\]

\[# \text{bars} = \frac{2.31}{0.44} = 5.25 = 6 \text{ bars} \#6\]

\[\text{Spacing} = \frac{63'' - 2(3'')}{5} = 11.4''\]

6 #6 each 11.4" c to c. both ways.

4 #7 dowels.
Checking the Development Length ($L_d$)

\[
L_d^{req} = \frac{0.04 A_d f_y}{\sqrt{f'_c}} = \frac{0.04 \times 0.44 \times 60000}{\sqrt{3000}} = 19.28''
\]

\[
L_d = 0.0004 \frac{d b f_y}{f'_c} = 0.0004 \times 0.75 \times 60000 = 18''
\]

\[
L_d = \frac{25.5'' - 3'' = 22.5''}{\text{available}} = \frac{19.28''}{\text{ok}}
\]

Checking the bearing strength

The bearing strength should be at least $0.85 f'_c$ with a limit of $\sqrt{A_2/A_1} < 2$.

\[
\sqrt{A_2/A_1} = \sqrt{\frac{63 \times 63}{12 \times 12}} = 5.25 \quad \text{use 2}
\]

\[
f_c = 0.85 \times 0.7 \times (3)(2) = 3.57 \text{ Ksi} \quad f'_c
\]

Actual contact $f_a = \frac{P_u}{A_{col}} = \frac{181}{144} = 1.26 \text{ Ksi} \quad 3.57 \text{ (or 3)}$

Transfer of Stress at Base of Column

At least four dowels with total $A_s > 0.005 A_d$

\[
A_s^{req} = 0.005 \times 144 = 0.72 \text{ in}^2 \quad \text{Use 4 # 7}
\]

Diameter difference $= 1.00 - 0.875 = 0.125 \quad 0.25 \text{ ok}$

\[
L_d = \frac{0.02 (f_y) (d_b)}{\sqrt{3000}} = \frac{0.02 \times 60000 \times 0.875}{\sqrt{3000}}
\]

\[
= 16'' \quad > \quad 11'' \quad \text{available}
\]
\[ L_{d_{reg}} = 0.003(f_y)(d_b) = 0.0003(60000)(0.875) \]
\[ = 13.1'' > 11'' \]

We should reduce \( L_d \) to \( L_{d_{id}} \)

\[ L_{d_{id}} = \frac{A_{req}}{A_{furnished}} = \frac{0.72}{2.40} \times 16'' = 4.8'' \]

Then the dowels can be bent and run them to the reinforcing steel layer.
b. Footings Subjected to Moment

In Chapter 4 is discussed the bearing capacity of eccentrically loaded footings. Knowing the load that acts on the footing and assuming the "effective" dimensions, a value of allowable bearing capacity can be obtained by using the Meyerhof expression given in Equation 4.10.

It must be noticed that the soil pressure is no uniformly distributed for these type of footings, therefore, the allowable soil pressure obtained has to be greater than the maximum value given by the strength of materials formula:

\[ q_{\text{max}} = \frac{P}{BL} \left(1 + \frac{6e}{L}\right) \]  

(7.1)

that is,

\[ q_a > q_{\text{max}} \]

Where the terms have the same meaning as defined before, it should be remembered that the case shown above is valid when the value of the eccentricity "e" is one way and it is located into the third middle of the footing that is

\[ \frac{-L}{6} < e < \frac{L}{6} \]  

(see Fig. 7.4a and 7.4b)

If the resultant R is located out of the third middle, the diagram of pressure distribution shows a zone in tension beneath the footing. It is known that the soil doesn't resist tensile forces, therefore, the portion of the footing subjected to tensile stress is assumed to be unable to carry
Fig 7.4 Pressure distribution under footings eccentrically loaded.

any load and the effective area of the footing is given by

$$A = B L'$$

where $L'$ is derived using principles of strength of materials and is equal to

$$L' = 3 (\frac{L}{2} - e)$$
letting "e" be greater than \( \frac{L}{6} \). The resultant \( R \) can be obtained as (see Figure 7.4c)

\[
R = \frac{q_{\text{max}}}{2} BL'
\]

Substituting \( L' \) into the expression for \( R \) is obtained.

\[
q_{\text{max}} = \frac{2R}{L} \leq q_{\text{allowable}}
\]

(7.2)

\[
\frac{L}{3B(-e)}
\]

When the eccentricity is in both ways, the soil pressure distribution can be obtained, if no footing separation occurs, as

\[
q_{\text{any point}} = \frac{p}{A} \frac{M_y}{I_y} + \frac{M_{xy}}{I_x}
\]

which is the equation (4.29) mentioned.

If tensile stresses are obtained, it is very complicated to obtain soil pressures distribution. Authors suggest to avoid designing these type of footings. There are approximate methods to calculate those pressures (see Deek 1973, pg. 391).

Once that the pressure distributions have been obtained and they are smaller than the allowable soil pressure, the footing design is carried out as it was indicated, recognizing that the soil pressure is no longer uniformly distributed and therefore the computation of the bending moments and shearing forces require more work.
CHAPTER VIII

SPECIAL FOUNDATIONS

When two columns are spaced so closely that spread footings are not practicable or when the property line is so close to a column that it is not possible to center a spread footing, it is necessary to use combined footings to transmit the load to the soil. Also, when settlements are a problem that cannot be solved using spread footings or heavy loads come the superstructure, mat foundation can be used.

a. Combined Footings

Combined footings may have rectangular or trapezoidal shape or may be linked by a strap beam as shown in Fig. 8.1. The dimension of these footings should be determined to obtain a uniform soil pressure distribution that is, the centroid of the soil reaction may coincide with the centroid of the resultant of loads in order to avoid excessive settlements or rotations in the footing.

Normally, the proportioning of the areas of the combined footing is solved by means of the laws of statics assuming that the allowable soil pressure is known. But, it can be noticed that in all the equations to determine the soil bearing capacity are included the footing dimensions and therefore the computation of soil bearing capacity becomes a
Figure 8.1. Combined footings: (a) Rectangular footing; (b) Trapezoidal footing; (c) Cantilever footing.

Figure 8.2. Rectangular footing
matter of trial and error. The point is, how can the allowable soil pressures be obtained independently from the footing dimensions since those are going to be obtained using an allowable soil pressure obtained previously. One possible way to obtain that bearing capacity is neglecting the last term of the equation 4.4 obtaining

\[ q_{allowable} = \frac{CN_c + \gamma D_f N_q}{F.S} \]  

(8.1)

which is a conservative value. Bowles suggests that when the width of the footing is smaller than 9' to 12' that term could be neglected with little error.

With the bearing capacity of the soil known, the dimensions for each of the footings mentioned can be obtained as follows:

1. Rectangular Combined Footing: (see Fig. 8.2)
   a. Obtain the area \( A \), as

   \[ A = \frac{P_1 + P_2}{q_{allow}} \]  

   (8.2)

   b. Determine the centroid of the column loads (\( x \))

   \[ x = \frac{P_2 (L_2)}{P_1 + P_2} \]  

   (8.3)

   c. Determine the length of the foundation (\( L \))

   \[ L = 2(L_1 + x) \]  

   (8.4)

where \( L_1 \) is the distance to the property line or to the outer face of the external column. Then
\[ L_3 = L - (L_1 + L_2) \]  
\[ (8.5) \]

d. Determine the width of the foundation \( B \).
\[ B = \frac{A}{L} \]  
\[ (8.6) \]

Since the design of the reinforcement requires the use of ultimate loads some authors suggest using ultimate loads when obtaining the centroid of the column loads.

The reinforced concrete design is carried out as usual for a simple supported beam:

\[ P_1(u) \quad P_2(u) \]

\[ q_u. \]

\[ L_1 \quad L_2 \quad L_3 \]

(a) Simple supported beam.

2. Trapezoidal Combined Footings (see Fig. 8.3)

\[ \text{Figure 8.3. Trapezoidal footing.} \]
a) Determine the area of the foundation as:

\[ A = \frac{P_1 + P_2}{h_{allow}} \quad (8.7) \]

and

\[ A = \frac{B_1 + B_2}{2} \quad \text{(from plane geometry) (1st equation)} \quad (8.8) \]

There are two unknowns, \( B_1 \) and \( B_2 \) then another equation must be based on the geometrical properties of the trapezoid.

b) From the property of the trapezoid:

\[ x + L_1 = \left( \frac{2B_1 + B_2}{B_1 + B_2} \right) \frac{L}{3} \quad \text{(2nd equation)} \]; \[ x = \frac{P_2 L_2}{P_1 + P_2} \quad (8.10) \]

Notice that the values of \( L, \ A, \) and \( L_1 \) can be known and solving the two equations shown above the values of \( B_1 \) and \( B_2 \) are obtained.

Notice that for a trapezoid

\[ \frac{L}{3} (x + L_1) \quad \frac{L}{2} \]

3. Cantilever Footing (see Fig. 8.4)

![Figure 8.4. Cantilever Footing.](image)
Figure 8.4. \(b\) Principle of cantilever footing.

a) Fox the dimension \(B_1\) and obtain \(e\) and \(L_1\). Notice that the values of "a" and "s" are normally known.

\[
e = \frac{B_1}{2} - a
\]  
\hspace{0.5cm} (8.11)

\[
L_1 = L - e
\]  
\hspace{0.5cm} (8.12)

b) Obtain \(R_1\) taking \(\Sigma M\) about \(R_2 = 0\), \(R_1 = \frac{P_1 L}{L_1}\)  
\hspace{0.5cm} (8.13)

Obtain \(R_2\) taking \(\Sigma M\) about \(R_1 = 0\) \(R_2 = \frac{P_2 L - P_1 e}{L}\)  
\hspace{0.5cm} (8.14)

check by \(F_v = 0\), \(R_1 + R_2 = P_1 + P_2\)

c) Obtain \(B_2\) and \(B_3\) as

\[
B_2 = \frac{R_1}{q_{allow.B_1}} \hspace{0.5cm} (8.15) \hspace{0.5cm} (B_3)^2 = \frac{R_2}{q_{all.}} \hspace{0.5cm} (8.16)
\]

d) Convert the \(q_{all}\) to \(q_{ult}\)

\(q_{ult} = q_{all}\). (UR)

where
\( \text{Ultimate total} \)
\( UR = \frac{P_{\text{working total}}}{P_{\text{total}}} \)

(e) Design the two footings as explained for spread footings and design the strap as a beam.

In order to avoid rotation of the exterior footing, Bowles suggests to design a rigid strap footing that is
\[ \frac{I_{\text{strap}}}{I_{\text{footing}}} > 2 \]
and check it to see if it is a deep beam.

Based on the assumption that the soil pressure is uniform, the following procedures should be used to obtain those dimensions for a rectangular footing more effectively.

**Rectangular Footings**

\[ Q_{\text{allow}} = \frac{C N_c + \gamma D f N_q + 1/2 B \gamma N_f}{F S} \]

where \( c, N_c, \gamma, D_f, N_q, \gamma N_f \), and \( F S \) are given.

Then, referring to Fig. 8.2,

\[ Q_{\text{all}} = \frac{P_1 + P_2}{B} \frac{c N_c + \gamma D f N_q + 1/2 B \gamma N_f}{B \times L} = \frac{1}{F S} \quad (8.17) \]

The length \( L \) is defined as before and \( H \) is known. Hence, solving the quadratic equation 8.2 the value of \( B \) is found.

b. **Mat Foundation**

As indicated, a mat foundation may be used to support heavy loads from columns, storage tanks or pieces of industrial equipment and the soil has low bearing capacity and therefore excessive settlements can be expected.
Types of Mat Foundations

In Fig. 8.5 is shown the different types of mat foundations.

Bearing Capacity of Mat Foundations

The Meyerhof's bearing capacity equation can be used to determine the ultimate bearing capacity.

\[ q_u = c N_c F_{cs} F_{cd} F_{ci} + \gamma D_f N_q F_{qs} F_{qd} F_{qi} + \frac{1}{2} \gamma B N \gamma F_{rs} F_{rd} F_{ri} \quad (8.18) \]

The allowable bearing capacity is given by

\[ q_{all} = \frac{q_u - \gamma D_f}{(\text{FS}) F_{s}} \quad \text{FS is usually 3} \]

The term B in the preceding equation is the smallest dimension of the mat.

Figure 8.5. Common types of mat foundations. (a) Flat plate; (b) plate thickened under columns; (c) beam-and-slab; (d) plate with pedestals; (e) basement walls as part of mat.
For mat foundations placed over sand, the bearing capacity may be obtained from the standard penetration resistance numbers.

\[
q_{all} = 11.98 \frac{N'}{3.28B} \left( \frac{2}{25.4} \right) \left( \frac{KN}{m^2} \right) \quad (8-19)
\]

where

\[
N' = \text{corrected standard penetration resistance}
\]

\[
B = \text{width (m)}
\]

\[
F_d = 1 + \frac{0.33}{B} \left( \frac{1.33}{15} \right)
\]

\[
S = \text{settlement in mm}
\]

\[
N' = 15 + 1/2 (N-15) \text{ if } N \geq 15 \text{ (Terzaghi and Peck)}
\]

**Differential Settlement of Mats**

The differential settlement may be obtained approximately as \(3/4\) of the total expected settlement if this is less than 2". Also, the American Concrete Institute suggested a method using the rigidity factor \((K_r)\) mentioned in Chapter V. That factor according to ACI can be obtained as

\[
K_r = \frac{E I_b}{E_s B^3} \quad (8.20)
\]

where

\[
E = \text{modulus of elasticity of the material used in the structure}
\]
\[ E_s = \text{modulus of elasticity of the soil} \]
\[ B = \text{width of foundation} \]
\[ I_b = \text{moment of inertia of the superstructure per unit of length at right angles to } B. \]

The term \( E I_b \) can be computed as

\[ E I_b = E(I_f + \sum I_{b'} + \sum \frac{a h^3}{12}) \]  \( (8.21) \)

where

\[ E I_b = \text{flexural rigidity of the superstructure and footing} \]
\[ E I_f = \text{flexural rigidity of the foundation per unit of length at right angles to } B \]
\[ E \sum I_{b'} = \text{flexural rigidity of the frame members} \]
\[ E \sum \frac{a h^3}{12} = \text{flexural rigidity of the shear walls perpendicular to } B. \]
\[ a = \text{shear wall thickness} \]
\[ h = \text{shear wall height} \]

and according to the ACI (see table below).

If calculations are made to obtain the settlement at the center and at the corner of the mat using the consolidation principles, the differential settlement of the mat may be computed but in most of the cases the differential settlement computed in that way will be higher than that obtained as
indicated above because the ACI method takes into account the stiffness of the superstructure.

<table>
<thead>
<tr>
<th>K_r</th>
<th>Differential settlement</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.35 x ΔH for long base, B ( = 0 ), L ( = L )</td>
</tr>
<tr>
<td></td>
<td>0.35 x ΔH for square mats, B ( = 1 ), L ( = L )</td>
</tr>
<tr>
<td>0.5</td>
<td>0.1 x ΔH</td>
</tr>
<tr>
<td>&gt; 0.5</td>
<td>Rigid mat, no differential settlement</td>
</tr>
</tbody>
</table>

ΔH = total settlement

**Design of Mat Foundations**

The structural design of mat foundations can be done by several methods depending on if the mat behaves as a rigid mat or a flexible mat. These methods are: the conventional rigid method, the approximate flexible method, the finite differences method (flexible) and the finite element method (flexible). In this report will be discussed the conventional rigid method only.

**Conventional Rigid Method**

The requirement to use the conventional rigid method can be listed as follows:
1. Distance between adjacent columns or loads between adjacent columns do not have to differ by about 20 percent.

2. The column spacing in a strip has to be less than $1.75/\beta$ and $\beta$ is equal to

$$\beta = \sqrt{\frac{BK}{4EI}}$$

(8.22)

where

$B =$ strip width

$K =$ modulus of subgrade reaction

$E =$ Young's modulus of the material of the mat

$I =$ moment of inertia of the cross-section of the strip

The structural design of a mat foundation can be carried out using the rigid method as follows (see Fig. 8.7):

1. Determine the total load on the mat, that is,

$$Q_T = \sum_{i=1}^{n} q_i$$

(8.24)

2. Determine the soil pressures under the point of interest by using the equation

$$q = \frac{Q}{A} + \frac{M_y x}{A I_y} + \frac{M_x y}{A I_x}$$

(8.24)

where

$A = B \times L$

$I_x = \frac{BL^3}{12} =$ moment of inertia about the $x$ axis
\[ I_y = \frac{1}{12} \]  = moment of inertia about the \( y \) axis  

\( x, y \) = coordinates of the point where the soil pressure is being found

\[ M_x = \text{moment of } Q_T \text{ about the } x \text{ axis} = Q_T \, e_y \]

\[ My = \text{moment of } Q_y \text{ about the } y \text{ axis} = Q_T \, e_x \]

\( e_x, e_y \) = load eccentricities that can be found as explained before

3. Compare the soil pressure values with the net allowable soil pressure. Remember \( q_{aw} \) must be \( \leq \) allowable net.

4. Divide the mat into several strips in \( x \) and \( y \) directions.

5. Draw the shear and the moment diagram for each individual strip. For simplicity, an average of soil pressure can be used to do so. That is, obtain the soil pressure under each corner of the strip and determine the average.

It is evident that the sum of the soil reaction given by \( q_{av} \) (strip area) and the column loads on the strip is not equal to zero because the shear between adjacent strip has not been considered. Therefore, the soil reaction and the column loads need to be adjusted as follows:

\[
\text{average load} = \frac{\text{soil reaction + column loads on strip}}{2}
\]

\( \text{(8.25)} \)
Figure 8.6. Critical sections for punching shear.

Figure 8.7. Conventional rigid mat foundation design.
\[ q_{av \ text{(adjusted)}} = \frac{q_{av \ text{average load}}}{\text{soil reaction}} \]  \hspace{1cm} (8.26)

\[ (\text{column load modification factor}) \quad F = \frac{q_{av \ text{average load}}}{\text{column loads on the strip}} \]  \hspace{1cm} (8.27)

So, each road on the strip has to be multiplied by \( F \)

\[ Q_{\text{modified}} = F \cdot Q_i \]

6. Determine the effective depth of the mat according to ACI shear requirements (see Fig. 8.6).

7. Determine the areas of steel required by flexion from the maximum positive and negative moments obtained in step 5 (see Fig. 8.8).

Figure 8.8. Typical shear and moment diagrams for a strip.
CHAPTER IX

LATERAL EARTH PRESSURE

a. Introduction

There are several types of civil engineering structures subjected to pressures due to the soil mass. Some types are shown in Fig. 9.1. The difference between these structures is in the way and the amount that they move. For instance, retaining walls rotate around their base, the braced walls of a cut move horizontally, the soil mass beneath a footing first moves downward and then is displaced laterally when the load is increased, basement walls are restrained at the top by the floor, etc. Therefore, it is necessary to know how to determine these earth pressures in order to design such structures safely and economically.

![Figure 9.1. Civil Engineering structures with different types of displacement caused by earth pressures. (a) Retaining wall; (b) bracing of cut; (c) footing; (d) basement wall.](image-url)
b. Active, Passive, at Rest Lateral Earth Pressure

According to the type of displacement between soil and construction element, the problem of lateral earth pressure may be divided into three types.

1. When the soil mass neither expands nor contracts. The pressure caused by soil is known as earth pressure at rest. It is the case of heavy structures at rest, basement walls when the backfill is placed after the floor slab is in place, bridge abutment fixed at the top of the bridge deck, tie-back retaining walls, etc.

2. When the soil mass either expands or contracts because of the displacement between soil and construction element. The pressure developed is "active" if the soil expands and "passive" if the soil contracts. This type includes retaining walls, sheetpiling and timbering of cuts (see Fig. 9.2).

3. This type is formed by those structures where vertical forces prevail. For example, foundations, buried structures and failure of soil beneath foundation structures.

Figure 9.2
In this chapter attention will be given to the different theories available to determine active, passive and at rest earth pressures.

As a way of illustration, the Mohr's circle can be used to understand the concepts mentioned above. Suppose a soil deposit with a horizontal surface and no shearing stresses on horizontal or vertical surface, as shown in Fig. 9.3. If an element of soil which originally was subjected to a vertical stress (\(\sigma_1\)) and to a horizontal stress (\(\sigma_2\)) is stretched in the horizontal direction, it will behave as a specimen in the triaxial machine where the confining stress is decreased and the axial stress is remained constant. The minimum stress reached before the soil fails is \(\sigma_3\) and is the active pressure mentioned. On the other hand, if the soil is compressed in the horizontal direction, it will behave as a soil specimen in the triaxial machine where the horizontal stress is increased and the vertical stress is kept constant. The maximum value that the horizontal stress can reach is \(\sigma_4\) if the Mohr's fail theory is used. This value is the passive earth pressure. These state of stresses are called the Rankine Active and Passive States.

Note in Fig. 9.3 that the circle 1-3 and 1-4 are failure circles which means that active and passive pressure are developed during soil displacement but until the soil is close to failure by shear. Both these circles represent a
Figure 9.3. Passive, Active, at Rest Pressure.

state of plastic equilibrium. The circle 1-2 represents a state of elastic equilibrium and it is not a state of failure. The inclination of the slip surface has been proved in the laboratory and it is similar to that shown in Fig. 9.3. From mechanics of materials concepts can be obtained the values of the horizontal stresses as:

$$\sigma_{03}^{(active)} = \sigma_{01} \tan^2 \left(45 - \frac{\phi}{2}\right) - 2c \tan \left(45 - \frac{\phi}{2}\right) \quad (9.1)$$

or

$$\sigma_{03}^{(active)} = \sigma_{01} K_a - 2c \sqrt{K_a}$$

$$\sigma_{04}^{(passive)} = \sigma_{01} \tan^2 \left(45 + \frac{\phi}{2}\right) + 2c \tan \left(45 + \frac{\phi}{2}\right) \quad (9.2)$$

$$\sigma_{04}^{(passive)} = \sigma_{01} K_p + 2c \sqrt{K_p}$$

where $K_a$ and $K_p$ are the active coefficient earth pressure and
passive coefficient earth pressure respectively.

Also,

\[ K_a = \frac{1 - \sin \phi}{1 + \sin \phi}, \quad K_p = \frac{1 + \sin \phi}{1 - \sin \phi} \]

\( \phi \) = angle of internal friction

\( C \) = cohesion

The coefficient for earth pressure at rest \( (K_o) \) can be approximated by

\[ K_o = 1 - \sin \phi \]  \hspace{1cm} (9.3)

for normally consolidated soils. For over-consolidated soils and compacted soils, \( K_o \) is normally larger than that given by equation (9.3). That can be seen in Table 9.1 which are the results of field observations.

Table 9.1  Coefficients of at-rest Earth Pressure

<table>
<thead>
<tr>
<th>Soil</th>
<th>( K )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compacted clay, hand tamped</td>
<td>1.0 to 2.0</td>
</tr>
<tr>
<td>Compacted clay, machine tamped</td>
<td>2.0 to 6.0</td>
</tr>
<tr>
<td>Clay, overconsolidated</td>
<td>1.0 to 4.0</td>
</tr>
<tr>
<td>Sand, loosely damp</td>
<td>0.5</td>
</tr>
<tr>
<td>Sand, compacted</td>
<td>1.0 to 1.5</td>
</tr>
</tbody>
</table>
Several theories have been developed to compute earth pressures against walls.

c. Coulomb's Earth Pressure Theory

The Coulomb's theory or the wedge theory take into account the following assumptions:

1. The soil is homogeneous and isotropic and has cohesion and internal friction.

2. The sliding wedge has a plane rupture surface and behaves a rigid body.

3. There is friction between the wall and the backfill material.

The principle of the Coulomb's theory is illustrated in Fig. 9.4

\[
\begin{align*}
W & = \text{weight of the soil wedge.} \\
R_a & = \text{active pressure} \\
R & = \text{soil reaction.}
\end{align*}
\]

Figure 9.4

Solving the triangle of forces with "w" known, it may be proved that the value of \(P_a\) depends on the inclination of the sliding wedge (\(\phi\)) then, applying the concept of the first
derivative in function of $P$ is equal to zero, that is

$$\frac{dP_a}{dP} = 0$$

the active earth pressure becomes

$$P_a = \frac{\gamma H^2}{2} \times \frac{\sin^2 (\alpha + \phi)}{\sin^2 \alpha \sin(\alpha - \phi) [1 + \sqrt{\frac{\sin(\phi + \delta) \sin(\phi - \beta)}{\sin(\alpha - \delta) \sin(\alpha + \beta)}}]^2}$$

or

$$P_a = \frac{\gamma H^2}{2} K_a$$

(9.5)

Note that the value of $K_a$ depends on $\alpha$, $\beta$, $\delta$ and $\phi$ which are:

$\alpha$ = backwall inclination

$\beta$ = surface inclination

$\delta$ = friction angle between soil and wall

$\phi$ = friction angle

Passive earth pressure can be obtained solving the triangle of forces shown in Fig. 9.5

\begin{figure}
\centering
\includegraphics[width=\textwidth]{coulombs_passive_pressure.png}
\caption{Coulomb's Passive Pressure.}
\end{figure}
If \( \frac{dp}{d\phi} = 0 \), \( P_p = \frac{gH^2}{2} \)

\[
\frac{\sin(\alpha - \phi)}{\sin^2(\alpha + \delta)[1 - \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \beta)}{\sin(\alpha + \delta)\sin(\alpha + \beta)}]}^2}
\]

(9.6)

or

\[
P_p = \frac{gH^2}{2} \quad \text{Kp}
\]

(9.7)

For a smooth vertical wall with horizontal backfill and right-angle wall that is, \( \delta = 0 \), \( \beta = 0 \), \( \alpha = 90^\circ \), the equations 9.4 and 9.6 become:

\[
P_a = \frac{gH^2}{2} \quad \tan^2 \left( 45 - \frac{\phi}{2} \right)
\]

\[
P_p = \frac{gH^2}{2} \quad \tan^2 \left( 45 + \frac{\phi}{2} \right)
\]

which coincide with the Rankine states shown in equations 9.1 and 9.2, if the cohesion of the material is zero. Values for active and passive coefficients based on equations 10.4 and 10.6 are shown in Table 9.1.

The wall friction angle may vary for different interface materials. In Table 9.2 are shown values that should be used as guides when no more information is available.

d. Rankine Earth Pressure Theory

Rankine made use of the state of plastic equilibrium
<table>
<thead>
<tr>
<th></th>
<th>ALPHA = 90°</th>
<th>BETA = 0°</th>
<th></th>
<th>ALPHA = 90°</th>
<th>BETA = 5°</th>
<th></th>
<th>ALPHA = 90°</th>
<th>BETA = 10°</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>α = 26°</td>
<td>β = 28°</td>
<td>30</td>
<td>α = 26°</td>
<td>β = 28°</td>
<td>30</td>
<td>α = 26°</td>
<td>β = 28°</td>
</tr>
<tr>
<td>0</td>
<td>2.494</td>
<td>3.203</td>
<td>3.492</td>
<td>2.915</td>
<td>4.177</td>
<td>4.585</td>
<td>5.046</td>
<td>5.572</td>
</tr>
</tbody>
</table>

Table 9.1. Active-earth-pressure coefficients $K_a$, based on Coulomb’s equation.

<table>
<thead>
<tr>
<th></th>
<th>ALPHA = 90°</th>
<th>BETA = 0°</th>
<th></th>
<th>ALPHA = 90°</th>
<th>BETA = 5°</th>
<th></th>
<th>ALPHA = 90°</th>
<th>BETA = 10°</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>α = 26°</td>
<td>β = 28°</td>
<td>30</td>
<td>α = 26°</td>
<td>β = 28°</td>
<td>30</td>
<td>α = 26°</td>
<td>β = 28°</td>
</tr>
<tr>
<td>0</td>
<td>3.385</td>
<td>3.713</td>
<td>4.080</td>
<td>4.496</td>
<td>4.968</td>
<td>5.507</td>
<td>6.125</td>
<td>6.841</td>
</tr>
</tbody>
</table>

Table 9.2. Passive-earth-pressure coefficients $K_p$, based on the Coulomb equation.

150
<table>
<thead>
<tr>
<th>Interface materials</th>
<th>Friction angle, ( \delta ), degrees‡</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mass concrete or masonry on the following:</td>
<td></td>
</tr>
<tr>
<td>Clean sound rock</td>
<td>35</td>
</tr>
<tr>
<td>Clean gravel, gravel-sand mixtures, coarse sand</td>
<td>29–31</td>
</tr>
<tr>
<td>Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel</td>
<td>24–29</td>
</tr>
<tr>
<td>Clean fine sand, silty or clayey fine to medium sand</td>
<td>19–24</td>
</tr>
<tr>
<td>Fine sandy silt, nonplastic silt</td>
<td>17–19</td>
</tr>
<tr>
<td>Very stiff and hard residual or preconsolidated clay</td>
<td>22–26</td>
</tr>
<tr>
<td>Medium stiff and stiff clay and silty clay</td>
<td>17–19</td>
</tr>
<tr>
<td>Steel sheet piles against:</td>
<td></td>
</tr>
<tr>
<td>Clean gravel, gravel-sand mixture, well-graded rock fill</td>
<td>22</td>
</tr>
<tr>
<td>with spalls</td>
<td></td>
</tr>
<tr>
<td>Clean sand, silty sand-gravel mixture, single-size hard-rock fill</td>
<td>17</td>
</tr>
<tr>
<td>Silty sand, gravel or sand mixed with silt or clay</td>
<td>14</td>
</tr>
<tr>
<td>Fine sandy silt, nonplastic silt</td>
<td>11</td>
</tr>
<tr>
<td>Formed concrete or concrete sheetpiling against:</td>
<td></td>
</tr>
<tr>
<td>Clean gravel, gravel-sand mixtures, well-graded rock fill</td>
<td>22–26</td>
</tr>
<tr>
<td>with spalls</td>
<td></td>
</tr>
<tr>
<td>Clean sand, silty sand-gravel mixture, single-size hard-rock fill</td>
<td>17–22</td>
</tr>
<tr>
<td>Silty sand, gravel or sand mixed with silt or clay</td>
<td>17</td>
</tr>
<tr>
<td>Fine sandy silt, nonplastic silt</td>
<td>14</td>
</tr>
<tr>
<td>Various structural materials:</td>
<td></td>
</tr>
<tr>
<td>Masonry on masonry, igneous and metamorphic rocks:</td>
<td></td>
</tr>
<tr>
<td>Dressed soft rock on dressed soft rock</td>
<td>35</td>
</tr>
<tr>
<td>Dressed hard rock on dressed soft rock</td>
<td>33</td>
</tr>
<tr>
<td>Dressed hard rock on dressed hard rock</td>
<td>29</td>
</tr>
<tr>
<td>Masonry on wood (cross grain)</td>
<td>26</td>
</tr>
<tr>
<td>Steel on steel at sheet-pile interlocks</td>
<td>17</td>
</tr>
<tr>
<td>Wood on soil</td>
<td>14–16§</td>
</tr>
</tbody>
</table>

† Based in part on NAFAC (1971).
‡ Single values ±2. Alternate for concrete on soil is \( \delta = \phi \).
§ May be higher in dense sand or if sand penetrates wood.

Table 9.2. Friction angles -- between various foundation materials and soil or rock.
mentioned above and neglected the wall friction. The principle of the Rankine theory is illustrated in Fig. 3.6.

![Diagram of Rankine's Principle for Active and Passive Earth Pressures]

Figure 3.6. Rankine's Principle for Active and Passive Earth Pressures.

Again, the value of $P_a$ or $P_p$ is a function of the inclination of the sliding wedge ($\phi$). Therefore setting

$$\frac{dP_a}{d\phi} = 0 \quad \text{and} \quad \frac{dP_p}{d\phi} = 0,$$

is obtained.

$$P_a = \frac{\gamma H^2}{2} \cos \beta \left( \frac{\cos \beta}{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \phi}} \right)$$

or

$$P_a = \frac{\gamma H^2}{2} K_a R$$  

(9.8)

$$P_p = \frac{\gamma H^2}{2} \cos \beta \left( \frac{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \phi}}{\cos \beta - \sqrt{\cos^2 \beta - \cos^2 \phi}} \right)$$

or

(9.9)
\[
\sigma_{H2} = \frac{Pp}{2} = \frac{KpR}{\rho}
\]

Values \(KAR\) and \(KPR\) are given in Table 9.3. Again, if \(C = 0\),

Table 9.3. Active-earth-pressure coefficients \(K\) for the Rankine equation, values not given for \(\beta > \phi\)

<table>
<thead>
<tr>
<th>(\beta)</th>
<th>0.0</th>
<th>0.1</th>
<th>0.2</th>
<th>0.3</th>
<th>0.4</th>
<th>0.5</th>
<th>0.6</th>
<th>0.7</th>
<th>0.8</th>
<th>0.9</th>
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<tbody>
<tr>
<td>0.0</td>
<td>1.0000</td>
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<td>1.0000</td>
<td>1.0000</td>
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<td>0.1</td>
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<td>1.1064</td>
<td>1.1064</td>
<td>1.1064</td>
<td>1.1064</td>
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<td>1.1064</td>
<td>1.1064</td>
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<td>1.2117</td>
<td>1.2117</td>
<td>1.2117</td>
<td>1.2117</td>
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</tr>
</tbody>
</table>

Table 9.3. Passive-earth-pressure coefficients \(Kp\) for the Rankine equation

<table>
<thead>
<tr>
<th>(\beta)</th>
<th>0.0</th>
<th>0.1</th>
<th>0.2</th>
<th>0.3</th>
<th>0.4</th>
<th>0.5</th>
<th>0.6</th>
<th>0.7</th>
<th>0.8</th>
<th>0.9</th>
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<tbody>
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</table>

that is, the ground surface is level, the equations 9.8 and 9.10 simplify to equation 9.1 and 9.2 if cohesion is 0.

In general, it can be said that the soil pressure is hydrostatic, that is, increases with depth linearly, as shown in Fig. 9.7

If water is present, its effect can be calculated knowing that \(Kp = Ka = 1\) for water and its distribution is linear with depth. Then, water pressure is another force to
Figure 9.7a. Pressure Diagrams for Active Rankine Earth Pressure

Figure 9.7b. Pressure Diagrams for Active Coulomb's Earth Pressure.
be added to the earth pressure. Also, the weight of the submerged soil is changed by the effect of water.

Neither the Coulomb theory nor the Rankine theory take into account the cohesion value (c) of the soil directly. The coefficients given include the friction angle $\phi$ and the weight of the backfill $\gamma$ as soil parameter. Although most of the backfill for retaining structures are built using granular materials, attention should be given to those backfills which show cohesion. That can be done using the equations 9.1 and 9.2 if the backfill slope is horizontal and, either Coulomb or Rankine coefficient can be used to obtain the earth pressure.

$$P_a = \frac{\gamma H^2}{2} \cdot K_a - 2CH \sqrt{K_a} \quad (9.12)$$

$$P_P = \frac{\gamma H^2}{2} \cdot K_p + 2CH \sqrt{K_p} \quad (9.13)$$

Note in equation 9.12 that when $H = 0$ the active pressure becomes negative which means that the soil is subjected to tensile stress provoking tensile cracks in the soil and modifying the original lateral pressure distribution diagram as shown in Fig. 9.8. Therefore, an analysis should be made to find out what is the active force before and after the tensile cracks occur.

When the backfill is not horizontal and shows cohesion some authors suggest to use the method called "the trial wedge" method which is a graphical one (see Bowles, page 410).
The theory of elasticity can be used to determine the pressure caused by surcharges. Those surcharges may be a point load caused by a column, a line load caused by a concrete-block wall or a pipeline and, strip load caused by highway or railroad which is parallel to the retaining structure. In Figure 9.10 is shown the formulas used to obtain earth pressures due to surcharges based on elasticity concepts. It can be noticed that the diagrams of pressure distribution are not linear and therefore is not easy to compute the resultant force of that diagram and its point of application. Something that can be done is to divide the total height $H$ of the wall into several segments and obtain the horizontal stress for each segment then assume linear distribution between those points so that the area of the pressure diagram can be obtained (see Fig. 9.11).
Also, Newmark presented an influence chart to obtain lateral pressure due to surcharges that consider Poisson's ratio value of 0.5 too. The horizontal stress is given by

\[ \sigma_H = IMq \]  (9.14)

where

\( \sigma_H \) = horizontal stress to the desired depth
\( M \) = number of squares enclosed in the load area
\( q \) = pressure intensity
\( I \) = influence coefficient given by the chart.

The Newmark chart is shown in Fig. 9.9.

e. Earth Pressure Theories in Retaining Walls

The Rankine and Coulomb theories can be used to evaluate active or passive pressure against retaining walls. To use these theories in design, several assumptions must be made. For instance, the Rankine theory needs to fulfill the following requirements:

1. The backfill must be a plane surface.
2. The wall must not obstruct the sliding wedge.
3. The angle \( \delta = 0 \) (no wall friction).

To meet these requirements, the Rankine pressure needed to be calculated as shown in Fig. 9.12.

It can be said that the Rankine theory should be used to determine earth pressures in cantilever walls as well as in gravity walls. The Coulomb theory should be used for gravity
Figure 9.9. Influence chart for computing lateral pressure at point $0$ for any type of loading in the influence field ($\mu = 0.5$). (Newark, 1942).
walls and for cantilever walls when the later are over 7 meters in height. Notice that when the Rankine theory is used, the weight of the soil above the heel (W_s) beside the weight of the concrete (W_c) must be taken into account for stability analysis. Unlikely, if the Coulomb theory is used, the weight of the soil is not used.
Fig 9.10. (a) Lateral pressure against rigid wall due to a point load and \( K = 0.5 \).

(b) Lateral pressure at points along the wall on each side of a perpendicular from the concentrated load.

(d) Lateral pressure against rigid wall due to a strip load. Note that at a wall depth, this equation can compute (-) pressure for certain \( \beta \) and \( \alpha \) combinations.

Figure 9.11. Assumed linear pressure distribution.
Figure 3.12. Requirements to use the Rankine Theory and Coulomb's Theory: (a) Cantilever wall, Rankine analysis; (b) Gravity wall, Rankine analysis; (c) Gravity wall, Coulomb analysis.
a. Proportioning Retaining Walls

Tentative dimensions have to be selected as a preliminary step when designing retaining walls. In Fig. 10.1 is shown tentative values which are based on experience obtained from walls which have had good performance in the past.

![Diagram](image)

Figure 10.1. Approximate dimensions for (a) Gravity wall (b) Cantilever wall.
b. Retaining Wall Design

The following steps are suggested to design retaining walls:

1. Obtain soil properties $\phi$, $c$ and $\gamma$ for both backfill and base soil. Cohesionless backfill are recommended in the Rankine zone (see Fig. 10.2).

![Figure 10.2. Backfill Rankine Zone.](image)

2. Select tentative dimensions based on Fig. 10.1.

3. Determine earth pressures by means of the Rankine or Coulomb theory taking in account the requirements mentioned in Fig. 9.12.

4. Determine all the forces acting on the retaining wall, as shown in Fig. 10.3.

5. Check the stability of the retaining wall, that is,

   5.1. Check for overturning about its toe that is about point A in Fig. 10.3.
Figure 10.3. Forces on gravity wall (a) Coulomb analysis; (b) Rankine analysis.

Figure 10.3a. Forces on cantilever wall. (a) Entire unit; free bodies for; (b) stem; (c) toe; (d) heel. Note that \( M_1 + M_2 + M_3 = 0.0 \).

The safety factor against overturning about the toe is given as
\[ FS = \frac{\sum M_R}{\sum M_0} \text{ standard practice requires } 1.5 < FS < 2 \text{ overturning} \]

where

\[ \sum M_0 = \text{sum of the moments of forces tending to overturn about point A (P_horizontal).} \]

\[ \sum M_R = \text{sum of the moments of forces tending to resist overturning about point A (Wc, Ws, P_vertical).} \]

5.2. Check for sliding failure along its base.

The safety factor against sliding is given as

\[ FS_{\text{sliding}} = \frac{\sum F_R'}{\sum F_d} \text{ standard practice requires } \]

\[ FS_{\text{sliding}} = 2.5 \]

where

\[ \sum F_R' = \text{sum of the horizontal resisting forces} \]

\[ \sum F_d = \text{sum of the horizontal driving forces} \]

or

\[ FS_{\text{sliding}} = \frac{(\sum v) \tan \phi + Bc}{P_{\text{horizontal}}} \]

where \( \phi \) and \( c \) are soil parameters in the base of the retaining wall and \( B \) is the width of the base.

5.3. Check for bearing capacity failure of the base. Similarly, the principles that were explained before in footing subjected to eccentric load are used to check the bearing capacity in the base of the wall

\[ q_{\text{base}} = \frac{V}{A} + \frac{(\sum M) x}{A - I_z} < q_{\text{allowable}} \]
where $q_{allowable}$ may be obtained as indicated for continuous footings. The safety factor against bearing capacity is given as

$$FS = \frac{q_u}{q_a} \text{ usually } \frac{q_u}{q_a} \approx 3$$

(bearing capacity)

5.4. Check for settlement. If the wall rests on a granular soil, the concepts on elastic settlement indicated in Chapter V can be used. If the wall rests on a cohesive soil, the consolidation principles given in Chapter II may be used to predict the settlement. It is a good practice to keep the resultant force near the middle of the base to reduce differential settlements. As a continuous footing the base of the wall may have problems of differential settlements in the longitudinal direction if the soil is very variable in its characteristics.

5.5 Check for overall stability. In addition to the stability checks mentioned above, the retaining wall should be checked against two other types of failures shown in Fig. 10.4. Some authors recommend to make the analysis for deep shear failure when the weak stratum is within the depth of 1.5 to 2H of the base of the wall. The stability analysis can be made by means of the Swedish circle method that can be found in any Soils Mechanics book. The safety factor obtained should be at least 2.0 for this analysis.
5.6. Reinforce Concrete Design.

For cantilever walls, once the steps given above have been carried out, the stem, the heel and the toe may be designed according to the ACI Code and using the free bodies shown in Fig. 10.3.

Figure 10.4. (a) Shallow shear failure; (b) deep shear failure.
c. Drainage, Wall Joints

It is a good practice to provide soil drainage instead of designing retaining walls against larger pressure caused by rainfall or other wet conditions. Adequate drainage should be provided using weepholes and/or perforated pipes as shown in Fig. 10.5.

In addition to that, joints may be constructed in a retaining wall to: (see Fig. 10.5)

1. Joint two successive pours of concrete vertically or horizontally. They are called "construction joints."

2. Control crack formation due to the shrinkage of the concrete. (They are vertical joints and are called "contraction joints" and are spaced from 8 to 12 meters).

3. Control expansion of concrete due to temperature changes. (They are built vertically and are called "expansion joints" and are spaced from 18 to 30 mt.)

![Diagram of Drainage of retaining walls]

Figure 10.5. Drainage of retaining walls.
Figure 10.6. (a) Construction joints; (b) contraction joint; (c) expansion.

Example 10.1

Design a gravity retaining wall for the conditions given below.

[Diagram showing a gravity retaining wall with a water tank on top, labeled with forces and dimensions.]
Soil Data

<table>
<thead>
<tr>
<th>Backfill</th>
<th>Base soil</th>
<th>$\gamma_c = 130$ pcf</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma$</td>
<td>110 pcf</td>
<td>100 pcf</td>
</tr>
<tr>
<td>$\phi$</td>
<td>30°</td>
<td>15°</td>
</tr>
<tr>
<td>$c$</td>
<td>$c = 0$</td>
<td>$c = 500$ psf</td>
</tr>
</tbody>
</table>

Tentative dimensions (use Fig. 10.1)

Earth Pressure (use Rankine theory)

$$P_a = \frac{\gamma H^2}{2} \quad Ka = \frac{110 \times (12)^2}{2} \quad 0.3333 = 2639.74 \text{ #'s say 2640 #'s.}$$

$$Ka = 0.3333 \quad \text{For } \phi = 30\degree \quad \beta = 0 \quad \text{From Table 3.3}$$

$$P_a = 2640 \text{ lbs/ft}$$
Forces in Retaining Wall (see Fig. 10.)

<table>
<thead>
<tr>
<th>Part</th>
<th>Weight (pounds)</th>
<th>Arm to Point A (feet)</th>
<th>( M(+) ) (pound-feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1)</td>
<td>( 0.5 \times 10.5 ) ( \frac{\text{---}}{2} ) = 342</td>
<td>1.83</td>
<td>+ 625.9</td>
</tr>
<tr>
<td>(2)</td>
<td>130 ( 1 \times 10.5 ) = 1365</td>
<td>2.5</td>
<td>+ 3412.5</td>
</tr>
<tr>
<td>(3)</td>
<td>110 ( 4 \times 10.5 ) = 4620</td>
<td>5</td>
<td>+ 23100</td>
</tr>
<tr>
<td>(4)</td>
<td>130 ( 1.5 \times 7 ) = 1365</td>
<td>3.5</td>
<td>+ 4777.5</td>
</tr>
</tbody>
</table>

\[ \Sigma V = 7692 \quad \Sigma M_R = 31915.9 \]

\[ P_a = 2640 \quad 4 \quad -10560 \]

\[ = \Sigma M_{C_1} \]

Pressure due to surcharge (use Newmark Chart, Fig. 9.9)

<table>
<thead>
<tr>
<th>Elevation</th>
<th>Scale, feet</th>
<th>Relative dimensions in terms of ( AB )</th>
</tr>
</thead>
<tbody>
<tr>
<td>- 3'</td>
<td>AB = 3</td>
<td>Side B = ( \frac{10}{3} ) = 3.33AB, ( \phi A = \frac{15}{3} = 5AB )</td>
</tr>
<tr>
<td>- 6'</td>
<td>AB = 7</td>
<td>B = 1.67 AB, ( \phi A = 2.54B )</td>
</tr>
<tr>
<td>- 9'</td>
<td>AB = 9</td>
<td>B = 1.1 AB, ( \phi A = 1.67AB )</td>
</tr>
<tr>
<td>- 12'</td>
<td>AB = 12</td>
<td>B = 0.83AB, ( \phi A = 1.25AB )</td>
</tr>
</tbody>
</table>

\[ \theta = 2 \times 0.001 \times 15 \times 1500 \]

\[ P = 2IMQ \]

(for rigid walls)
Assuming that the resultant forces except the 18#s force act in the middle of each pressure diagram, the total resultant is located at:

\[ \bar{y} = \frac{18(10) + 144(7.5) + 270(4.5) + 279(1.5)}{18 + 144 + 270 + 279} = \frac{8693.3}{711} = 4.70 \]

Psurcharge 711 arm 4.07 Moment - 2893.8 = \[ \Sigma M_{T} \]

\[ \Sigma M_{T} = -10560 - 2893.8 = 13453.8 \]

**Stability Check**

\[ FS_{overturning} = \frac{\Sigma M_{R}}{\Sigma M_{O}} = \frac{31915.9}{13453.8} = 2.37 \] ok

\[ FS_{sliding} = \frac{\Sigma F_{R} \tan 150^\circ + (7 \times 500)}{\Sigma F_{d}} = \frac{5581.27}{3351} = \frac{5581.27}{2640 + 711} = 3.35 \]

= 1.66 > 1.5 ok

**Bearing Capacity Check**

\[ q_{toe} = \frac{\Sigma V}{B} (1 + \frac{6e}{B}) \]

\[ q_{toe} = \frac{7692}{7} (1 + \frac{6(1.13)}{7}) = 2134.92 \text{ psf} \]

\[ q_{heel} = \frac{\Sigma V}{B} (1 - \frac{6e}{B}) \]
\[
q_{\text{heel}} = \frac{7692}{7} \times \left(1 - \frac{6(1.10)}{7}\right) = 62.8 \text{ psf}
\]

\[
\phi = \tan^{-1} \frac{3351}{7692} = 23.54^\circ
\]

\[
e = \frac{7/2}{7692} (31915.9 - 13453.8) = 1.10' \text{ (B/6 = 1.17) (only compression under base)}
\]

\[
e = B/2 - \frac{\sum M_R - \sum M_0}{\sum V}
\]

\[
qu = cN_c F_{cs} F_{cd} F_{ci} + N_q \gamma D_f F_{qs} F_{cd} F_{q_i}
\]

\[
\phi = 15^\circ \quad N_c = 11
\]

\[
N_q = 3.9 \text{ (from Table 4.2)}
\]

\[
N_\gamma = 2.2
\]

\[
F_{cs} = F_{qs} = F_s = \text{ (for continuous footings)}
\]

\[
B' = B - 2e = 7 - 2(1.1) = 4.8 \text{ (eccentrically loaded footings)}
\]

\[
D_f = 1 + \frac{0.4}{B} \text{ for } D_f/B < 1
\]

Assuming \(D_f = 3'\)

\[
D_f = \frac{3}{7} = 0.43 < 1
\]

\[
F_{cd} = 1 + 0.4(3/7) = 1.17
\]
\[ F_{qd} = 1 + 2 \tan \phi \left( 1 - \sin \phi \right)^2 \frac{D_f}{B} \frac{D_f}{B} < 1 \]

\[ F_{qd} = 1 + 2 \tan 15^\circ \left( 1 - \sin 15^\circ \right)^2 \frac{3}{7} = 1.29 \]

\[ F_d = 1 \]

\[ F_{ci} = F_{qi} = \left( 1 - \frac{B_0}{300} \right)^2 \]

\[ F_{ci} = F_{qi} = \left( 2 - \frac{23.54^\circ}{900} \right)^2 = 0.55 \]

\[ F_{gi} = \left( 1 - \frac{B_0}{\phi} \right)^2 = \left( 1 - \frac{23.54^\circ}{15^\circ} \right)^2 = 0.32 \]

\[ q_u = 500(11)(1.17)(0.55) + 100(3)(1.29)(0.55)(3.3) \]

\[ + \frac{1}{2} (4.8)(100)(1.1)(0.32) \]

\[ q_u = 4453.85 \text{ psf} \]

\[ q_a = \frac{4453.85}{3} = 1484.6 < 2134.92 \]

Being \( q_{all} < q_{toe} \), a new section must be tried!
CHAPTER XI

a. Introduction

Piles are structural members made out of steel, timber, and/or steel, used to transfer loads to lower strataums in the soil, for the following purposes, among others:

1. To transfer loads through water or soft soil to a suitable bearing stratum.

2. To transfer loads to a depth of relatively weak soil by means of "skin friction".

3. To control settlements when spread footings are on highly compressible stratum.

4. To support large horizontal or inclined forces.

5. To compact granular soils.

6. To carry the foundation through the depth of scour to avoid the loss of bearing capacity that a shallow foundation may suffer due to the erosion of the soil.

7. To stiffen the soil beneath machine foundations.

8. To anchor down structures subjected to uplift forces.

b. Types of Piles

Piles may be classified according to their composition or function.
According to their composition:
- Timber: plain or treated with preservative.
- Concrete: precast or cast in place.
- Composite: concrete with steel or wood.

According to their function, they can be classified as:
point-bearing capacity piles, friction piles and compaction piles.

c. Determination of Type and Length of Piles

Determining the type and length of piles to be used in a project is difficult and good judgement is required. The following ideas should be kept in mind as a guide. Point-bearing piles may be used if bedrock is within a reasonable depth, then from borings records the length of the piles can be fairly established. On the other hand, friction piles may be used if bedrock is not within a reasonable depth and the use of point-bearing piles are not economically feasible. The length of friction piles is difficult to predict and it depends on the characteristics of the soil, the applied load, and the pile size. Finally, the compaction piles are used to increase the density of granular soils close to the ground surface. Their length depends on the stage of density of the soil before and after compaction and the required depth of compaction, test driving is necessary to figure out the adequate length. In general, they are generally short.
d. Pile Capacity

When a pile foundation is designed, it is necessary to obtain its length and its cross section, the pile selected must be able to support the loads from the superstructure without causing excessive settlements. The bearing capacity of a single pile is controlled by the structural strength of the pile based on allowable stress (code value) and the supporting stress of the soil based on the in situ soil properties (bearing capacity equations). The smaller of the two values is used for the design. In addition, pile driving formulas, pile-load test, or a combination are used on the field to compute the pile capacity and check if the pile has been correctly designed and placed, in other words, check if the pile has reached the designed capacity at the depth predetermined.

1. Pile capacity - structural strength

Being the piles structural members embedded in soil which give lateral support, the slenderness effect can be neglected to obtain the structural capacity of a pile subjected to axial load. Therefore, the structural strength is given in the general form

\[ P_{\text{all}} = A_{\text{fall}} \]  \hspace{1cm} (11.1)

where

\[ P_{\text{all}} = \text{allowable design load} \]

\[ A = \text{cross-sectional area of pile at cap} \]
fall = allowable stress of the material being used (code value)

Below are presented formulas to obtain the allowable design load based on the structural strength.

\[
\begin{align*}
\text{Timber piles} & \quad P_{al} = A_p \cdot f_a \\
& \text{Ap: pile cross-sectional area} \\
& \text{fa: allowable stress for type of timber used} \\
\text{Precase concrete} & \quad P_{al} = A_g \cdot (0.33f'c - 0.27fe) \\
& \text{Ag: gross concrete area} \\
& \text{f'c: concrete compressive strength (35-55 MPa)} \\
& \text{fe: effective pretress after losses (5 MPa)} \\
\text{Cast in place piles} & \quad P_{al} = A_c \cdot f_c + A_s f_s \\
& \text{Ac, Ac = area of, respectively, concrete and steel} \\
& \text{fc, fc = allowable material stresses} \\
\text{Steel piles} & \quad P_{al} = A_p \cdot f_s \\
& \text{Ap = cross-sectional area of pile at cap} \\
& \text{fs: allowable steel stress}
\end{align*}
\]

Table 11.1

2. **Pile Capacity - Bearing Capacity Equations**

The ultimate bearing capacity of pile subjected to axial-compression force can be obtained as a contribution of the pile point-capacity plus the skin resistance from the stratum penetrated by the pile, that is,

\[
Q_u = Q_p + Q_s
\]

(11.2)
where

\[ Q_u = \text{ultimate bearing capacity of the pile} \]
\[ Q_p = \text{point-capacity contribution} \]
\[ Q_s = \text{skin resistance contribution} \]

The allowable bearing capacity is given by:

\[ Q_{all} = \frac{Q_u + Q_s}{FS} \quad (11.3) \]

where \( Q_s \) is small compared with \( Q_p \) the pile is treated as a point bearing pile and if \( Q_s \) is large compared with \( Q_p \) the pile is treated as a friction pile (see Fig. 11.1).

Figure 11.1. (a) and (b) Point bearing piles; (c) friction piles.

The expression to obtain \( Q_s \) and \( Q_p \) are presented below for cohesionless, cohesive and \( \phi-c \) soils.
Point Bearing Capacity

The ultimate point bearing capacity can be given as

\[ q_p = Ap \cdot q_p \]  \hspace{1cm} (11.4)

where

\[ q_p = c N_c + q' N_q \]  \hspace{1cm} (11.5)

and

\[ Ap = \text{area of pile at the pile tip} \]
\[ C = \text{cohesion of the supporting material} \]
\[ q' = \text{effective vertical stress at the level of pile tip} \]
\[ N_c^*, N_q^* = \text{bearing capacity factors} \]
\[ q_p = \text{unit point capacity} \]

The factors \( N_c^* \), \( N_q^* \) can be obtained by several methods. The Meyerhof's method and Vesic's method are commonly used.

The Meyerhof's method is used as follows:

a) Given \( \theta \), determine \( \frac{L_b}{D} \text{crit from Fig. 11.3.} \)

b) Determine \( \frac{L_b}{D} \), if \( \frac{L_b}{D} \leq 0.5 \frac{L_b}{D} \text{ and } \theta < 30^\circ \) obtain \( N_c^*, N_q^* \) from Fig. 11.2 taking the maximum values \( (N'c, N'q) \). If \( \frac{L_b}{D} < 0.5 \frac{L_b}{D} \text{crit, then} \)

\[ N_c^* = N_c + (N'c - N_c)(--\text{crit}) \]  \hspace{1cm} (11.5)

\[ \frac{L_b}{D} \]
\[
\frac{L_b}{D} (--) \\
N_a^* = \frac{N_q + (N_q - N_q)}{2} \frac{L_b}{D \cdot 0.5(--) \text{crit}}
\]

(11.7)

c) \( \phi > 30^\circ \) take maximum values \((N_c', N_q)\)

For sand, \( c = 0 \) equation (11.5) becomes

\[
Q_p = A_p q' N_q^* \quad \text{and} \quad q_L
\]

(11.8)

\[
q_L = 50 \frac{N_q^* \tan (--)}{m^2}
\]

(11.9)

For saturated clays, undrained conditions \((\phi = 0)\).

\[
Q_p = 9 C_u A_p
\]

(11.10)

where

- \( q = N_c^* \) from Fig. 11.2
- \( C_u = \) cohesion from unconfined compression test

The Vesic's method is used as follows:

\[
Q_p = A_p (cN_c^* + \sigma_{o'}^0 N_q^*)
\]

(11.11)

where

\[
\sigma_{o'}^0 = \frac{1 + 2 K_o}{3} q'
\]

\[
k_o = 1 - \sin \phi
\]

\( N_c^*, N_q^* = \) bearing capacity factors given in Table 11.1.

In Table 11.1,

\[
\frac{I_r}{I_{rr}} = \frac{I_r}{1 + I_r \Delta}
\]

(11.12)
\[ I_r = \text{rigidity index} = \frac{E_s \rho}{2(1 + \mu_s)(c + q' + \tan\phi)} \]

\[ G_s = \frac{G_s}{c + q' \tan\phi} \]

where

- \( G_s \) is the shear modulus of the soil

and

- \( \Delta \) is the average of volumetric strain in the plastic zone below the pile point.

Where there is not volume change (dense sand or saturated clay) \( \Delta = 0 \) and the equation 11.12 becomes \( I_r = I_r \).

Recommended values of \( I_r \) are shown below.

<table>
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<tr>
<th>Soil</th>
<th>( I_r )</th>
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<tbody>
<tr>
<td>Sand ((D_r = 0.5 - 0.8))</td>
<td>75-150</td>
</tr>
<tr>
<td>Silt and clay ((\text{drained conditions}))</td>
<td>50-100</td>
</tr>
<tr>
<td>Clays ((\text{Undrained condition}))</td>
<td>100-200</td>
</tr>
</tbody>
</table>

It can be noticed that the Vesic's method requires many work in the laboratory to obtain \( \phi, c, E_s, \mu \) and \( \gamma \).

Another method to obtain point bearing capacity is by means of the standard penetration test. Meyerhof (1976) suggests to obtain the value of \( Q_p \) as:
\[ G_p = \frac{A_p (38N)}{D^2} \left< 380 N(A_p) KN \right. \]  \hfill (11.13)

where

\[ N = \text{statistical average of the SPT numbers in a zone about 8D above to 3D below the pile point.} \]

\[ \frac{L_b}{A_p}, \frac{L_b}{D} \text{ as defined before (see Fig. 11.1)} \]

![Diagram](image)

**Figure 11.2**

To obtain the point bearing capacity, it is recommendable to use the three methods presented above and compare results to see which one gives critical values.

e. 'Frictional Resistance'

The skin resistance can be expressed as:

\[ Q_s = \sum p \Delta f \]  \hfill (11.14)

where

\[ p = \text{perimeter of pile section} \]
Figure 11.3 Critical embedment ratio and bearing capacity factors for various soil friction angles (after Meyerhof, 1976)
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Table 11.2. Bearing capacity factors for deep foundations, \( N_c \) and \( N_r \).
\[ L = \text{pile length} \]
\[ f = \text{unit friction resistance at any depth} \]

For sand "f" is expressed as
\[ f = k \sigma' v \tan \delta \] (11.15)

where
\[ K = \text{earth pressure coefficient} \]
\[ \sigma' v = \text{effective vertical stress at dept under study} \]
\[ \delta = \text{soil-pile friction angle} \]

K varies with depth, the following values are recommended.

<table>
<thead>
<tr>
<th>K</th>
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</thead>
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<tr>
<td>1. (1 - \sin \phi = K_0)</td>
<td>bored piles</td>
</tr>
<tr>
<td>2. (K_0 - \text{lower limit})</td>
<td>Low-displacement driven piles</td>
</tr>
<tr>
<td>1.4 (K_0 - \text{upper limit})</td>
<td></td>
</tr>
<tr>
<td>3. (K_0 - \text{lower limit})</td>
<td>High displacement driven piles</td>
</tr>
<tr>
<td>1.8 (K_0 - \text{upper limit})</td>
<td></td>
</tr>
</tbody>
</table>

Taken "K" as a constant and "\( \delta \)" as a constant which ranges of \(0.5 \phi\) to \(0.8 \phi\), the only term in equation 11.15 that varies with depth is \( \sigma' v \). In sand, that variation is from 15 to 20 times the pile diameter, therefore, knowing that \( f \) is a function of \( \sigma' v \), the variation of \( f \) with depth may be as shown in Fig. 11.4.

Then, the \( Q_s \) value may be obtained approximately and conservatively as,
\[ Q_s = p L' f_{av} + p (L - L') f_{av}^{\text{15D to L}} \] (11.16)
Figure 11.4. Variation of "f" with depth for piles on sand.

There are three methods currently used to compute skin resistance in cohesive soils, the $\lambda$ method, the $\beta$ method and, the $\alpha$ method. In general, in the three methods the skin resistance is given by equation 11.4.

$$Q_s = \sum p \Delta L (f) \text{ or } Q_s = p L f_{av}$$

The difference among them is how they obtain the value of the unit skin resistance, "f".

$\lambda$ Method: Proposed by Vijayvergiya and Focht (1972) obtains the value of $f$ as:

$$f_s = \lambda (\bar{\sigma}_v + 2 c_u)$$

(11.17)

where

$\lambda$ = value obtained from Fig. 11.5

$\bar{\sigma}_v$ = average effective vertical stress for the entire embedment length

$c_u$ = average undrained shear strength (\(\phi = 0\))
$\bar{\lambda}$ and $c_u$ can be obtained as shown in Fig. 11.6 for layered soils.

**Method:** Proposed by Tomlinson (1971) suggests to compute the unit skin resistance as:

$$f = \alpha c_u$$  \hspace{1cm} (11.16)

where $\alpha$ is an empirical factor that can be obtained from Fig.

---

**Figure 11.5.** Variation of $\lambda$ with pile embedment length (redrawn after McClelland, 1974).
Figure 11.6 Application of λ method in layered soils.

11.7 and $c_u$ as defined before. The skin resistance is given by

$$Q_s = \sum C_u \Delta L (\rho) \quad (11.19)$$

β Method: This method expresses the value of unit frictional skin as

$$f = \beta \sigma_v' \quad (11.20)$$

where

$$\beta = K \tan \phi_R$$

$\phi_R$ = drained friction angle of remodeled clay

$K$ = earth pressure coefficient

$\sigma_v'$ = vertical effective stress

The value of $K$ can be taken as $K_0$ or
\[ K = 1 - \sin \phi_R \text{ (normally consolidated clay)} \quad (11.21) \]
\[ K = (1 - \sin \phi_R) \sqrt{\sigma'_{cR}} \text{ (overconsolidated clay)} \quad (11.22) \]
plugging the values of K into equation (11.20), is obtained
\[ f = (1 - \sin \phi_R) \tan \phi_R \sigma'_{l} \text{ (normally consolidated clays)} \quad (11.24) \]
\[ f = (1 - \sin \phi_R) \tan \phi_R \sqrt{\sigma'_{cR}} \sigma'_{l} \text{ (overconsolidated clays)} \quad (11.25) \]

Again, \( Q_s \) is given by
\[ Q_s = \sum f \cdot P \Delta L \]

Goodman (1980) proposed the following equation for piles resting on rocks:
\[ q_p = q_u (N_\phi + 1) \quad (11.26) \]

where

![Graph showing variation of \( \alpha \) with \( C_u \)]

**Figure 11.7. Variation of \( \alpha \) with \( C_u \).**
\[ N = \tan^2(45 + \phi) \]

\[ q_u = \text{unconfined compression strength of rock} \]
\[ \phi = \text{drained angle of friction} \]

then, the allowable point capacity is expressed as:

\[ Q_p \text{ allowable} = \frac{[q_u(N\phi + 1)]A_p}{FS} \geq 3 \quad (11.27) \]

e. Efficiency of Pile Groups

Single piles are scarcely used, on the contrary, piles are used in groups. Typical pile-group patterns are shown in Fig. 11.8. It can be noted that the bearing capacity equations were presented for single piles. This equation is going to be used to obtain the bearing capacity of a group of piles. That group bearing capacity may decrease because of the overlapping of the stresses transmitted by the piles if they are spaced closely or that group capacity may remain as the total contribution of the piles if they are spaced adequately. The minimum allowable spacing of piles is normally given by codes, a common practice is to keep the minimum distance between centers of piles equal to 25.0 for round piles or two times the diagonal dimension for rectangular piles if those piles are driven to soil. For piles driven to rock, the minimum spacing should not be less than \( D + 300 \) mm for round piles or the diagonal plus 300 mm.
for rectangular piles. The stress overlapping is shown in Figure 11.9.

From the comments made above, it is important to determine how efficient a group of pile is in order to find out if the piles work at full capacity or this capacity is decreased. Hence, the efficiency of a group can be defined as the ratio of the actual group capacity to the sum of the individual pile capacities, that is,

\[ n = \frac{Q_{g(u)}}{\sum Q_u} \]  

(11.28)

where

- \( n \) = pile group efficiency
- \( Q_{g(u)} \) = actual group capacity
- \( \sum Q_u \) = sum of the individual capacities

**Piles in sand**

To obtain the group efficiency for piles on sand, the following expression can be used (see Fig. 11.10):

\[ n = \frac{2(m + n - 2)s + 4D}{Pmn} \]  

(11.29)

where,

- number of piles = \( m \times n \)
- \( p \) = perimeter of cross section of each pile
- \( D \) = pile diameter
- \( s \) = pile spacing
Figure 11.8. Typical pile group patterns; (a) for single footings; (b) for foundation walls.

Figure 11.9. Stresses surrounding a friction pile and the summing effects.
Figure 11.10. Pile group.

Then, from equation 11.28 the group capacity is given by

\[ Q_g(u) = n \sum Q_u \]

where \( Q_u \) is obtained as explained before, that is

\[ Q_u = p \cdot L \cdot f_{av} \]

If "n" is bigger than one, use one.

**Piles in clay**

The following steps can be followed to obtain the group capacity of piles in clay.

1. Determine pile bearing capacity as if the piles behave individually and sum each capacity, that is,

\[ \sum Q_u = mn(Q_p + Q_s) \]

Equation (11.10)

\[ Q_p = C_u \cdot A_p \cdot \sigma_{(p)} \]

Equation (11.19)
\[ Q_s = \sum_{p} C_u \Delta L \]

then

\[ Q_u = mn(9 C_u \frac{A_p}{(p)} + \sum_{p} C_u \Delta L) \quad (11.30) \]

2. Determine pile bearing as if the piles behave as a group, that is,

\[ Q_p = Lg Bg C_u \frac{N_c^*}{(p)} \]

where the value \( N_c^* \) can be obtained from Fig. 11.11 and

\[ Lg = (m - 1)s + D \]
\[ Bg = (n - 1)s + D \]

and

\[ Q_s = \sum Bg C_u L = \sum 2(Lg + Bg)C_u \Delta L \]

then

\[ Q_{g(u)} = Lg Bg C_u \frac{N_c^*}{(u)} + \sum 2(Lg + Bg)C_u \Delta L \quad (11.31) \]

3. Compare the two values \( Q_{g(u)} \) and \( \sum Q_u \) and use the lower one for design.

**Piles in rock**

For piles on rock, the group bearing capacity can be taken as the sum of the individual capacities, \( Q_{g(u)} = \sum Q_u \) if the spacing for piles on rock given above are respected.

**f. Pile Settlements**

The consolidation settlement for a group of piles can be obtained using the 2:1 method to calculate the induced
Figure 11.11. Variation of $N_q$ with $L/B$ and $H/B$-based on Bjerrum and Eide's equation.

stresses and using the equation (2.10). The following considerations have to be taken into account (see Fig. 11.12).

1. The load on the group is

$Q_T$ if the pile cap is above the original ground surface

$Q_T = G_T - \gamma_{soil \ above \ Pile \ Group}$

remove by excavation

where

$Q_T$ is the total load from the superstructure.

2. The induced stresses begin to act at a depth of $2/3$ $L$ from the top of the pile.
Figure 11.12. Induced-stresses Pile Group.

The elastic settlement for a pile group may be obtained by the equation proposed by Vesic (1969).

\[ S_g(e) = \sqrt{\frac{B_g}{D S_e}} \quad (11.33) \]

where

- \( S_g(e) \) = elastic settlement of a group of piles
- \( B_g \) = width of pile group section
- \( D \) = width or diameter of each pile in the group
- \( S_e \) = elastic settlement of each pile given by equation 11.32

\[ S_e = \frac{(Q_{wp} + \xi Q_{ws})}{A_p E_p} L + \frac{Q_{wp} D}{A_p E_s} (1 - \mu^2) I_{wp} + \]

197
\[
\frac{Q_{WP}}{P} \frac{D}{L} \frac{1 - \mu^2}{E_s} I_{WS} = (11.33)
\]

where

\(Q_{WP}\) = load carried at pile point under working load conditions

\(Q_{WS}\) = load carried by skin resistance under working load conditions

\(A_p\) = area of pile cross-section

\(L\) = length of pile

\(E_p\) = Young's modulus of the pile material

\(E_s\) = Young's modulus of the soil

\(D\) = pile diameter

\(\mu_s\) = Poisson's ratio of soil

\(p\) = pile perimeter

\(\xi\) = factor which depends on the nature of the skin resistance along the shaft pile

\[\xi = 0.5\]

\[\xi = 0.5\]

\[\xi = 0.67\]

\(I_{WP}\) = influence factor that can be obtained from Fig. 11.13 using \(r\) values

\(I_{WS}\) = factor given by \(I_{WS} = 2 + 0.35 \sqrt{\frac{L}{D}}\)

\(g.\) Pile Driving Formulas and Pile Load Tests

As already indicated, dynamic formulas or pile load
tests are used on site to determine if the piles develop the capacity at which they were designed. There is a variety of formulae available to obtain bearing capacity when a pile is being driven. Here will be presented the modified Engineering News Record formula (ENR) only. This formula expresses the ultimate load capacity \( Q_u \) as

\[
Q_u = \frac{EWRh}{S + C} \frac{Wr + n^2 Wp}{WR + Wp}
\]

(11.34)

where

\[
E = \text{hammer efficiency (given below)}
\]

<table>
<thead>
<tr>
<th>Hammer type</th>
<th>( E )</th>
</tr>
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<tbody>
<tr>
<td>Single-double acting hammers</td>
<td>0.7-0.85</td>
</tr>
<tr>
<td>Diesel hammers</td>
<td>0.8-0.9</td>
</tr>
<tr>
<td>Drop hammers</td>
<td>0.7-0.9</td>
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</table>

\( Wr = \text{weight of the ram (specified value)} \)

\( h = \text{height of fall of the ram} \)

\( s = \text{penetration of pile per hammer blow (usually based on the average obtained from the last few driving blows).} \)

\( c = \text{0.254 cm or 0.1 in dependent on units of} \ s \text{ and} \ h \)

\( n = \text{coefficient of restitution between ram and pile cap} \)
Figure 11.13. Values of $\alpha$, $\alpha_{av}$, and $\alpha_r$.

Pile material

<table>
<thead>
<tr>
<th>Material Description</th>
<th>Factor $n$</th>
</tr>
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<tbody>
<tr>
<td>Cast iron hammer and concrete pile (without cap)</td>
<td>0.4-0.5</td>
</tr>
<tr>
<td>Wood cushion on steel piles</td>
<td>0.3-0.4</td>
</tr>
<tr>
<td>Wooden piles</td>
<td>0.25-0.3</td>
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</tbody>
</table>

$W_p = \text{weight of the pile}$

A factor of safety of 4-6 may be used in equation 11.34.

Pile load tests are conducted in the field to obtain data which permit to plot a curve load versus net settlement as shown in Fig. 11.4. From that curve a value of $Q_u$ is
obtained. Pile load tests should be conducted after several days the pile has been driven in granular soils but in cohesive soils that test should be carried out after a certain period of time (30-90 days) to permit the soil gain its thixotropic strength that is part of its original shear strength.

![Diagram of Load vs Settlement](image)

**Figure 11.14. Pile-load Test Curve.**

**Example 11.1**

Find the allowable bearing capacity of the pile group shown in Fig. below, if they are embedded in a saturated homogeneous clay.

- $D = 406$ mm
- $S = 850$ mm
- $L = 18.5$ m
- $C_u = 95.8$ kN/m$^2$
- $m = 3$ $n=3$
1. Piles working individually

\[ \sum Q_u = \text{mm} (9 \times c_u \times A_P + \sum \alpha \times p \times c_u \Delta L) \]  

\[ \alpha = 0.53 \text{ from Fig. 11.7 for } c_u = 9.58 \text{ K}_{m^2} \]

\[ A_P = \frac{\pi (0.406)^2}{4} = 0.1295 \text{ m}^2 \]

\[ p = \pi (0.406) = 1.2755 \text{ m} \]

\[ \Delta L = 18.5 \text{ m (the stratum is homogeneous)} \]

\[ \sum Q_u = 3 \times 3 \left[ (9 \times 95.8 \times 0.1295) + (0.53 \times 1.2755 \times 95.8 \times 18.5) \right] \]

\[ \sum Q_u = 11787.81 \text{ KN} \]

2. Piles working as a group

\[ Qg(u) = Lg \times Bg \times c_u \times N_c^* + \sum 2 (Lg + Bg) \times c_u \times L \]  

\[ Lg = (m-1)s + D = (3-1)0.850 + 0.406 = 2.106 \text{ m} \]

\[ Bg = (n-1)s + D = (3-1)0.850 + 0.406 = 2.106 \text{ m} \]

\[ \frac{Lg}{Bg} = 1 \quad \frac{L}{Bg} = \frac{18.5}{2.106} = 8.78 \quad N_c^* = 9 \]

\[ Qg(u) = (2.106 \times 2.106 \times 95.8 \times 9) + 2 (2.106 + 2.106) \times 95.8 \times 18.5 \]

\[ Qg(u) = 18753.92 \text{ KN} \]

3. \( Qg(u) \times \sum Q_u \) use \( \sum Q_u = 11787.81 \)

\[ Q_a = \frac{11787.81}{3} = 3929.27 \text{ KN} \]

If the group of piles is embedded in sand, what is the efficiency of the group?
\[ n = \frac{2(m + n - 2)s + 4D}{P_{mn}} \quad \text{Eq. 11.29} \]

\[ n = \frac{2(3 + 3 - 2)0.850 + 4(0.406)}{1.2755(3)} = 0.74 \]

then the efficiency is 74\%. 
BIBLIOGRAPHY


# CLASS OUTLINE

*Text - Braja M. Das -- Principles of Foundation Engineering, Brooks/Cole Engineering Division, 1984*

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NOTES ON FOUNDATION ENGINEERING

by

JOSE O. CHAVEZ

Diploma in Civil Engineering
Universidad Nacional Autonoma de Honduras
Honduras, 1981

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AN ABSTRACT OF A MASTER'S REPORT

submitted in partial fulfillment of the
requirements for the degree

MASTER OF SCIENCE

Department of Civil Engineering

Kansas State University
Manhattan, Kansas

1985
ABSTRACT

Soils have been a civil engineering material widely used since ancient times. Man has used the soil for different purposes: to build a road, to build an earth dam, to support structures and so on. In spite of being the soil, a common engineering material, its behavior under load as well as its properties were not well understood until Terzaghi's work in 1925. Since that time, analytical methods have been introduced to explain the soil behavior under different load conditions and laboratory tests have been improved to determine soil properties required in those analytical methods. Today, it is possible to predict, with an acceptable degree of accuracy, the soil bearing capacity and the settlement which soil could suffer when it is being submitted to load.