/NOTES ON FOUNDATION ENGINEEERING/

by

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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 masonary 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.
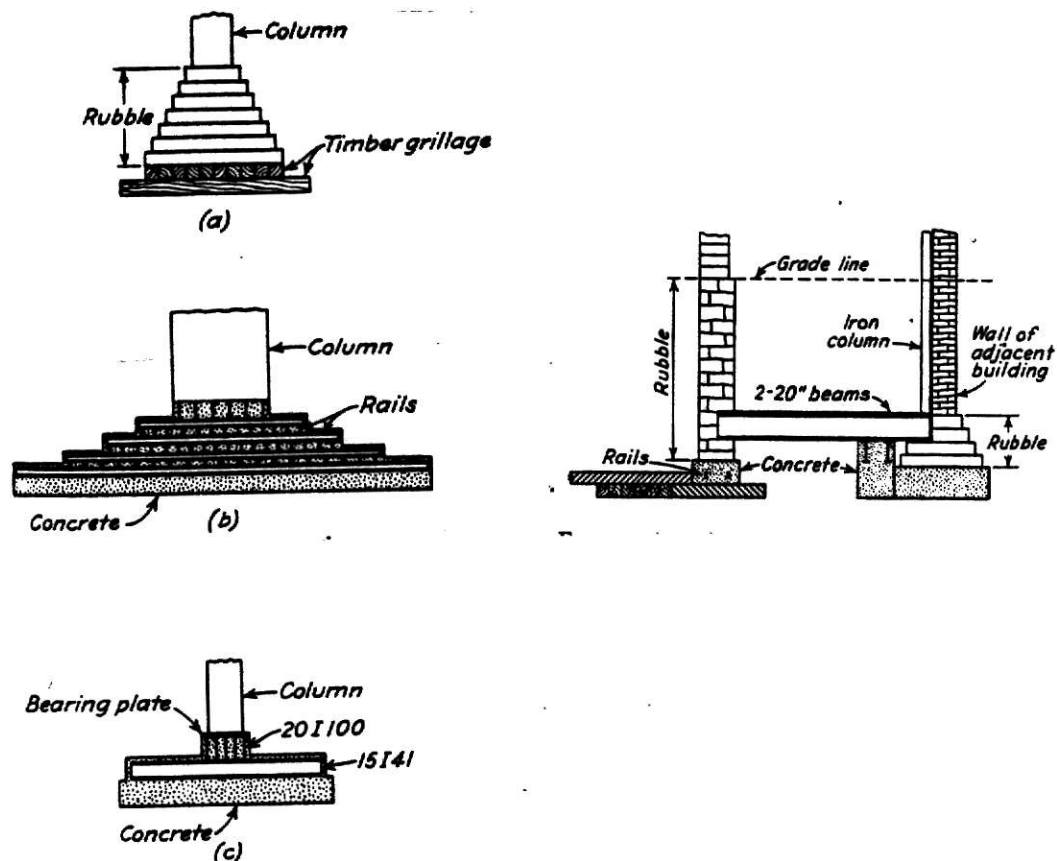
Figure 1.1. Historical development of grillage foundations of (a) timber, (b) railroad rails, (c) steel I-beams, (d) Cantilever footing supporting exterior column of Auditorium Building, Chicago, 1887.

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.
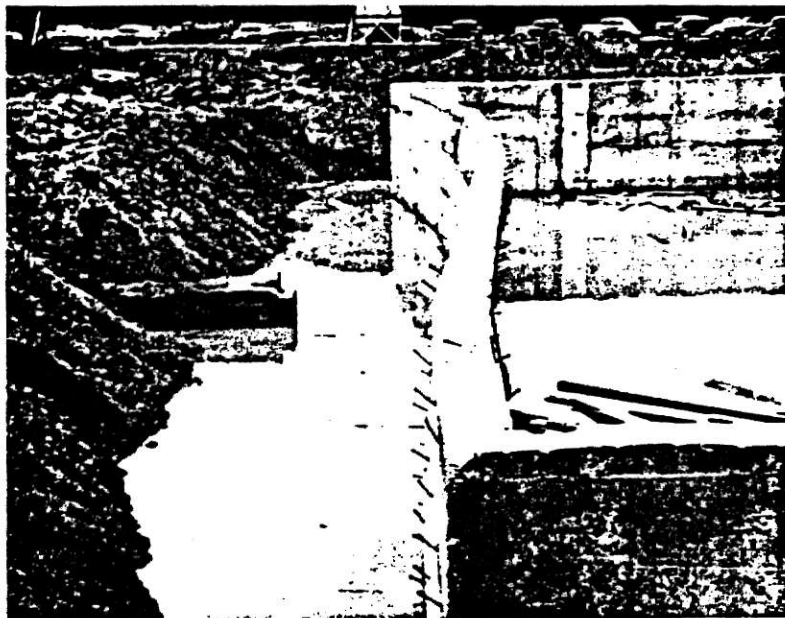
Figure 35 — Clay backfill after a storm.
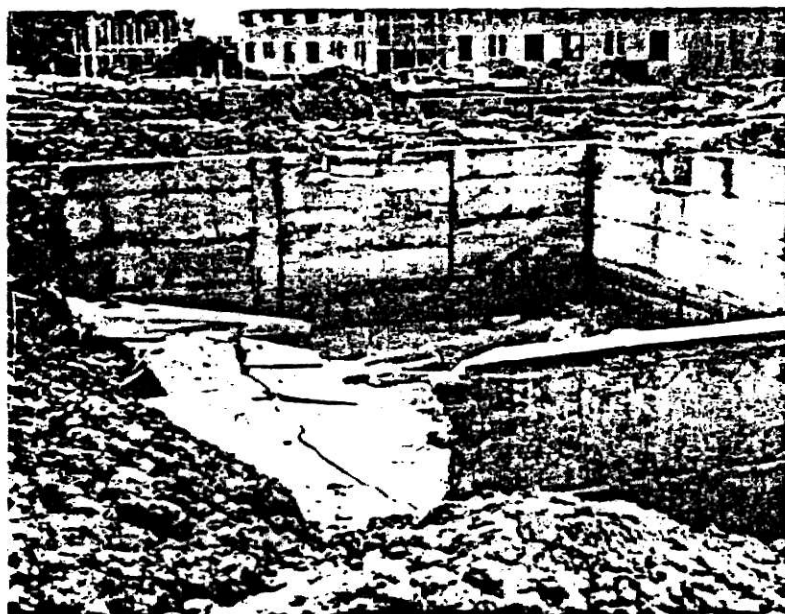
Figure 1.2.   Clay backfill and water collapsed the wall.

Clay backfill which became liquid after a heavy storm
provoked lateral pressures greater than for which it was
designed.

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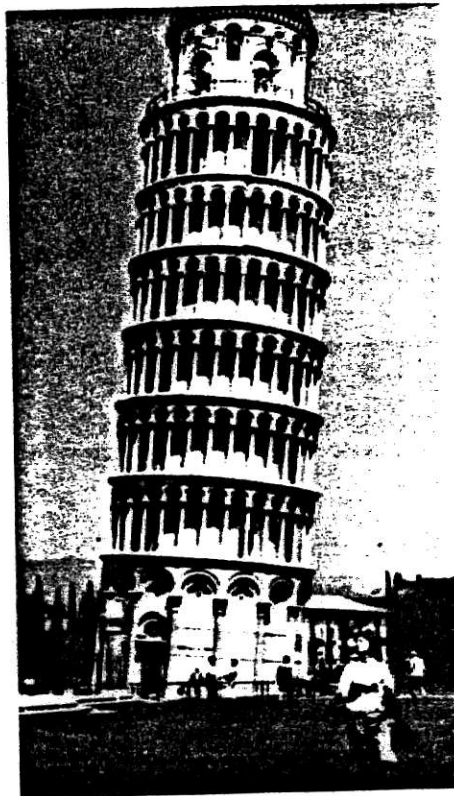
Figure 1.3. Pisa Tower. The most successful foundation failure.

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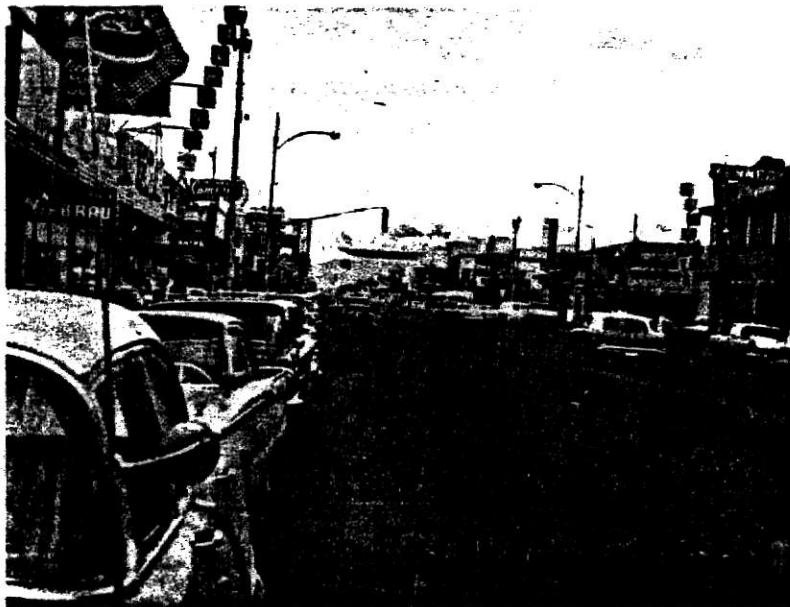
Figure 1.4.   4th Avenue, Anchorage, May 1963.

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.

Foundation type	Use	Applicable soil conditions
Spread footing. wall footings	Individual columns. walls. bridge piers	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
Mat foundation	Same as spread and wall footings. Very heavy column loads. Usually reduces differential settlements and total settlements	Generally soil bearing value is less than for spread footings: over one-half area of building covered by individual footings. Check settlements
Pile foundations Floating	In groups (at least 2) to carry heavy column. wall loads: requires pile cap	Poor surface and near surface soils. Soils of high bearing capacity 20–50 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
Bearing	In groups (at least 2) to carry heavy column. wall loads: requires pile cap	Poor surface and near-surface soils: soil of high bearing capacity (point bearing on) is 8–50 m below ground surface
Caisson (shafts 75 cm or more in diameter) generally bearing or combination of bearing and skin resistance	Larger column loads than for piles but eliminates pile cap by using caissons as column extension	Poor surface and near-surface soils: soil of high bearing capacity (point bearing on) is 8–50 m below ground surface
Retaining walls. bridge abutments	Permanent retaining structure	Any type of soil. but a specified zone (Chaps. 11. 12) in back of.wall usually of controlled backfill
Sheet-pile structures	Temporary retaining structures as excavations. waterfront structures. cofferdams	Any soil: waterfront structure may require special alloy or corrosion protection. Cofferdams require control of fill material

Table 1. Foundation types and typical usage.

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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

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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 stratums 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:

$$e = \frac{v_v}{v_s} \qquad (2.1)$$

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

$$n = \frac{v_v}{v} \qquad (2.2)$$

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.

$$w(\%) = \frac{w_w}{w_s} \times 100 \qquad (2.3)$$

Unit weight, ____: The ratio of the weight of soil to the corresponding volume.

$$\gamma = \frac{w_t}{v} \qquad (2.4)$$

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

$$\text{moisture unit weight} \quad \gamma_m = \frac{w_t}{v}$$

where $W_t = W_s + W_w$. Assuming the weight of the air is equal to zero.

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Dry unit weight $\gamma_d = \dfrac{W_s}{V}$

Saturated unit weight $\gamma_s = \gamma_m$ if $v_v = v_w$

Submerged unit weight $\gamma_{sub} = \gamma_s - \gamma_w$

<u>Degree of saturation, S</u>: the ratio of the volume of water to the total volume of soil voids.

$$S(\%) = \frac{v_w}{v_v} \times 100 \qquad (2.5)$$

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

<u>Specific gravity, G_s</u>: The ratio of the unit weight of a material in air to the unit weight of distilled water at 4°C.

$$G_s = \frac{w_s/v_s}{w_w/v_w} = \frac{w_s/v_s}{w} \qquad (2.6)$$

$\gamma_w = 62.4$ 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.

Soil type	G_s
Quartz sand	2.64–2.66
Silt	2.67–2.73
Clay	2.70–2.9
Chalk	2.60–2.75
Loess	2.65–2.73
Peat	1.30–1.9

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Table 2-2. Void Ratio, Moisture Content, and Dry Unit Weight
for Some Soils.

Type of soil	Void ratio e	Natural moisture content in saturated condition (%)	Dry unit weight γ_d (kN/m³)
Loose uniform sand	0.8	30	14.5
Dense uniform sand	0.45	16	18
Loose angular-grained silty sand	0.65	25	16
Dense angular-grained silty sand	0.4	15	19
Stiff clay	0.6	21	17
Soft clay	0.9–1.4	30–50	11.5–14.5
Loess	0.9	25	13.5
Soft organic clay	2.5–3.2	90–120	6–8
Glacial till	0.3	10	21

Example 2.1

- Underlined information in Table is the data given:

- Obtain the required data in each case.

G Specific Gravity	Moisture w(%)	Saturation S %	e Void Ratio	n Porosity	γ_d (pcf)	$\gamma_{sat.}$ (pcf)	$\gamma_{sub.}$ (pcf)	$\gamma_{total.}$ (pcf)
2.65	20	84.2	0.6292	0.3862	101.50	125.6	63.2	121.8
2.65	18	66.2	0.72	0.4186	96.14	122.1	59.8	113.44
2.65	20	100	0.5300	0.3464	108.08	129.6	67.2	129.69
2.65	20	50	1.060	0.5146	80.27	112.3	49.9	96.32

<u>Soil 1</u>
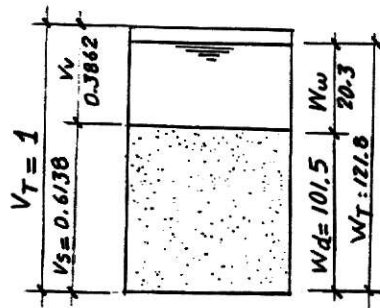
$$w = \frac{w_W}{w_S} \;, \quad 0.20 = \frac{w_W}{w_S} \;, \quad 0.20\, w_S = w_W \quad (1)$$

$$G = \frac{w_S}{v_S\; 62.4} \;, \quad 2.65(6.24)v_S = w_S \quad (2)$$

$$\gamma_{sub} = \gamma_{sat} - \gamma_w \qquad\qquad (3)$$

$$63.2 = \gamma_{dry} + 62.4\, v_V - 62.4$$

$$63.2 = 2.65(62.4)v_S + 62.4(v_V) - 62.4$$

$$125.6 = 165.36\, v_S + 62.4\, v_V.$$

$$\gamma_d = 2.65(0.6138)62.4 = 101.50 \text{ pcf}$$

$$\gamma_{sat} = 101.5 + 62.4(0.3862) = 125.6 \text{ pcf}; \quad v_V + v_S = 1 \quad (4)$$

Solving (3) and (4).

$$\gamma_{tot} = 101.5 + 62.4(0.3253) = 121.8 \text{ pcf}$$

$$\gamma_{sub} = 125.6 - 62./4 = 63.2 \text{ ok}$$

$$S = \frac{v_W}{v_V} : \frac{0.3253}{0.3862} : 0.842$$

$$: 84.27\%$$

Solving (3) and (4)

$$w_W = 0.20(101.50) = 20.3; \quad 125.6 + 165.36\,(1 - v_V) + 62.4\, v_V$$

$$v_W = \frac{20.3}{62.4} = 0.3253 \qquad v_V = 0.3862$$

$$v_S = 0.6138.$$

$$e = \frac{v_V}{v_S} = \frac{0.3862}{0.6138} \;, \quad e = 0.6292$$

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$$n = \frac{v_v}{vt_s} = \frac{0.3862}{1} : 0.3862.$$

Soil 2.

$$e = \frac{v_v}{v_s} \quad 0.72 \; v_s = v_v \qquad (1)$$

$$v_v + v_s = 1 \quad v_v = 1 - v_s \qquad (2)$$

Solving (1) and (2).

$$0.72 \; v_s = 1 - v_s$$

$$v_s = 0.5814$$

$$v_v = 0.4186$$

$\gamma_{sat} = 96.14 + 62.4(0.4186) : 122.26$ pcf

$$w_s = 2.65(0.5814)(62.4) = 96.14 \rightarrow \gamma_d$$

$\gamma_{sub} = 122.26 - 62.4 = 59.86$ pcf

$$w = \frac{w_w}{w_s} \;, \quad 0.18 = \frac{w_w}{96.14} \;, \quad w_w = 17.30$$

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

$$v_w = \frac{17.30}{62.4} = 0.2773$$

$$s = \frac{v_w}{v_v} = \frac{0.2773}{0.4186} = 0.662 \;, \quad 66.27\% = s$$

$$n = \frac{v_v}{v_t} = \frac{0.4186}{1} = 0.4186$$

<u>Soil 3.</u>

$$w = \frac{w_w}{w_s} \ , \ 0.20 = \frac{w_w}{w_s} \ , \ 0.20 \ w_s = w_w \ (1)$$

$$S = \frac{v_w}{v_v} = 1 \ , \ v_w = v_v$$

From Eq. (1)

$0.20 \ v_s \ (2.65) = v_w$

$0.53 \ v_s = v_w \ (2).$

$v_w + v_s = 1 \ (3)$ the soil is saturated.

Solving (3) and (2)

$0.53 \ v_s = 1 - v_s$

$v_s = 0.6536$

$v_w = 0.3464$

$$e = \frac{0.3464}{0.6536} : 0.5300$$

$$n = \frac{0.3464}{1} = 0.3464.$$

$\gamma_d = 2.65(62.4)(0.6536) = 108.08$

$\gamma_v = 108.08 + 62.4 \ (0.3464) = 129.69 = \gamma_{sat.}$

$\gamma_{sub} = 129.69 - 62.4 = 67.29$

<u>Soil 4.</u>

$$w = \frac{w_w}{w_s} = 0.20 \ , \ 0.20 \ w_s = w_w. \qquad (1)$$

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$$s = \frac{v_W}{v_V} = 0.5 \quad , \quad v_W = 0.5 \, v_V. \quad \quad (2)$$

From (1).

$v_V + v_S = 1$ $\quad \quad 0.20(2.65)(62.4)(v_S) = 62.4(0.5 \, v_V)$

$$0.20(2.65)(62.4)(1 - v_V) = 62.4(0.5 \, v_V)$$

$$v_V = 0.5146$$

$$v_W = 0.2573$$

$$v_S = 0.4854$$

$$e = \frac{0.5146}{0.4854} = 1.060$$

$$n = \frac{0.5146}{1} = 0.5146$$

$\gamma_d = 2.65(62.4)(0.4854) = 80.27 \text{ pcf}$

$\gamma_{tot} = 80.27 + 62.4(0.2573) = 96.32 \text{ pcf}$

$\gamma_{sat} = 80.27 + 62.4(0.5146) = 112.38 \text{ pcf}$

$\gamma_{sub} = 112.38 - 62.4 = 49.98 \text{ pcf}$

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-

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Test	Data Obtained	Used For
1. Specific Gravity (Gs)	Ratio between density of soil and density of water	Auxiliary factor to compute other soil properties (n,c) in consolidation studies of clay, degree of saturation of a soil, etc.
2. Atterberg Limits	Liquid Limit (LL) Plastic Limit (PL) Plastic Index (PI)	Classification of soils (cohesive) The LL can give some idea about how expansive the soil is
3. Grain Size Analysis	Grain Size Distribution curve	Classification of soils (size particle bigger than No. 200 sieve)
4. Hydrometer Analysis	Grain Size Distribution curve	Classification of soils (size particle smaller than No. 200 sieve)
5. Water Content	Content of Water in Soil	Auxiliary factor to calculate other Soil Properties
6. Void ratio and porosity	e: void ratio n: porosity	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
7. Compaction Test	Optimum moisture content and maximum dry density	Used for determining the soil moisture-density relationship of a soil used for building a fill (road, dam)
8. Permeability Test	K: coefficient of permeability	Studying: quantity of leakage through and under dams Rate of consolidation Stability of slopes, embankments Seepage velocity through the soil
9. Consolidation test	: compression index : coefficient of consolidation	to obtain the amount of settlement to obtain the rate of settlement with time
10. Direct Shear Test	Shear resistance ( ) in terms of , c. $= c + N + g$	Determining the possible bearing capacity of the soil in foundations and earthwork
11. Triaxial compression test	Shear resistance ( ) in terms of , "c"	Making estimates of the probable bearing capacity, stability calculations of

		pore water pressure	earthworks, earth retaining structures and foundations. Analyzing stress-strain relationships of loaded soils
12.	Unconfined Compression Test	Shear resistance ( ) in terms of "c"	Measuring the consistency of cohesive soils giving an idea about rupture of embankments, slopes. Determining bearing capacity of the soil
13.	Vane shear test	Shear strength of the soil in terms of torsional moments	Obtaining the shearing resistance of the soil when it is not possible to run unconfined compression or tri-axial test
14.	California Bearing Ratio (CBR)	CBR which is a comparative measure of the shearing resistance of the soil	Designing asphalt pavement structures
15.	Plate Bearing Test	The strength of any elevation in an asphalt pavement	The design and evaluation of asphalt pavements
16.	Standard Penetration test (SPT)	N" values	Obtaining the bearing capacity of soils directly
17.	Pile load tests	Pile capacity" or ultimate pile load", pile head	To obtain the capacity of a pile to support loads

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 23) is used in foundation design.

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

Symbol	Description
G	Gravel
S	Sand
M	Silt
C	Clay
O	Organic silts and clay
P_t	Peat and highly organic soils
H	High plasticity
L	Low plasticity
W	Well graded
P	Poorly graded

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,

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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 (ϕ)

3. cohesion between particles (c).

Hence, the shear strength is given by Coulomb as

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

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

where s = shear strength

c = cohesion

, ϕ = friction angle

σ = 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.

Major divisions			Group symbols	Typical names	Classification criteria
Coarse-Grained Soils — More than 50% retained on No. 200 sieve[b]	Gravels — 50% or more of coarse fraction retained on No. 4 sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines	$C_u = D_{60}/D_{10}$ Greater than 4 $C_z = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines	Not meeting both criteria for GW
		Gravels with Fines	GM	Silty gravels, gravel-sand-silt mixtures	Atterberg limits plot below "A" line or plasticity index less than 4 / Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixtures	Atterberg limits plot above "A" line and plasticity index greater than 7
	Sands — More than 50% of coarse fraction passes No. 4 sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines	$C_u = D_{60}/D_{10}$ Greater than 6 $C_z = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			SP	Poorly graded sands and gravelly sands, little or no fines	Not meeting both criteria for SW
		Sands with Fines	SM	Silty sands, sand-silt mixtures	Atterberg limits plot below "A" line or plasticity index less than 4 / Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures	Atterberg limits plot above "A" line and plasticity index greater than 7
Fine-Grained Soils — 50% or more passes No. 200 sieve[b]	Silts and Clays — Liquid limit 50% or less		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands	
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
			OL	Organic silts and organic silty clays of low plasticity	
	Silts and Clays — Liquid limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts	
			CH	Inorganic clays of high plasticity, fat clays	
			OH	Organic clays of medium to high plasticity	
Highly Organic Soils			PT	Peat, muck, and other highly organic soils	

Classification on basis of percentage of fines:
Less than 5% pass No. 200 sieve: GW, GP, SW, SP
More than 12% pass No. 200 sieve: GM, GC, SM, SC
5% to 12% pass No. 200 sieve: Borderline classification requiring use of dual symbols

Plasticity Chart

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

Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols.

Equation of A line:
$$PI = 0.73(LL - 20)$$

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Table 2.5. Typical properties of compacted soils.

Group Symbol	Soil Type	Range of Maximum Dry Unit Weight, pcf	Range of Optimum Moisture, Percent	Typical Value of Compression — Percent of Original Height		Cohesion (as compacted) psf	Cohesion (saturated) psf	φ (Effective Stress Envelope Degrees)	Tan φ	Typical Coefficient of Permeability ft./min.	Range of CBR Values	Range of Subgrade Modulus k lbs/cu in.
				At 1.4 tsf (20 psi)	At 3.6 tsf (50 psi)							
GW	Well graded clean gravels, gravel-sand mixtures.	125 – 135	11 – 8	0.3	0.6	0	0	>38	>0.79	5×10^{-2}	40 – 80	300 – 500
GP	Poorly graded clean gravels, gravel-sand mix	115 – 125	14 – 11	0.4	0.9	0	0	>37	>0.74	10^{-1}	30 – 60	250 – 400
GM	Silty gravels, poorly graded gravel-sand-silt.	120 – 135	12 – 8	0.5	1.1	.....	.....	>34	>0.67	$>10^{-6}$	20 – 60	100 – 400
GC	Clayey gravels, poorly graded gravel-sand-clay.	115 – 130	14 – 9	0.7	1.6	.....	.....	>31	>0.60	$>10^{-7}$	20 – 40	100 – 300
SW	Well graded clean sands, gravelly sands.	110 – 130	16 – 9	0.6	1.2	0	0	38	0.79	$>10^{-3}$	20 – 40	200 – 300
SP	Poorly graded clean sands, sand-gravel mix.	100 – 120	21 – 12	0.8	1.4	0	0	37	0.74	$>10^{-3}$	10 – 40	200 – 300
SM	Silty sands, poorly graded sand-silt mix.	110 – 125	16 – 11	0.8	1.6	1050	420	34	0.67	5×10^{-5}	10 – 40	100 – 300
SM-SC	Sand-silt clay mix with slightly plastic fines.	110 – 130	15 – 11	0.8	1.4	1050	300	33	0.66	2×10^{-6}	5 – 30	100 – 300
SC	Clayey sands, poorly graded sand-clay-mix.	105 – 125	19 – 11	1.1	2.2	1550	230	31	0.60	5×10^{-7}	5 – 20	100 – 300
ML	Inorganic silts and clayey silts.	95 – 120	24 – 12	0.9	1.7	1400	190	32	0.62	$>10^{-5}$	15 or less	100 – 200
ML-CL	Mixture of inorganic silt and clay.	100 – 120	22 – 12	1.0	2.2	1350	460	32	0.62	5×10^{-7}	.....	
CL	Inorganic clays of low to medium plasticity.	95 – 120	24 – 12	1.3	2.5	1800	270	28	0.54	$>10^{-7}$	15 or less	50 – 200
OL	Organic silts and silt-clays, low plasticity.	80 – 100	33 – 21	.....	.....	.....	.....	.....	.....	.....	5 or less	50 – 100
MH	Inorganic clayey silts, elastic silts.	70 – 95	40 – 24	2.0	3.8	1500	420	25	0.47	5×10^{-7}	10 or less	50 – 100
CH	Inorganic clays of high plasticity	75 – 105	36 – 19	2.6	3.9	2150	230	19	0.35	$>10^{-7}$	15 or less	50 – 150
OH	Organic clays and silty clays	65 – 100	45 – 21	.....	.....	.....	.....	.....	.....	.....	5 or less	25 – 100

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. (>) indicates that typical property is greater than the value shown.
   (...) indicates insufficient data available for an estimate.

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TYPICAL NAMES OF SOIL GROUPS	GROUP SYMBOLS	IMPORTANT PROPERTIES				RELATIVE DESIRABILITY FOR VARIOUS USES *									
		PERMEABILITY WHEN COMPACTED	SHEARING STRENGTH WHEN COMPACTED AND SATURATED	COMPRESSIBILITY WHEN COMPACTED AND SATURATED	WORKABILITY AS A CONSTRUCTION MATERIAL	ROLLED EARTH DAMS			CANAL SECTIONS		FOUNDATIONS		ROADWAYS		
						HOMOGENEOUS EMBANKMENT	CORE	SHELL	EROSION RESISTANCE	COMPACTED EARTH LINING	SEEPAGE IMPORTANT	SEEPAGE NOT IMPORTANT	FILLS FROST HEAVE NOT POSSIBLE	FILLS FROST HEAVE POSSIBLE	SURFACING
WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	GW	PERVIOUS	EXCELLENT	NEGLIGIBLE	EXCELLENT	—	—	1	1	—	—	1	1	1	3
POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	GP	VERY PERVIOUS	GOOD	NEGLIGIBLE	GOOD	—	—	2	2	—	—	3	3	3	—
SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES	GM	SEMIPERVIOUS TO IMPERVIOUS	GOOD	NEGLIGIBLE	GOOD	2	4	—	4	4	1	4	4	9	5
CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES	GC	IMPERVIOUS	GOOD TO FAIR	VERY LOW	GOOD	1	1	—	3	1	2	6	5	5	1
WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	SW	PERVIOUS	EXCELLENT	NEGLIGIBLE	EXCELLENT	—	—	3 IF GRAVELLY	6	—	—	2	2	2	4
POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	SP	PERVIOUS	GOOD	VERY LOW	FAIR	—	—	4 IF GRAVELLY	7 IF GRAVELLY	—	—	5	6	4	—
SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES	SM	SEMIPERVIOUS TO IMPERVIOUS	GOOD	LOW	FAIR	4	5	—	8 IF GRAVELLY	5 EROSION CRITICAL	3	7	8	10	6
CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES	SC	IMPERVIOUS	GOOD TO FAIR	LOW	GOOD	3	2	—	5	2	4	8	7	6	2
INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS WITH SLIGHT PLASTICITY	ML	SEMIPERVIOUS TO IMPERVIOUS	FAIR	MEDIUM	FAIR	6	6	—	—	6 EROSION CRITICAL	6	9	10	11	—
INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS	CL	IMPERVIOUS	FAIR	MEDIUM	GOOD TO FAIR	5	3	—	9	3	5	10	9	7	7
ORGANIC SILTS AND ORGANIC SILT-CLAYS OF LOW PLASTICITY	OL	SEMIPERVIOUS TO IMPERVIOUS	POOR	MEDIUM	FAIR	8	8	—	—	7 EROSION CRITICAL	7	11	11	12	—
INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS	MH	SEMIPERVIOUS TO IMPERVIOUS	FAIR TO POOR	HIGH	POOR	9	9	—	—	—	8	12	12	13	—
INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	CH	IMPERVIOUS	POOR	HIGH	POOR	7	7	—	10	9 VOLUME CHANGE CRITICAL	9	13	13	8	—
ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY	OH	IMPERVIOUS	POOR	HIGH	POOR	10	10	—	—	—	10	14	14	14	—
PEAT AND OTHER HIGHLY ORGANIC SOILS	PT	—	—	—	—	—	—	—	—	—	—	—	—	—	—

* NOTE NO 1 IS THE BEST

Table 2.4. Engineering use chart

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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.
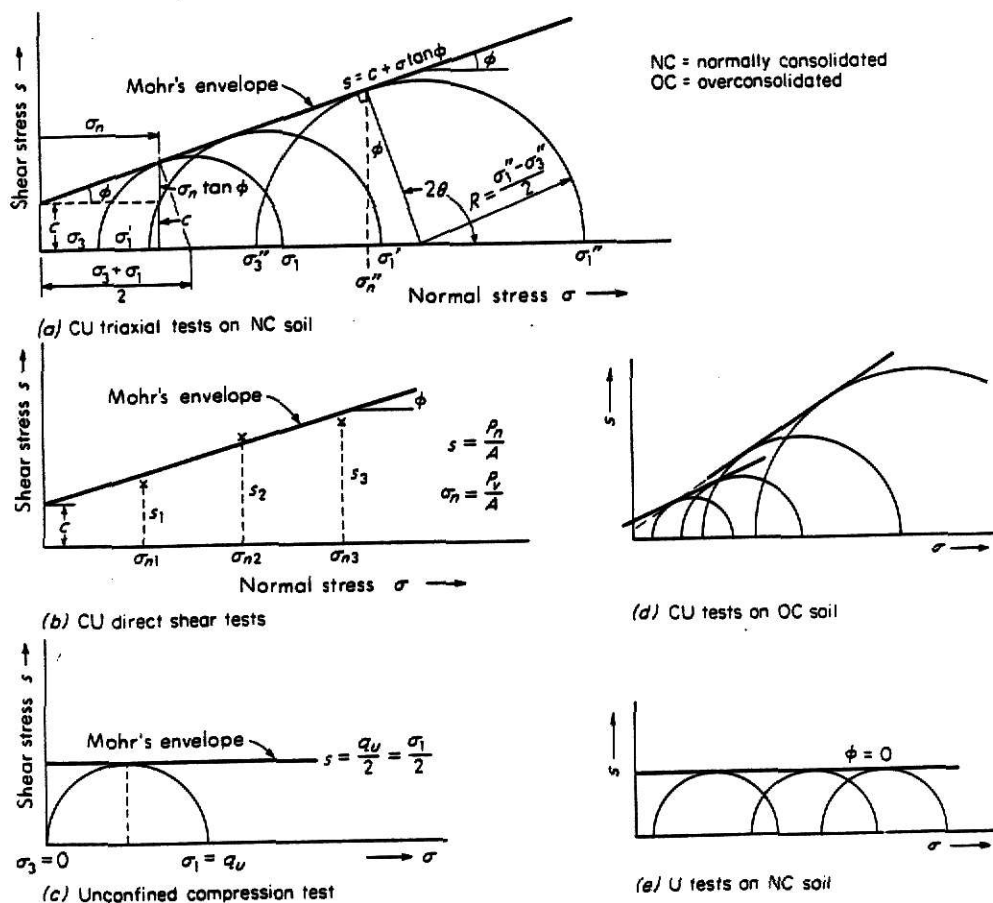
Figure 2.1. Mohr's circles and rupture envelopes for several shear-strength tests shown.

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:

Cell pressure σ_3 (lb/in²)	15	25	35
Principal stress difference at failure $\sigma_1 - \sigma_3$ (lb/in²)	20	21	19

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:

Normal stress (lb/in²)	8·9	17·9	26·8
Shear stress at failure (lb/in²)	10·5	14·3	18·6

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)

-27
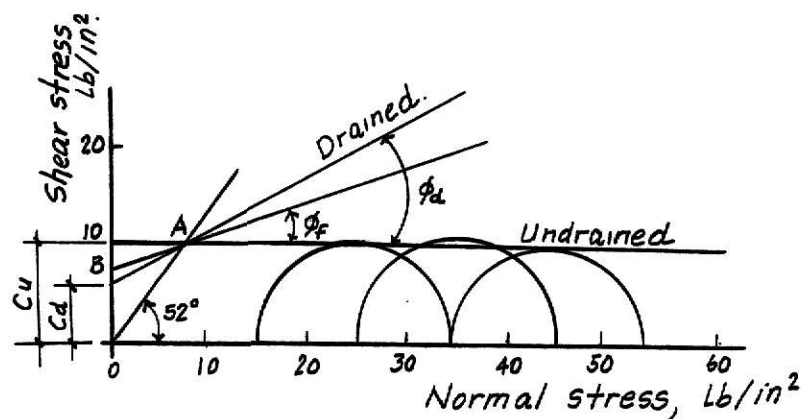
the best Coulomb line gives c = 10 lb/in² and ϕ_u = 0°. 45°
+ $\phi_f/2$ = 52°, whence ϕ_f = 14°.

A line drawn from the origin at 52° to the horizontal
axis represents the plane of rupture for the case of σ_3 = 0.
This line intersects the Coulomb line at A. From A, draw a
line inclined at 14° to the horizontal. This intersects the
shear stress axis at B, and OB scales 8 lb/in², which is the
true cohesion.

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

  c_d = 6.5 lb/in² and ϕ_d = 24°

Example 2.3

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

Lateral pressure σ_3 (lb/in²)	10	50
Total vertical pressure σ_1 (lb/in²)	43·5	128·0
Pore-water pressure U (lb/in²)	− 4·0	13·5

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 lb/in², and
ϕ_u = 21° (see Fig. below).

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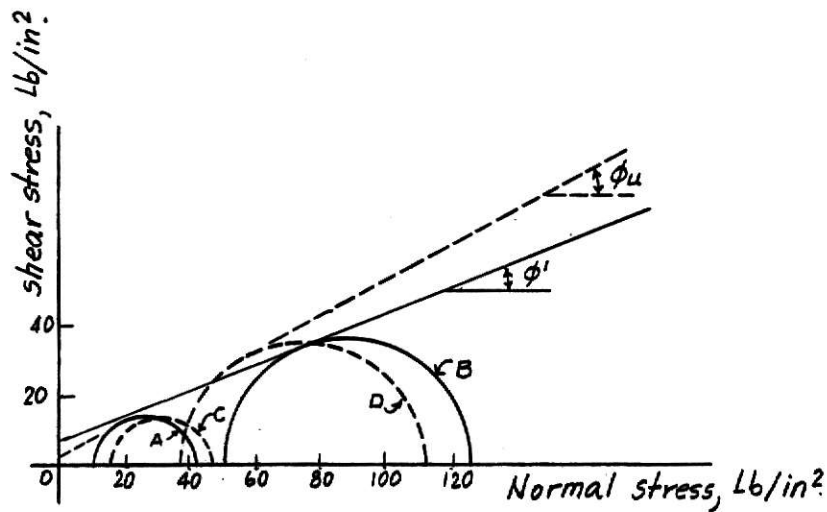
For (b) the effective stresses $(\sigma - u)$ are:

σ_1'   43.5 + 4.0 = 47.5   128.0 - 13.5 = 114.5
σ_3'   10.0 + 4.0 = 14.0   50.0 - 13.5 = 36.5

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 lb/in² and ϕ' = 30° (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} $\frac{cm}{sec}$ 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 $10^{-3} < k < 10^{-4} \frac{cm}{sec}$ 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 $k \simeq 10^{-5} \frac{cm}{sec}$, the time required to dissipate the pore water pressure is usually too long, therefore undrained conditions must be used to run the test 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 $10^{-4} < k < 10^{-6} \frac{cm}{sec}$ 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 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.

$$t = 0 \qquad \Delta \sigma = \Delta p$$
$$\Delta u = (\Delta hi)\, \gamma_w = \Delta p$$
$$\Delta \sigma' = \Delta \sigma - \Delta u = 0$$

$$t = \infty \qquad \Delta \sigma = \Delta P$$
$$\Delta u = 0$$
$$\Delta \sigma' = \Delta P - \Delta u$$
$$\Delta \sigma' = \Delta P.$$

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:

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$$t = 0 \qquad \Delta\sigma = \Delta p$$

$$\Delta u = (\Delta hi)\,\gamma_w = \Delta p$$

$$\Delta\sigma' = \Delta\sigma - \Delta u = 0$$

$$t = \infty \qquad \Delta\sigma = \Delta P$$
$$\Delta u = 0$$
$$\Delta\sigma' = \Delta P - \Delta u$$
$$\Delta\sigma' = \Delta P.$$

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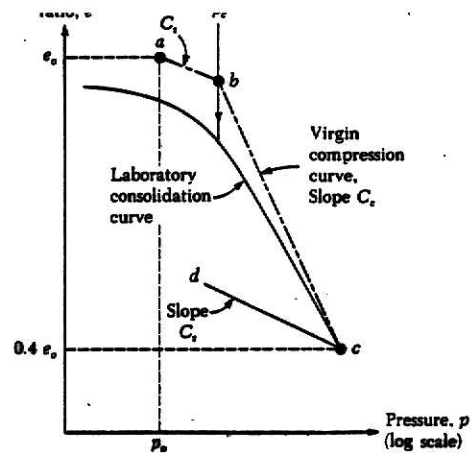
31

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 sumbitted. It can be obtained by the Casagrande method. The value obtained is compared with the current effective overburden pressure, p_o.

If

1. $p_o = p_c$ ($\pm$ about 10%) soil normally consolidated

2. $p_o > p_c$ specimen may have excessive disturbance

3. $p_o < p_c$ 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.

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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
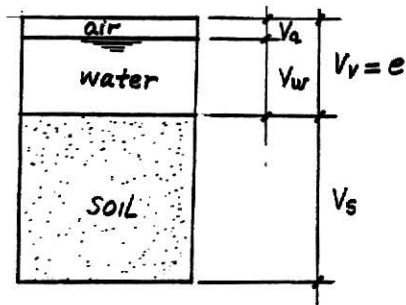
$$e = \frac{v_V}{v_S} \quad \text{if } v_S = 1, \ e = v_V,$$

then the initial volume of the soil v_O is

$$v_O = v_S + v_V$$

$$v_O = 1 + e_O$$

where e_O = 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:

$$s_{unit} = \frac{v_f - v_O}{v_O} \quad \text{where } v_f = \text{final volume.}$$

$$v_f = 1 + e_{final}$$

$$s_{unit} = \frac{1 + e_{final} - (1 + e_O)}{1 + e_O}$$

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$$s_{unit} = \frac{\Delta e}{1 + e_O} \qquad\qquad (2.8)$$

If the stratum of soil has a thickness H_C, the total settlement, s is equal to

$$s = \frac{\Delta e}{1 + e_O} H_C \qquad\qquad (2.9)$$

It is apparent that the value to be computed is the Δe 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, Δp due to loads, is added to the existing overburden pressure p_O. Then from equation 2.7, the change in void ratio becomes: $c = c_C \log \frac{p + \Delta p}{p_O}$ and substituting this value into equation 2.9, the settlement is expressed as:

$$s = \frac{c_C H_C}{1 + e_O} \log \frac{p_O + \Delta p}{p_O}$$

or

$$s = \frac{c_C H_C}{1 + e_O} \log \left(1 + \frac{\Delta p}{p_O}\right) \qquad\qquad (2.10)$$

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

(a) $p_O + \Delta p < p_C$ (see Fig. 2.4)

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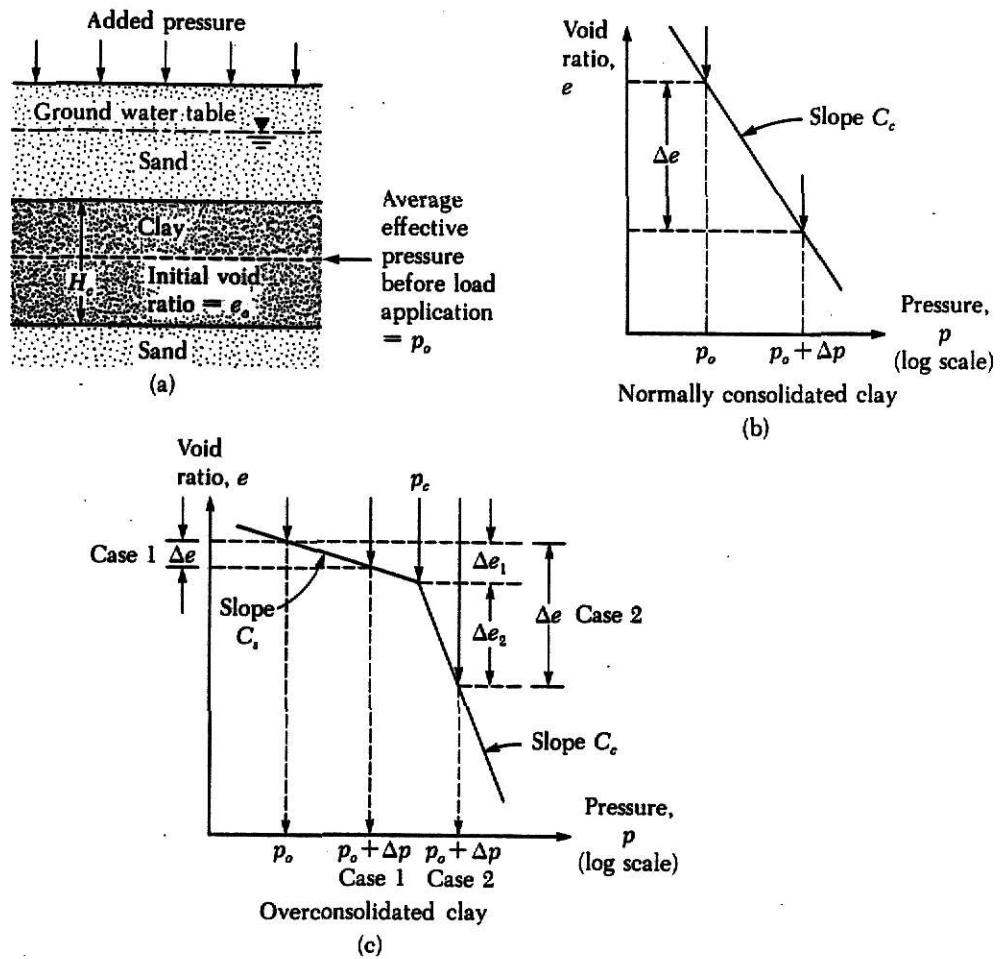Again, $\Delta e = c_s \log \dfrac{p_o + \Delta p}{p_o}$ (2.11)

(a)

Normally consolidated clay

(b)

Overconsolidated clay

(c)

Figure 2.4. One-dimensional settlement calculations: (b) is for Eq. (2.10); (c) is for Eqs. (2.13) and (2.14).

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and

$$s = \frac{c_s \ H_c}{1 + e_o} \ \log \ (1 + \frac{\Delta P}{P_o}) \qquad\qquad (2.12)$$

(b) $p_o + \Delta p > p_c$. 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,

$$\Delta e = \ \Delta e_1 + \ \Delta e_2 \ or \ \Delta e = c_s \ \log \ \frac{P_c}{P_o} \ +$$

$$c_c \ \log \ (1 + \frac{\Delta p}{P_o}) \qquad\qquad (2.13)$$

and the total settlement becomes:

$$s = \frac{c_s \ H_c}{1 + e_o} \ \log \ \frac{P_c}{P_o} + \frac{c_c \ H_c}{1 + e_o} \ \log \ \frac{P_o + \Delta p}{P_c} \qquad (2.14)$$

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:

$$\frac{\partial (\Delta u)}{\partial t} = \frac{c_v \ \partial^2 (\Delta u)}{\partial z^2} \qquad\qquad (2.15)$$

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where $\dfrac{\partial (\Delta u)}{\partial t}$ = rate of change of the excess
                              pore water pressure

$\dfrac{\partial^2 (\Delta u)}{\partial z^2}$ = change in hydraulic pressure gradient

$c_v \dfrac{\partial^2 (\Delta u)}{\partial z^2}$ = 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).
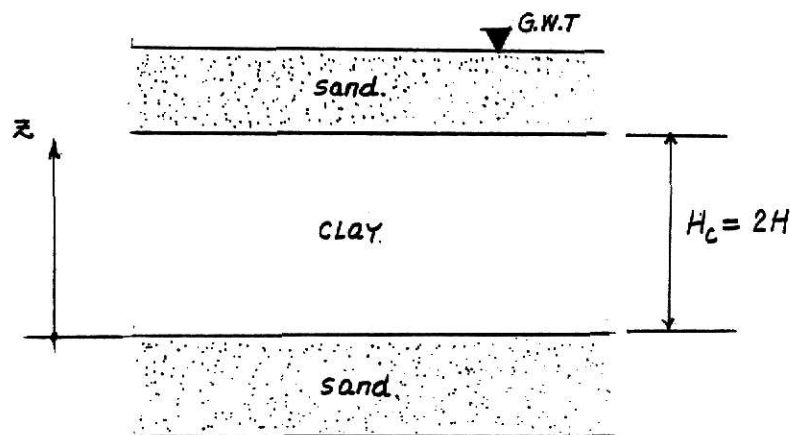
Figure 2.5

and at time $t = 0$, $\Delta u = \Delta u_0$ where Δu_0 is the initial pore

water pressure after load application. A solution for Eq.
(2.15) is given by

$$\Delta u = \sum_{m = 0}^{m = \infty} [\frac{2(\Delta u_O)}{M} \sin (\frac{Mz}{H})] \exp. ^{-M^2 T_v} \qquad (2.16)$$

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 = an integer = 1,2, ...

z = depth where the excess pore water pressure is
computed

H = 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.

Figure 2.6.

- 39
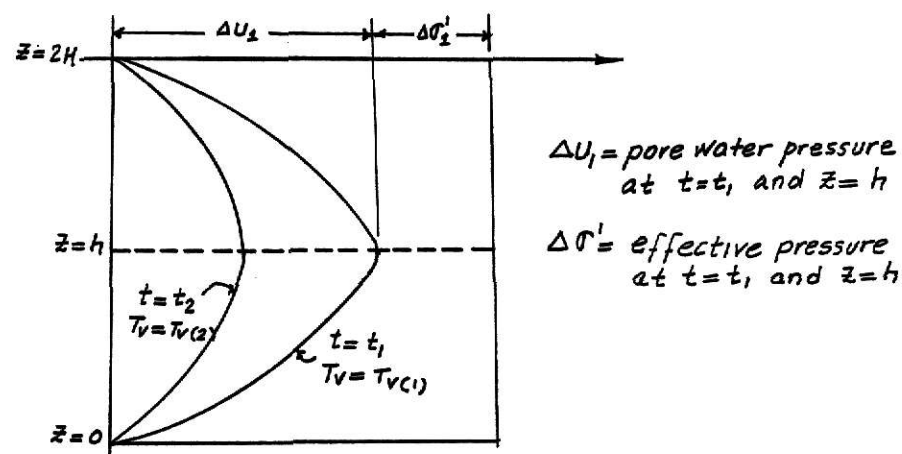
It must be remembered that at any time and at any depth, the total stress is given by $\Delta\sigma_t = \Delta\sigma' + \Delta u$, then at

$$t = 0 \qquad \Delta\sigma_t = 0 + \Delta u = \Delta p$$

$$t = t_1 \qquad \Delta_t = \Delta\sigma'_1 + \Delta u_1 = \Delta p$$

$$t = \infty \qquad \Delta\sigma_t = \Delta\sigma' + 0 = \Delta p$$

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_{max}, after full consolidation at a given loading has been achieved. That is

$$U = \frac{s_t}{s_{max}} \times 100 \qquad\qquad (2.17)$$

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,

$$s_t = \int_0^{2H} (\Delta u_0)\, dz - \int_0^{2H} (\Delta u)\, dz \quad \text{(change in area in diagram of Fig. 2.6)}$$

$$s_{max} = \int_0^{2H} (\Delta u_0)\, dz \quad \text{(maximum area enclosed in diagram of Fig. 2.6)}$$

$$U = \frac{(\Delta u_0)2H - \int_0^{2H} \Delta u\, dz}{(\Delta u_0)\, 2H}$$

or

$$U = 1 - \frac{\int_0^{2H} \Delta u \, dz}{(\Delta u_o) \, 2H} \qquad (2.18)$$

where Δu is given by Eq. 2.16).

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

$$U = 1 - \sum_{m=0}^{\infty} \left(\frac{2}{M^2}\right) \exp.^{-M^2 T_V} \qquad (2.19)$$

A curve of the time factor T_V versus degree of consolidation U is shown in Fig. 2.7.
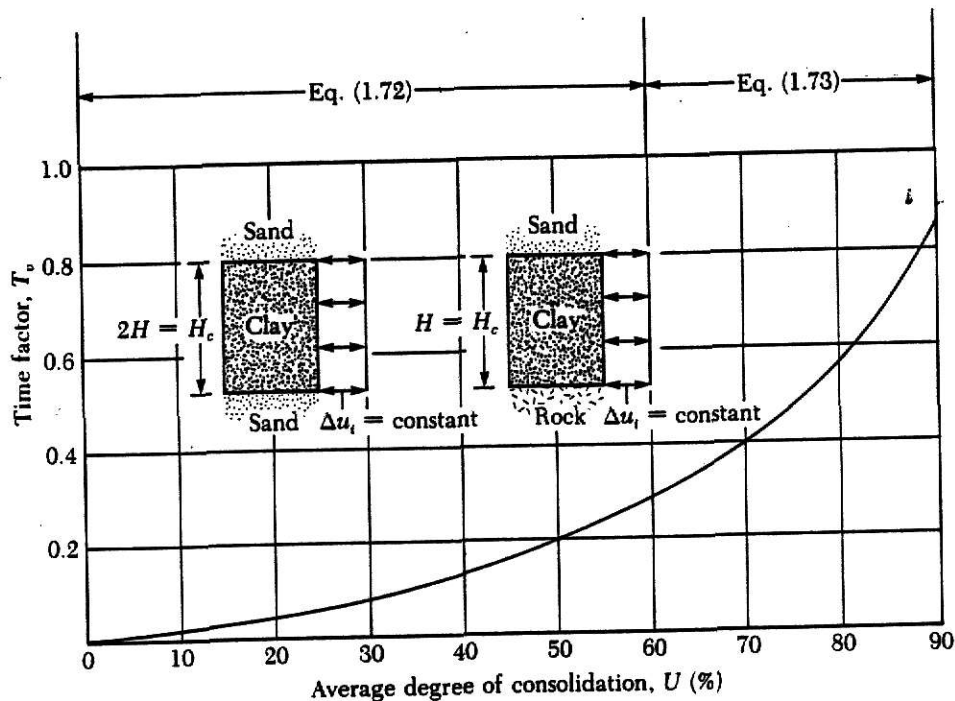

Figure 2.7. Plot of time factor against average degree of consolidation (Δu_o = constant)

The value of T_V is given by the following approximated relationships:

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

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

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 P_3 to obtain <u>zero percent of consolidation line</u>.

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.

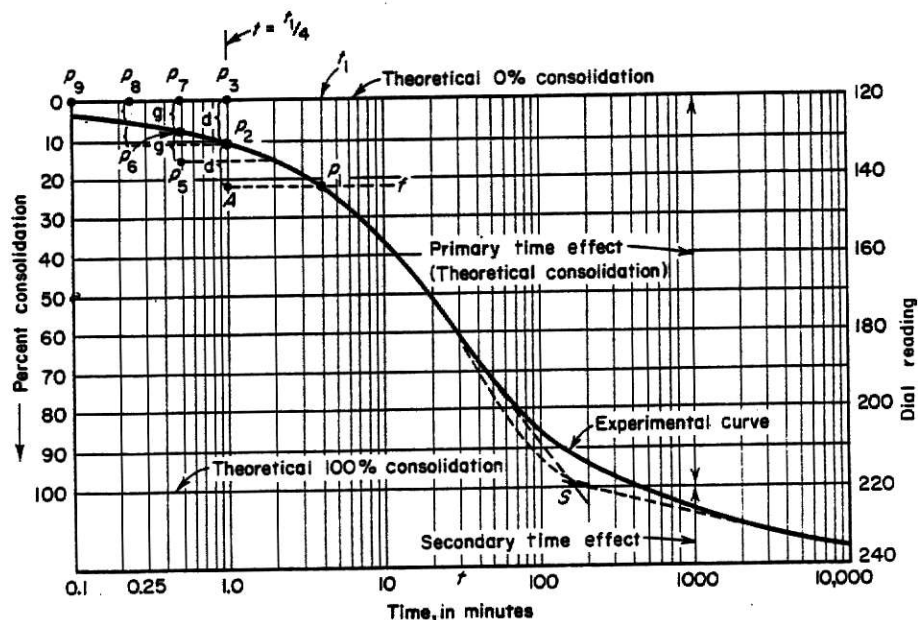8. The distance between the zero and 100% is divided into equal parts, for instance each 10%.

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

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$$C_v = \frac{0.197 \ H^2}{t_{50}}$$

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

$$t = \frac{T_v \ H_f^2}{C_v}$$

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.
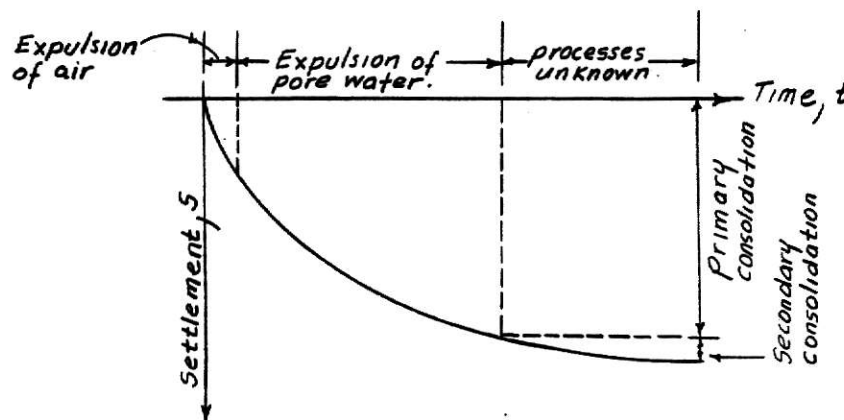
Figure 2.9. Elements of time-settlement curve of a cohesive soil at constant load.

Elastic Properties of the Soil

The modulus of elasticity E_s, the shear modulus G, and the Poisson's ratio $_s$, 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.

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

Table 2.6. Typical range of values for the static stress-strain modulus E, for selected soils. Field values depend on stress history, water content, density, etc.

Type of soil	μ
Clay, saturated	0.4–0.5
Clay, unsaturated	0.1–0.3
Sandy clay	0.2–0.3
Silt	0.3–0.35
Sand (dense)	0.2–0.4
Coarse (void ratio = 0.4–0.7)	0.15
Fine-grained (void ratio = 0.4–0.7)	0.25
Rock	0.1–0.4 (depends somewhat on type of rock)
Loess	0.1–0.3
Ice	0.36
Concrete	0.15

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 + D_f. (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

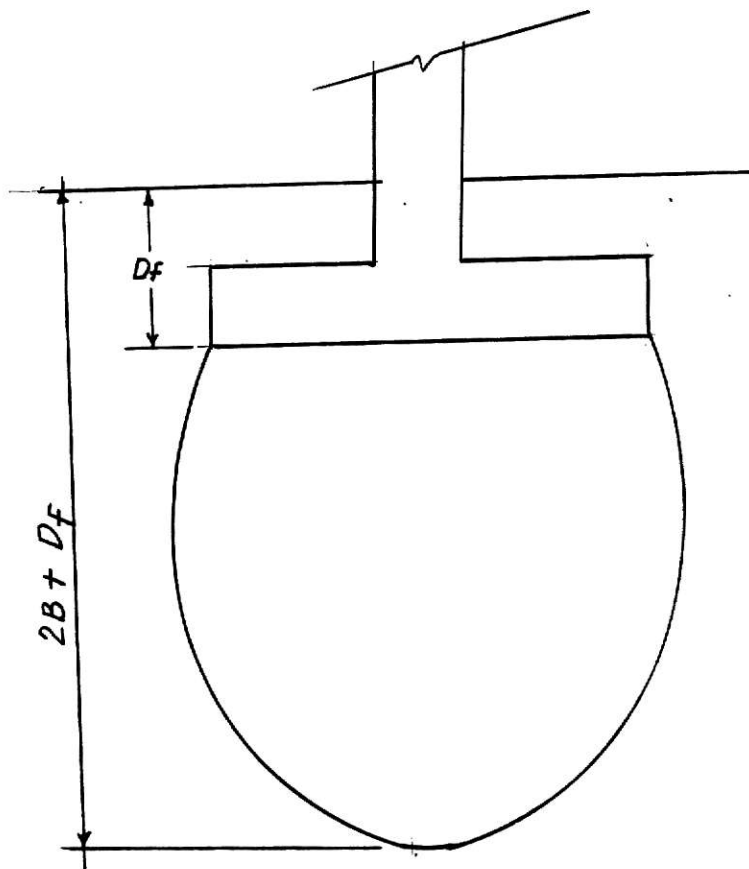6. The boring logs drawn to a stated scale.

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 Boussinesg 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.

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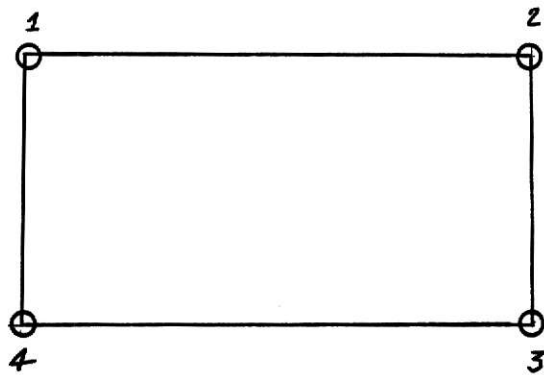
Possible boring location for regular stratified soil.

Possible depth of borings.

Figure 3.1

1	2	3	4	5	6	7	8	9	10	11	12	13	14
Scale of depth below ground surface in feet	Depth to layer in feet	Depth to bottom of sampling tube in feet	Sample tube and Sample Nos.	Recovery in.	Recovery, Jar sample	Soil tests	Nos. of blows per foot	Symbol	Number of blows per foot on casing	Description	Wet and dry unit weights in lb/ft³	Moisture content by dry weight in %	Moisture distribution diagram

0.0 Ground Surface   Ground-water Table

Top Soil

Light gray silty clay mottled with dark-yellow (rusty looking) oxidation stains. Channeled with vertical roots of vegetal matter. Stiff, plastic. Very wet. Foul odor.

Triaxial test
Some little frags. of slate. Dark gray clayey silt. Foul odor. Very wet.
Consolidation test.
The strong odor suggests gas from decomposed organic matter. Contains tiny mica particles and fragments of dark gray slate.

Consolidation test

Tan, stiff, homogeneous clay. Foul odor.

Consolidation test

Dark gray silty clay, brownish-gray in some spots. Foul odor. Some varving may be seen.

Dark gray silty clay, varved somewhat. Foul odor.

Consolidation test

Explanations
U.C. = Unconfined compression test
Consolidation test
J.S. = Jar (spoon) samples
L.L. = Liquid limit
P.L. = Plastic limit
P.I. = Plasticity index

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. 4.6). 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
$(\simeq \frac{B}{2})$ 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 = relative density

D_f = depth of foundation

$B*$ = $\dfrac{2BL}{B+L}$

B = width of footing

L = 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

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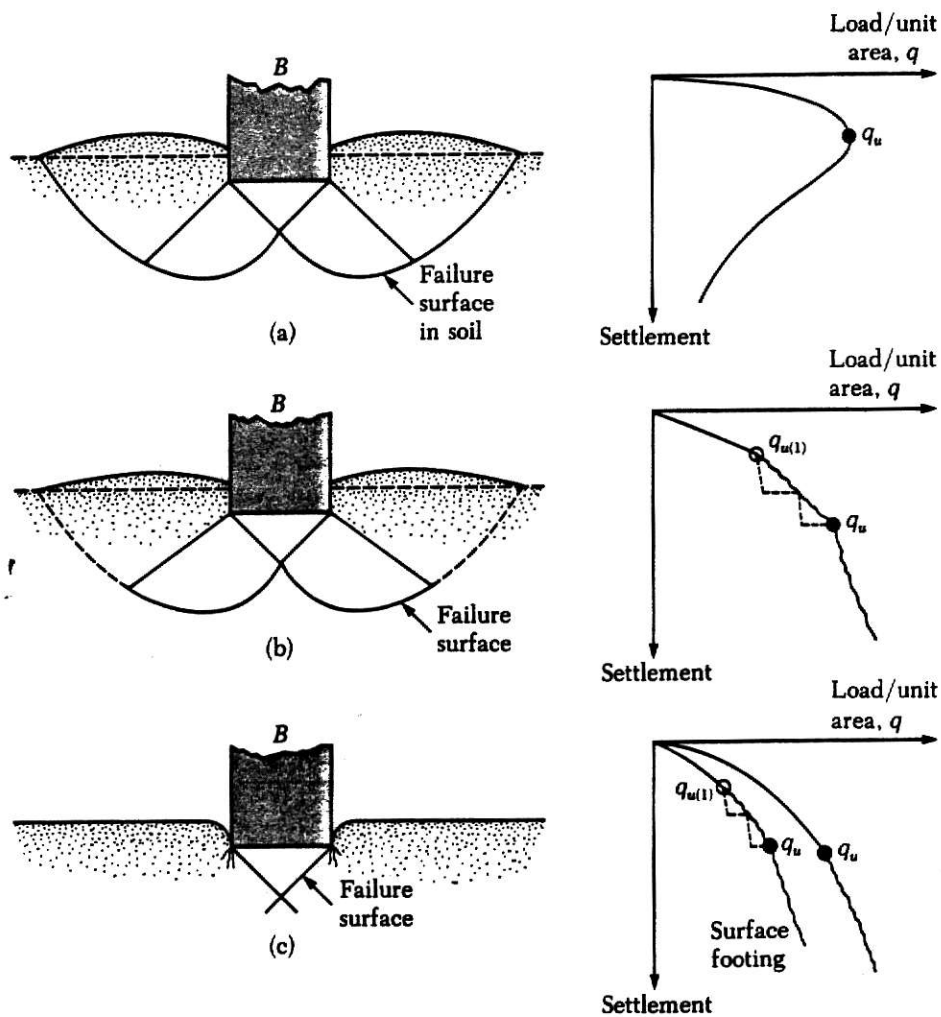
Nature of bearing capacity in soil: (a) General shear failure; (b) Local shear failure; (c) Punching shear failure (redrawn after Vesic, 1973).
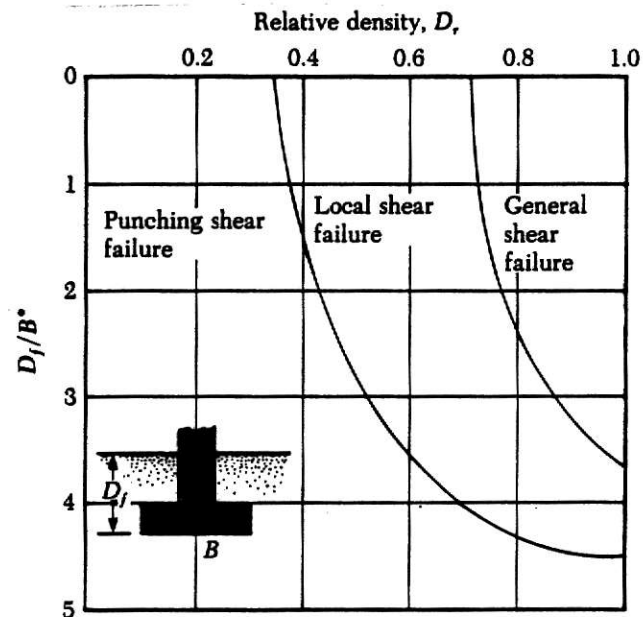
Figure 4. Modes of foundation failure in sand (after Vesic, 1973).
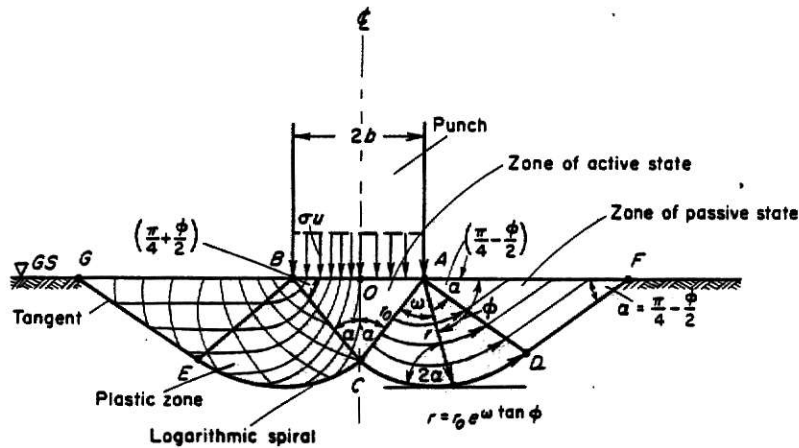
Figure 41. Prantl's system of study.

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

$$\sigma_u = \frac{c}{\tan\phi} [\tan^2(45° + \frac{\phi}{2}) e^{\pi\tan} - 1] \qquad (4.1)$$

The system used by Prandth 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' = $rt\tan\phi$ and

t = equivalent height of surcharge of soil given by

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$$t = \frac{\text{area of wedges and sector}}{\text{length GEL}} \quad \text{(one half of the system)}$$

γ: soil unit weight.

$$\sigma_u = \frac{c + c^1}{\tan\phi} \ [\tan^2(45^\circ + \phi/2) \ e^{\pi\tan} - 1] \qquad 4.2$$

Now, if $c = 0$, the $\sigma_u = 0$.

(a) Forces in Prandtl's system

(b) Stress diagram in compression test
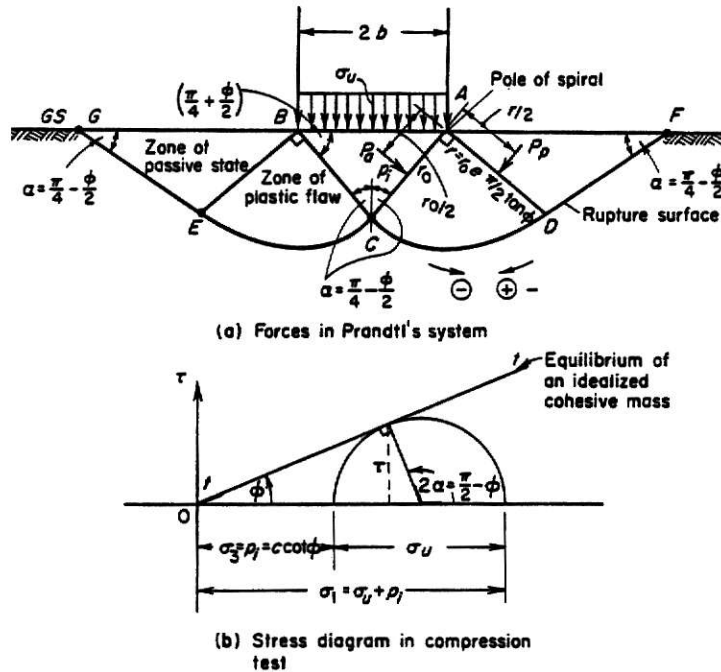
Figure 4.1.a Prandtl's system for a (ϕ-c) soil.

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

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analysis can be made taking into account the weight of the

Figure 4.2. Prandtl-Terzaghi's system.

soil within ADB or disregarding it. If the weight mentioned is disregarded, the equilibrium of the strip footing requires that

$$Q_{crit} = 2P_p + B \ c \ \tan\phi. \tag{4.3}$$

where

Pp = passive pressure

B = footing width

ϕ = friction angle

c = cohesion.

This equation represents the solution of the problem where Pp is known.

The passive earth pressure required to cause a slip on DEF can be divided into two parts, p₁ and p₂. The force p₁ is the resistance due to the weight of BDEF and its point of

application is the lower third point of BD.   The force p_2 is composed of two parts.   One is due to cohesion P_C and the other is due to surcharge γz, P_q. P_C and P_q are uniformly distributed,   then its point of application is located at the middle of BD.

Now, the equation 4.3 is arranged as

$$Q_{crit} = 2(p_1 + p_C + p_q) + BC \tan \phi$$

where the passive pressures are given by

$$N_C = \frac{2p_C}{BC} + \tan \phi$$

$$N_q = \frac{2p_q}{\gamma z B}$$

$$N = \frac{4p_1}{\gamma B^2}$$

plotting that values into the above equation is obtained.

$$Q_{crit} = B(cN_C + \gamma z N_q + 1/2 \gamma B N_\gamma) \tag{4.4}$$

the values n_C, N_q and N_γ are called the "bearing capacity factors and they are dimensionless quantities which depend only on the value of ϕ.   These values are shown in Table 4.1. where

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

$$N_q = \frac{e^{2(3\pi/4 - \phi/2)\tan\phi}}{2 \cos^2(45 + \frac{\phi}{2})} \tag{4.6}$$

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$$N_\gamma = \frac{1}{2} \left(\frac{Kp\gamma}{\cos^2\phi} - 1 \right) \tan\phi$$

$Kp\gamma$ = passive pressure coefficient.

Values of N_γ for ϕ of 34 and 48° are original Terzaghi values and used to back-compute $K_{p\gamma}$ for forward computations of N_γ by author

ϕ, deg	N_c	N_q	N_γ	$K_{p\gamma}$
0	5.7	1.0	0.0	10.8
5	7.3	1.6	0.5	12.2
10	9.6	2.7	1.2	14.7
15	12.9	4.4	2.5	18.6
20	17.7	7.4	5.0	25.0
25	25.1	12.7	9.7	35.0
30	37.2	22.5	19.7	52.0
34	52.6	36.5	36.0	
35	57.8	41.4	42.4	82.0
40	95.7	81.3	100.4	141.0
45	172.3	173.3	297.5	298.0
48	258.3	287.9	780.1	
50	347.5	415.1	1153.2	800.0

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 AGI.

The wedge ABD is pushed downward by the load footing and

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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

$$\text{Square } q_u = 1.3 \, N_c + \gamma D_f + 0.4 \, \gamma B N_\gamma \tag{4.8}$$

$$\text{Round } q_u = 1.3 C N_c + \gamma D_f + 0.3 \, \gamma B N_\gamma \tag{4.9}$$

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:

$$q = C N_c F_{cs} F_{cd} F_{ci} + \gamma D_f N_\gamma F_{qd} F_{qi} F_{qs}$$
$$+ 1/2 \, \gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i} \tag{4.10}$$

where

c = choesion

γD_f = q = effective stress at the level of the bottom of foundation

γ = unit weight of the soil

B = width of foundation (diameter for a circular footing)

F_{cs}, F_{qs}, $F_{\gamma s}$ = shape factors

F_{cd}, F_{qd}, $F_{\gamma d}$ = depth factors

F_{ci}, F_{qi}, $F_{\gamma i}$ = 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 ϕ by Terzaghi is considered equal to $45 + \dfrac{\phi}{2}$ by Meyerhof and those factors become:

$$Nq = e^{\pi \tan\phi} \tan^2 (45^\circ + \frac{\phi}{2}) \qquad (4.11)$$

$$N_c = (Nq - 1) \cot\phi \qquad (4.12)$$

$$N = (Nq - 1) \tan (1.4\phi) \qquad (4.13)$$

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

ϕ, deg	N_c	N_q	N_γ	2 tan $(1 - \sin\phi)^2$	N_γ
0	5.14	1.0	0.19	0	0
5	6.50	1.6	0.24	0.15	0.1
10	8.30	2.5	0.30	0.24	0.4
15	11.00	3.9	0.36	0.29	1.1
20	14.80	6.4	0.43	0.32	2.9
25	20.70	10.7	0.51	0.31	5.8
30	30.10	18.4	0.61	0.29	15.7
35	46.10	33.3	0.72	0.25	37.1
40	75.30	64.2	0.85	0.21	93.7
45	133.90	134.9	1.01	0.17	262.7
50	266.90	319.0	1.20	0.13	873.7

Shape factors can be obtained as

$$F_{cs} = 1 + \left(\frac{B}{L}\right) \left(\frac{N_q}{N_c}\right) \tag{4.14}$$

$$F_{qs} = 1 + \left(\frac{B}{L}\right) \tan \tag{4.15}$$

$$F_{\gamma s} = 1 - 0,4 \left(\frac{B}{L}\right) \tag{4.16}$$

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

Depth factors can be obtained as:

$$F_{cd} = 1 + 0.4 \left(\frac{D_f}{B}\right) \quad \text{for } D_f/B < 1 \tag{4.17}$$

$$F_{qd} = 1 + 2 \tan \quad (1 - \sin)^2 \frac{D_f}{B} \quad \text{for } D_f/B < 1 \tag{4.18}$$

$$F_{\gamma d} = 1 \tag{4.19}$$

for $D_f/B > 1$

$$F_{cd} = 1 + 0.4 \tan^{-1} \left(\frac{D_f}{B}\right) \tag{4.20}$$

$$F_{qd} = 1 + 2 \tan \phi \ (1 - \sin \phi)^2 \tan^{-1} \left(\frac{D_f}{B}\right) \tag{4.21}$$

$$F_{\gamma d} = 1 \tag{4.22}$$

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

And, the inclination factors can be obtained as

$$F_{ci} = F_{qi} = \left(1 - \frac{B^\circ}{90^\circ}\right)^2 \tag{4.23}$$

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$$F_{\gamma i} = (1 - \frac{B^o}{\phi})^2 \qquad\qquad (4.24)$$

where B^o 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:

$$q = D_1 \gamma + D_2 (\gamma_{sat} - \gamma_w) \qquad\qquad (4.25)$$

where

γ_{sat} = saturated unit weight of the soil

γ_w = unit weight of water

The value of γ in the last term of the bearing capacity equation must be changed to

$$\gamma' = \gamma_{sat} - \gamma_w = \gamma_{submerged} \qquad\qquad (4.26)$$

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

The factor q remains the same, that is

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$$q = \gamma D_f \qquad (4.27)$$

The factor γ in the last term of the equation must be changed to

$$\bar{\gamma} = \gamma' + \frac{d}{B}(\gamma - \gamma') \qquad (4.28)$$

c) Water table at $d \geqslant B$.
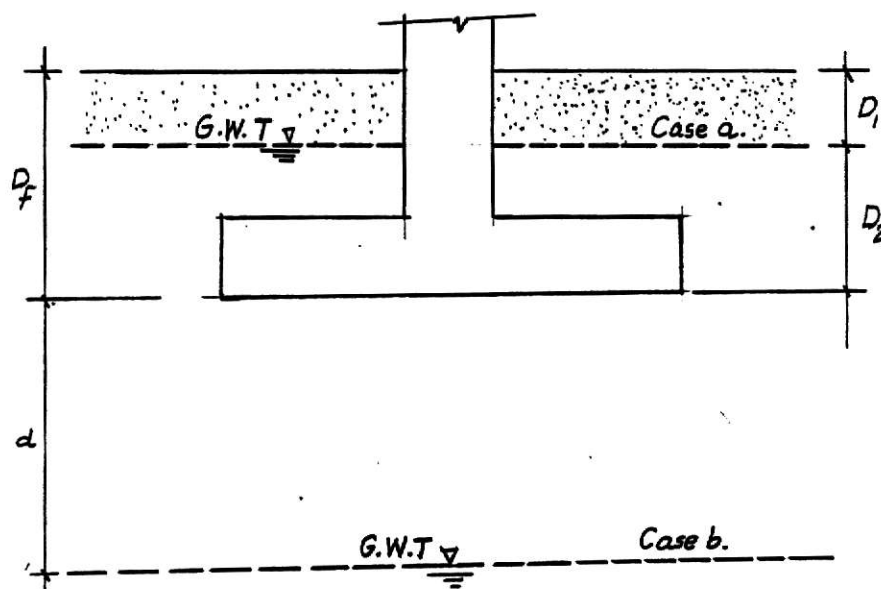
The factors remain the same

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.

$$q = P(\frac{1}{A} \quad \frac{e_x \, x}{I_y} \quad \frac{e_y \, y}{I_x}) \qquad (4.29)$$

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 presure 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

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L', B' to evaluate the shape factors and to replace B' by B in the γBN_γ terms. To obtain the depth factors, the actual value of B must be used.

The ultimate load is obtained as

$$P_{ult} = q_u \, B' \, L' \qquad\qquad (4.30)$$

$$V = q_{ult}(B'L')/F$$
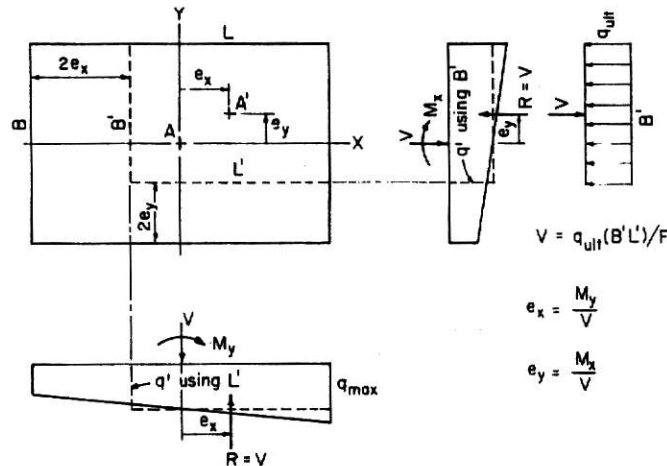
$$e_x = \frac{M_y}{V}$$

$$e_y = \frac{M_x}{V}$$

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

f.2 Foundations in Layered Soils

The utlimate 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 anisrotopy of the soil by

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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):

$$q_u = C N_c F_{cs} F_{cd} F_{ci} + \gamma D_f N_q F_{qs} F_{qd} F_{qi}$$
$$+ 1/2 \gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}$$

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

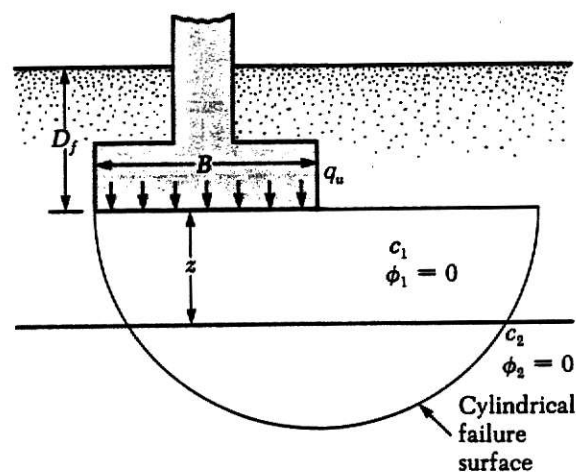$$q_u = C_1 N_c F_{cs} F_{cd} + \gamma D_f \qquad (4.31)$$
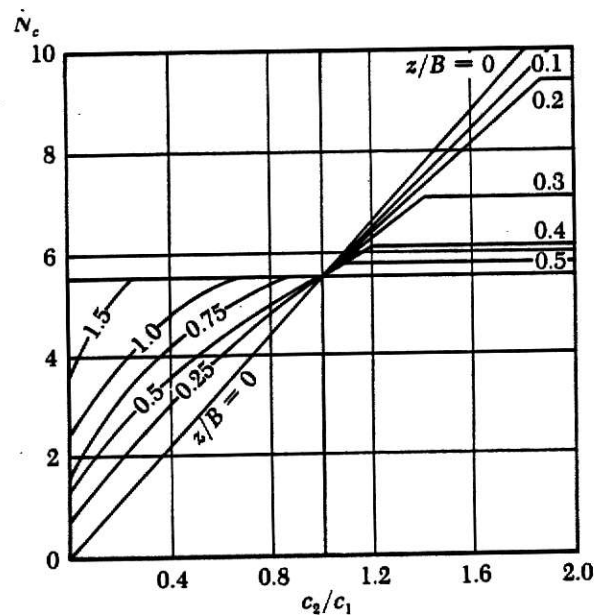
If $\phi_1 = 0$.

Figure 4.5.

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Figure 4.6.   Bearing capacity on layered clay soils -- $\phi = 0$ (redrawn after Reddy and Srinevasan, 1967).

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 amont 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.

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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 4.4   Values of customary safety factors

Failure mode	Foundation type	F
Shear	Earthworks Dams, fills, etc.	1.2-1.6

Shear	Retaining structure
	    Walls
	    Sheetpiling, coffer-
	    dams
	    Braced excava-
	    tions
	    (temporary)
		
		1.5-2.0
		1.2-1.6
		
		1.2-1.5

Shear	Footings
	    Spread
	    Mat
	    Uplift
		
		2-3
		1.7-2.5
		1.7-2.5

Seepage	Uplife, Heaving Piping	1.5-2.5 3-5

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Table 4.4a Foundation loads

Load	Includes
Dead load (DL)	Weight of structure and all permanently attached material
Live load (LL)	Any load not permanently attached to the structure, but to which the structure may be subjected
Snow load (S)	Acts on roofs; value to be used generally stipulated by codes
Wind loads (W)	Acts on exposed parts of structure
Earthquake (E)	A lateral force (usually) which acts on the structure
Hydrostatic (HS)	Any loads due to water pressure and may be either (+) or (-)
Earth pressure (EP)	Any loads due to earth pressures-- commonly lateral but may be in other directions

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

$$(K_{sf}) \quad q_a = 720(N-3) (\frac{B + 1}{2B})^2 \ w^1 \ K_d \qquad\qquad (4.32)$$

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)

$$K_d = 1 + \frac{D}{B} \quad 2.0$$

D = depth of foundation.

Meyerhof suggested less conservative values given as

$$(K_{sf}) \quad q_a = \frac{N}{2.5} \ w^1 \ K_d \ \text{for B} \quad 4' \qquad\qquad (4.33)$$

$$(K_{sf}) \quad q_a = \frac{N}{4} (\frac{B + 1}{B})^2 \ w' \ K_d \ \text{for B} \quad 4' \qquad\qquad (4.34)$$

Gibbs and Holtz proposed a set of curves to obtain the relative density of a soil given the standard penetration

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resistance "N" and the vertical effective stress. These
curves are shown in Fig. 4.7. The average effective pressure
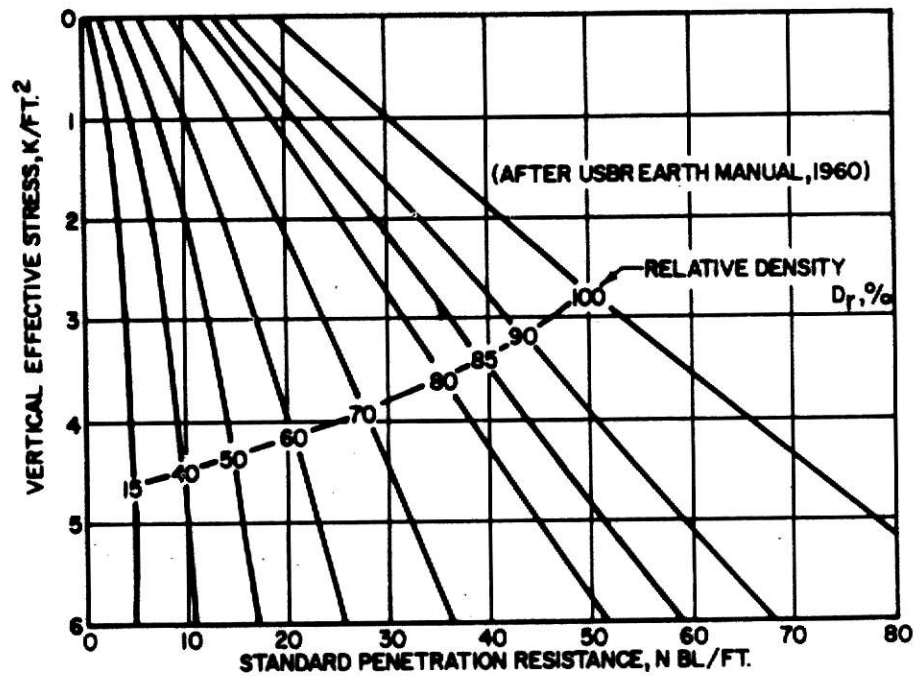should be computed at the depth $D_f + \dfrac{B}{2}$

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 (γ_d) is given and the soil is classified according to the unified system. The value of the friction angle $(\emptyset)$ obtained above can be used in the Terzaghi equations to obtain the allowable bearing capacity.
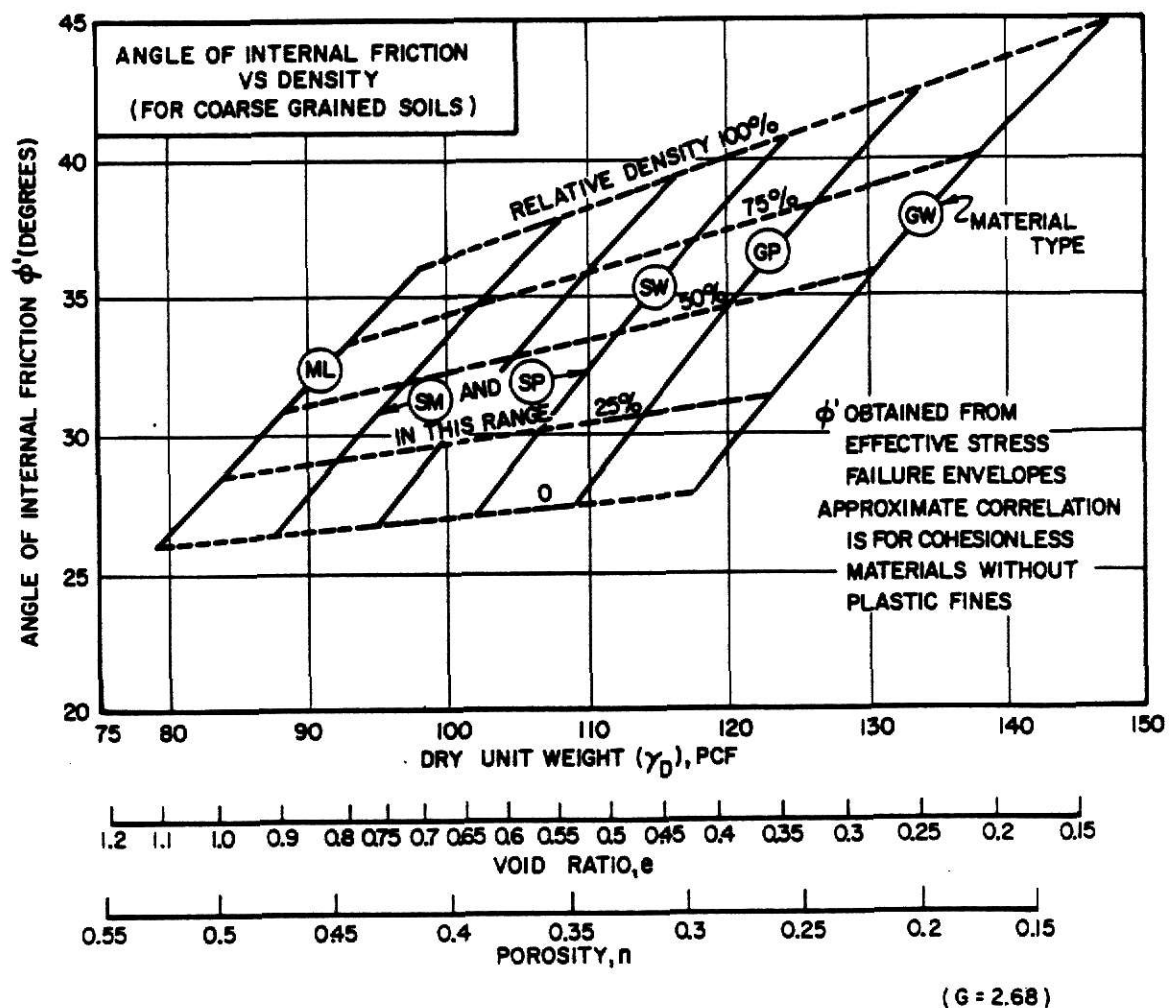
Figure 4.8. Coorelations of Strength Characteristics for Granular Soils.

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

$$S = S_e + S_c + S_s \tag{5.1}$$

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 $10^{-6} \frac{m}{sec}$ 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 expulsed. 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

$$S_s = C \; H_{ts} \log \frac{t}{t_p} \tag{5.2}$$

where $C = \dfrac{\Delta H_t / H_t}{\log t_2 - \log t_1}$ coefficient of secondary consol-

idation (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

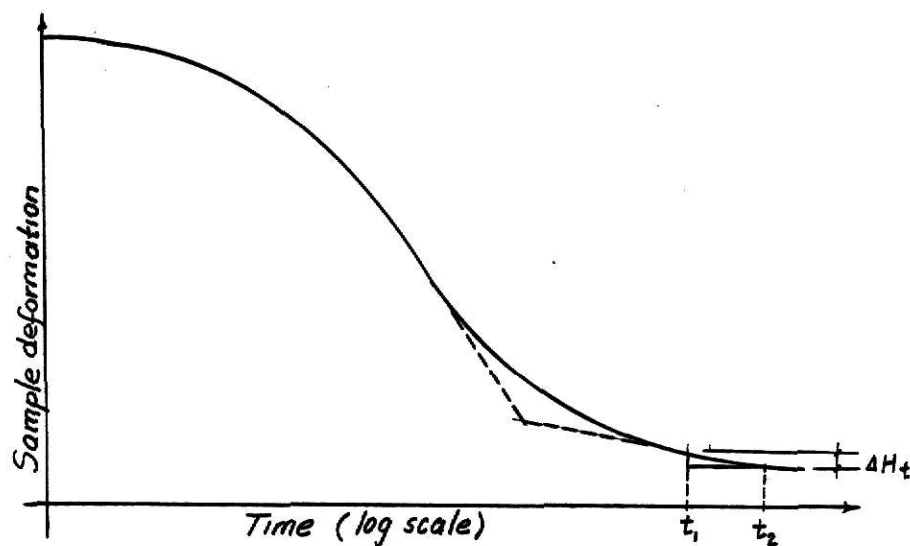ΔH_t = change in thickness of clay layer during secondary consolidation

Figure 5.1.

b. Elastic Settlement

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

$$Se_1 = q \, B \, \frac{1 - \mu^2}{E_s} \, I_w \qquad (5.3)$$

where

S_{e_1} = 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, μ = 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

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$$S_e = q\ B'\ \frac{1 - \mu^2}{E_s}\ (F_1 + \frac{1 - 2\mu}{1 - \mu}\ F_2) \qquad\qquad (5.4)$$

where

$$F_1 = \frac{1}{\pi}\ [M\ Ln\ \frac{(1 + \sqrt{M^2 + 1})(\sqrt{M^2 + N^2})}{M(1 + \sqrt{M^2 + N^2 + 1})}$$

$$+\ Ln\ \frac{(M + \sqrt{M^2 + 1})\sqrt{1 + N^2}}{M + \sqrt{M^2 + N^2 + 1}}]$$

$$F_2 = \frac{N}{2\pi}\ tan^{-1}\ (\frac{M}{N\sqrt{M^2 + N^2 + 1}})\ ,\ tan^{-1}\ in\ rads$$

$$M = \frac{L'}{B'}\qquad N = \frac{H}{B'}\qquad B' = \frac{B}{2}\ for\ center$$

$$B' = B\ for\ center$$

$$L' = \frac{L}{2}\ for\ center$$

$$L' = L\ for\ corner.$$

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

$$S_e = S_{1\ or\ 2}\ (F_3)$$

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

$$K_r = \frac{1}{6} \left(\frac{1 - \mu_s^2}{1 - \mu_F^2}\right) \left(\frac{E_f}{E_s}\right) \left(\frac{T}{b}\right)^3$$

$K_r = \infty$ means a perfectly rigid footing

$K_r = 0$ means a perfectly flexible footing

where

μ_s, μ_F :   Poisson's ratio of soil and footing, respectively

E_s, E_F :   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 , for various-shaped members and for flexible and rigid footings

| Shape | Flexible | | | Rigid |
	Center	Corner	Average	I_w
Circle	1.00	0.64 (edge)	0.85	0.88
Square	1.12	0.56	0.95	0.82
Rectangle:				
L/B = 0.2				
0.5				
1.5	1.36	0.68	1.15	1.06
2.0	1.53	0.77	1.30	1.20
5.0	2.10	1.05	1.83	1.70
10.0	2.54	1.27	2.25	2.10
100.0	4.01	2.00	3.69	3.40
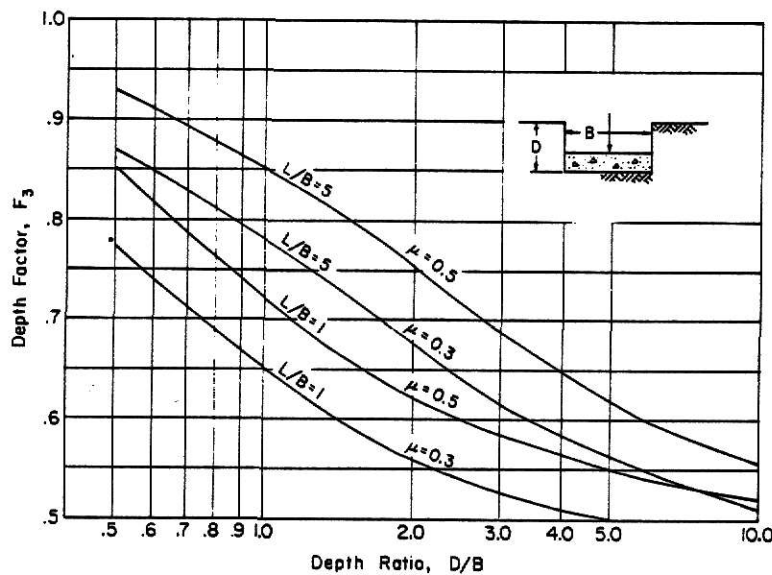
Figure 5.2. Influence factor for footing at a depth D. Use actual footing width and depth dimension for D/B ratio.

The model used by Borowicka was footings reating 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
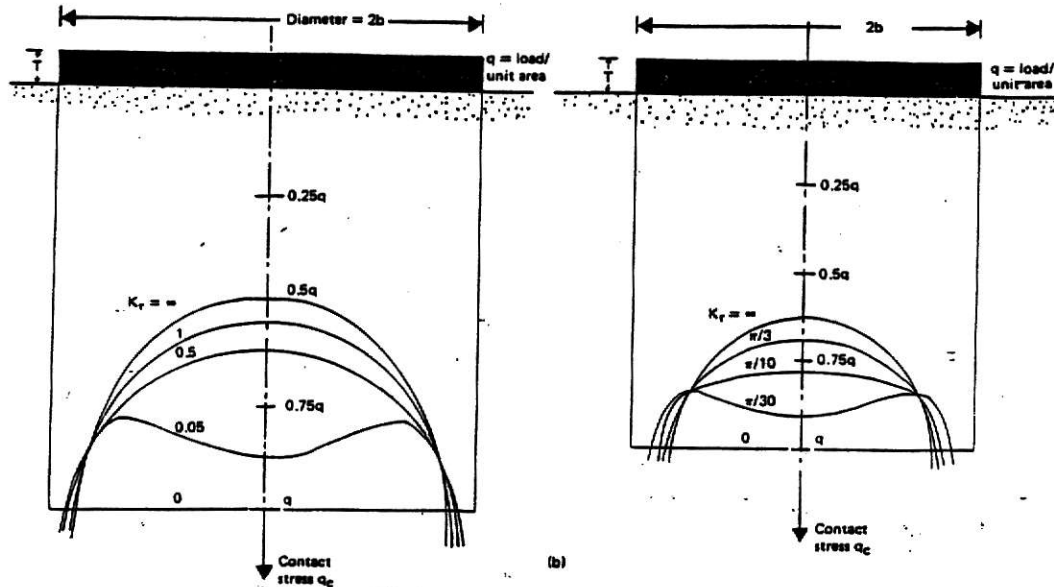

Figure 5.3. Contact stress over rigid foundations resting on an elastic medium. (a) Circular foundation. (b) Strip foundation.

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 caly
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
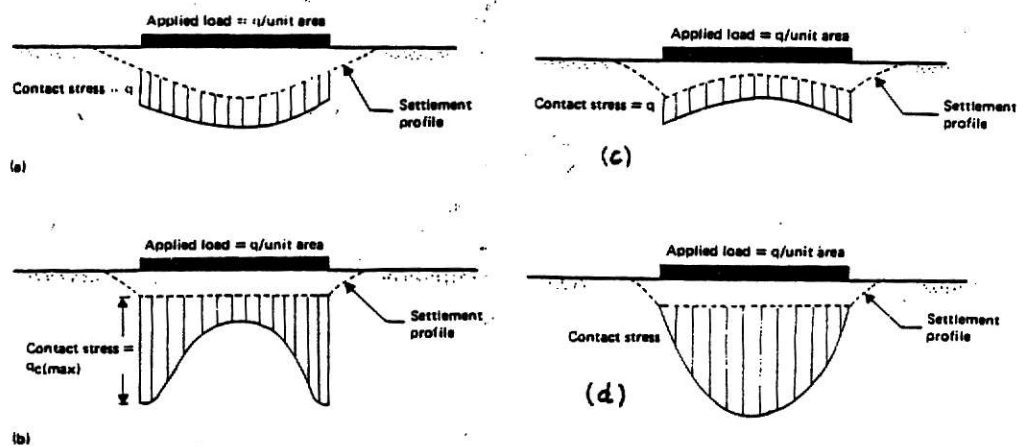will behave as shown in Fig. 5.4c.

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

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.
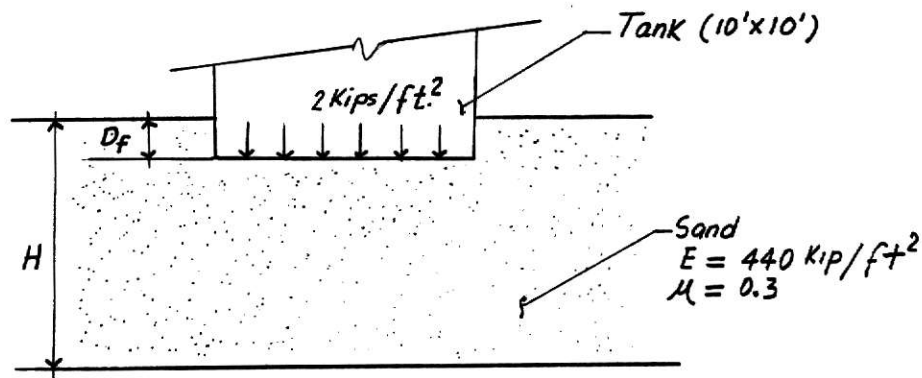
a) $D_f = 0$ $H = $

b) $D_f = 4.5'$ $H = 30'$

Figure 5.5

Solution

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

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

$$I_{w_{average}} = 0.95 \text{ from Table 5.1}$$

$$Se_1 = 2(10) \ \frac{1 - \overline{0.3}^2}{440} \ 0.95 = 0.039 \text{ ft} = 0.47"$$

b) $D_f = 4.5'$ $H = 30'$

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Using Eq. 5.4

$$Se_2 = q\ B'\ \frac{1-^2}{E_s}\ (F_1 + \frac{1-^2}{1-}\ F_2)$$

$$B' = \frac{B}{2}\ (center),\ B' = \frac{10'}{2} = 5',\ M = \frac{L'}{B'} = \frac{5'}{5'} = 1$$

$$L' = \frac{L}{2}\ (center),\ L' = \frac{10}{2} = 5'\ ,\ N = \frac{H}{B'} = \frac{30}{5} = 6$$

$$F_1 = \frac{1}{}\ [\ 1\ Ln\ \frac{(1 + \sqrt{1^2 + 1})(\sqrt{1^2 + 6^2})}{1(1 + \sqrt{1^2 + 6^2 + 1})}$$

$$+\ Ln\ \frac{(1 + \sqrt{1^2 + 1})(\sqrt{1 + 6^2})}{1 + \sqrt{1 + 6^2 + 1}}]$$

$$F_1 = 0.46$$

$$F_2 = \frac{6}{2}\ \tan^{-1}\ (\frac{1}{6\sqrt{1 + 6^2 + 1}}) =$$

$$F_2 = \frac{6}{2}\ \times\ 0.027\ rad = 0.026$$

Then

$$Se_2\ center = 2(5)(2)\ \frac{1 - 0.3^2}{440}\ (0.46 + \frac{1 - 2(0.3)}{1-0.3}\ 0.026)$$

$$= 0.0196\quad (\text{two times } F_1 \text{ and } F_2 \text{ because of four corner distributions})$$

and

$$Se_3\ center = 0.24"\quad F_3 = 0.24"\ \times\ 0.77 = 0.19"$$

$$F_3\ = 0.77\qquad D/B = 0.45$$
(from Fig. 5.2) $L/B = 1,\quad = 0.3$

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'
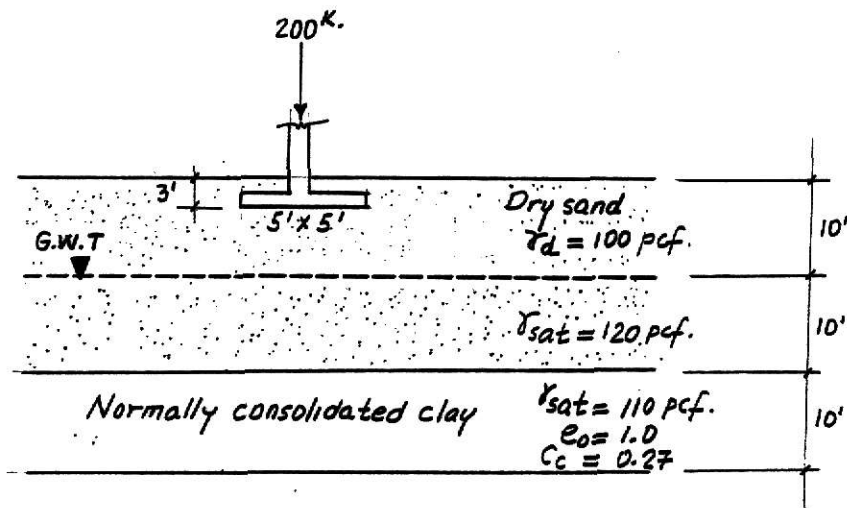

Figure 5.6

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 (ΔP) 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:
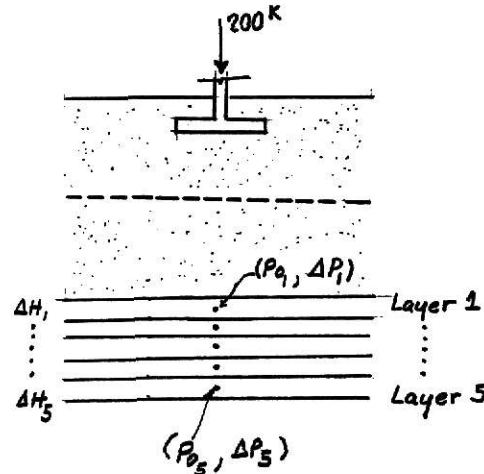
Calculation of the effective stress:

$$Po_1 = 100 \times 10 + (120 - 62.4) \, 10 + (110 - 62.4) \, 1$$

$$= 1623.6 \text{ Psf.}$$

$$Po_2 = 100 \times 10 + (120 - 62.4) \, 10 + (110 - 62.4) \, 3$$

$$= 1718.8 \text{ Psf.}$$

$$Po_3 = 110 \times 10 + (120 - 62.4) \, 10 + (110 - 62.4) \, 5$$

$$= 1814 \text{ Psf.}$$

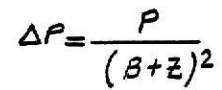$$Po_4 = 110 \times 10 + (120 - 62.4) \, 10 + (110 - 62.4) \, 7$$
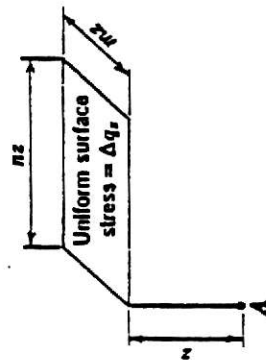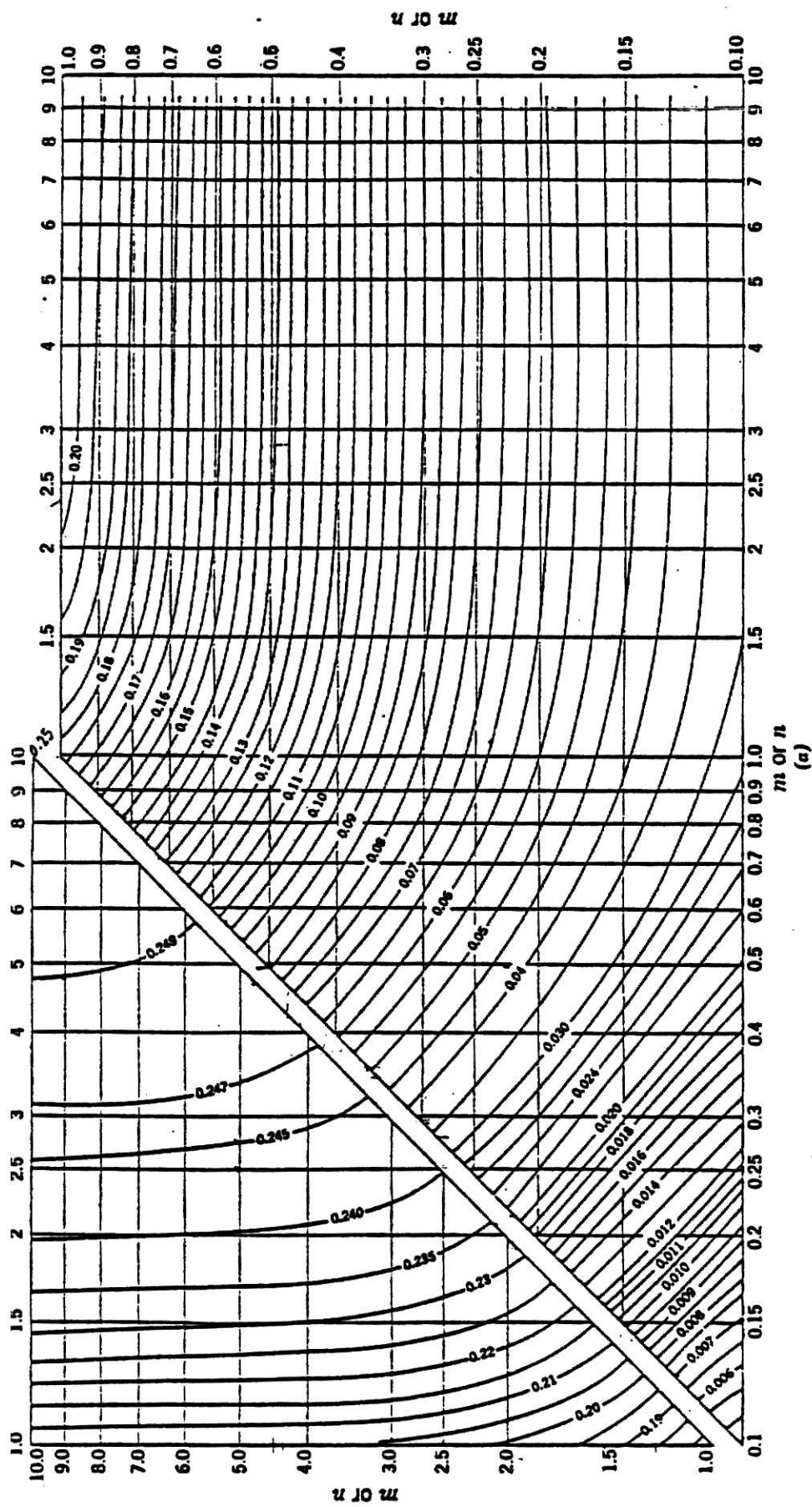
$$= 1909.2 \text{ Psf.}$$

$$Po_5 = 110 \times 10 + (120 - 62.4) \, 10 + (110 - 62.4) \, 9$$

$$= 2004.4 \text{ Psf.}$$

Calculation of the induced stress (P).

There are several methods available, the 2:1 method will
be used for simplicity.

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$$\Delta P = \frac{P}{(B+Z)^2}$$

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

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

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

$$P_3 = \frac{200000}{(5 + 24)^2} = 237.81 \text{ Psf}$$

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

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 $=$

Fig 5.7 (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, $\Delta\sigma_z = \Delta q_s \times f(m, n)$. (From Newmark, 1942)

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If the induced stress are obtained by Newmark chart (see Fig. 5.7) , is obtained

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

$$\triangle P = \left(\frac{200000}{5 \times 5}\right) 4 \, I_f$$
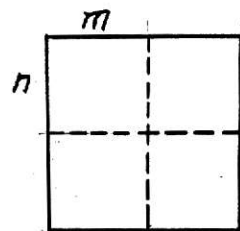
The settlement obtained by using the Newmark's induced stresses is equal to S_{total} = 0.060' = 0.72"

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

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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   pro-

vides   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

Criterion	Isolated foundations	Rafts
Angular distortion (cracking)	1.300	
Greatest differential settlement:		
Clays	1 1/4 (1 1/2)	
Sands	1 1/4(1)	
Maximum settlement:		
Clays	3(2 1/2)	3-5(2 1/2-4)
Sands	2(1 1/2)	2-3)1 1/2-2 1/2)

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 = length between two
adjacent points under consideration. H = height of
wall above foundation. *

Structure	On sand or hard clay	On plastic clay	Average max. settlement, cm		
Crane runway	0.003	0.003			
Steel and concrete frames	0.0010	0.0013	10		
End rows of brick-clad frame	0.007	0.001	15		
Where strain does occur	0.005	0.005			
Multistory brick wall L/H to 3	0.005	0.004	8 10	L/H L/H	2.5 1.5
Multistory brick wall L/H over 5	0.003	0.007			
One-story mill buildings	0.001	0.001			
Smokestacks, water towers, Ring foundations	0.004	0.004	30		

Structures on permafrost		
Reinforced concrete	0.002-0.0015	15 at 4 cm/year
Masonary, precast concrete	0.003-0.002	20 at 6 cm/year
Steel frames	0.004-0.0025	25 at 8 cm/year
Timber	0.007-0.005	40 at 12 cm/year

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
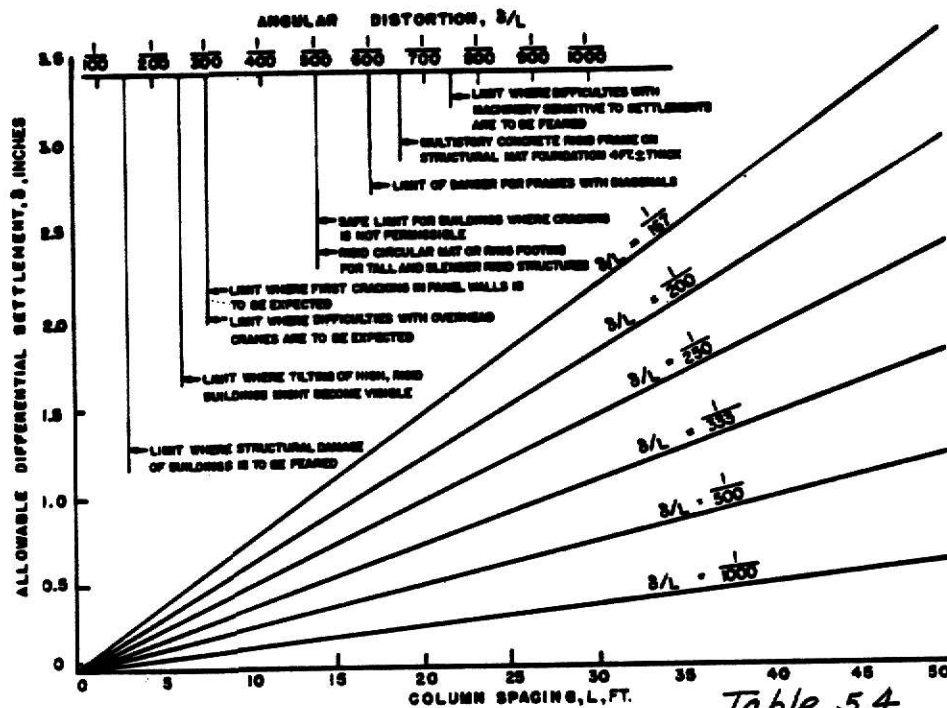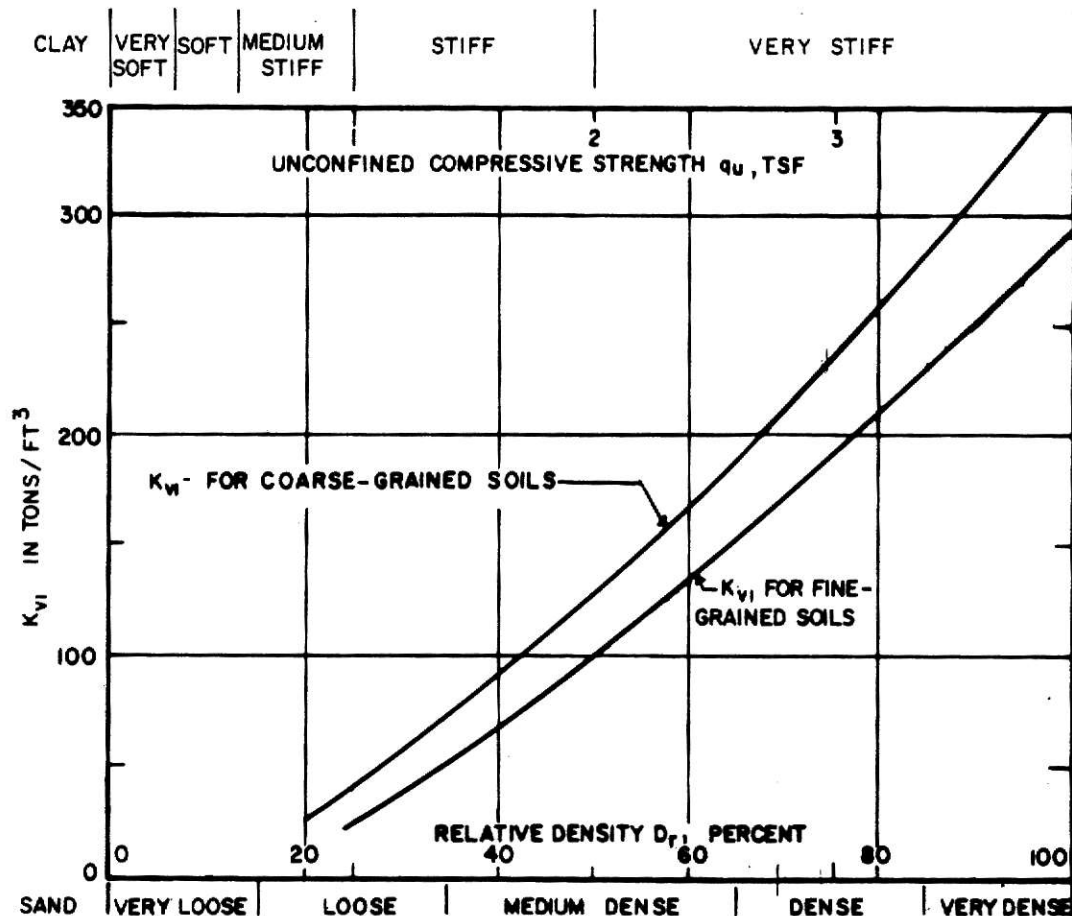obtained   from   standard   penetration data   as   explained   in
Chapter IV.

Table 5.4

CLAY | VERY SOFT | SOFT | MEDIUM STIFF | STIFF | VERY STIFF

UNCONFINED COMPRESSIVE STRENGTH q_u, TSF

K_{VI} IN TONS/FT^3

K_{VI} - FOR COARSE-GRAINED SOILS

K_{VI} FOR FINE-GRAINED SOILS

RELATIVE DENSITY D_r, PERCENT

SAND | VERY LOOSE | LOOSE | MEDIUM DENSE | DENSE | VERY DENSE

DEFINITIONS

ΔH_i = IMMEDIATE SETTLEMENT OF FOOTING
q = FOOTING UNIT LOAD IN tsf
B = FOOTING WIDTH

D = DEPTH OF FOOTING BELOW GROUND SURFACE
K_{VI} = MODULUS OF VERTICAL SUBGRADE REACTION

COARSE-GRAINED SOILS

(MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH)
SHALLOW FOOTINGS $D \leq B$
FOR $B \leq 20$ FT:

$$\Delta H_i = \frac{4\,q\,B^2}{K_{VI}\,(B+1)^2}$$

FOR $B \geq 40$ FT:

$$\Delta H_i = \frac{2\,q\,B^2}{K_{VI}\,(B+1)^2}$$

INTERPOLATE FOR INTERMEDIATE VALUES OF B

DEEP FOUNDATION $D \geq 5B$
FOR $B \leq 20$ FT:

$$\Delta H_i = \frac{2\,q\,B^2}{K_{VI}\,(B+1)^2}$$

NOTES: 1. NONPLASTIC SILT IS ANALYZED AS COARSE-GRAINED SOIL WITH MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH.
2. VALUES OF K_{VI} SHOWN FOR COARSE-GRAINED SOILS APPLY TO DRY OR MOIST MATERIAL WITH THE GROUNDWATER LEVEL AT A DEPTH OF AT LEAST 1.5B BELOW BASE OF FOOTING. IF GROUNDWATER IS AT BASE OF FOOTING, USE $K_{VI}/2$ IN COMPUTING SETTLEMENT
3. FOR CONTINUOUS FOOTINGS MULTIPLY THE SETTLEMENT COMPUTED FOR WIDTH "B" BY 2.

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 volometric 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 z_f.   That is, $m = 2z_f$ 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 z_f.
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.
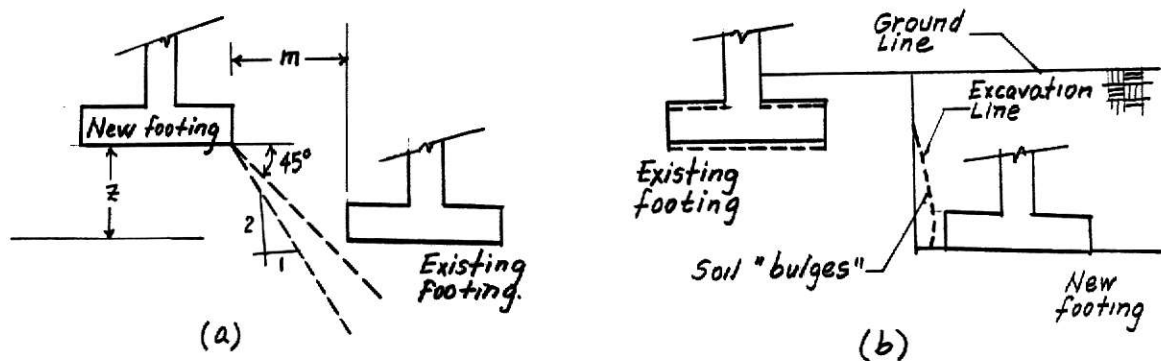

Figure 6.1. Location considerations for spread footings. (a) An approximation for spacing footings to avoid interference between old and new footings. If "new" footing is in relative position of "existing" footing, interchange words "exiting" and "new". Make m > z . (b) Possible settlement of "existing" footing because of loss of lateral support of soil wedge beneath existing footing.
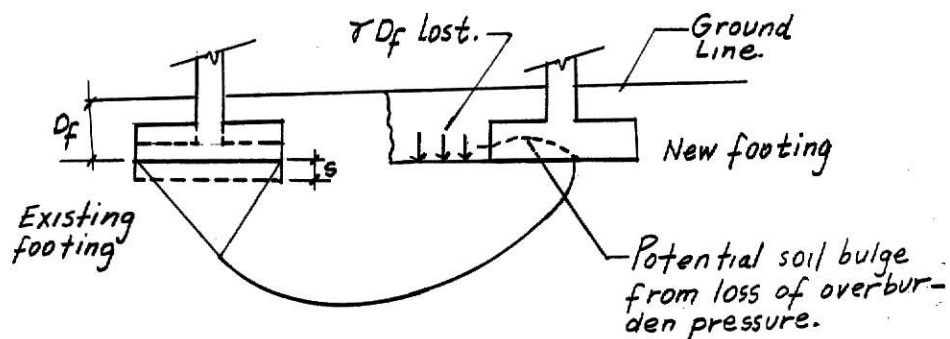
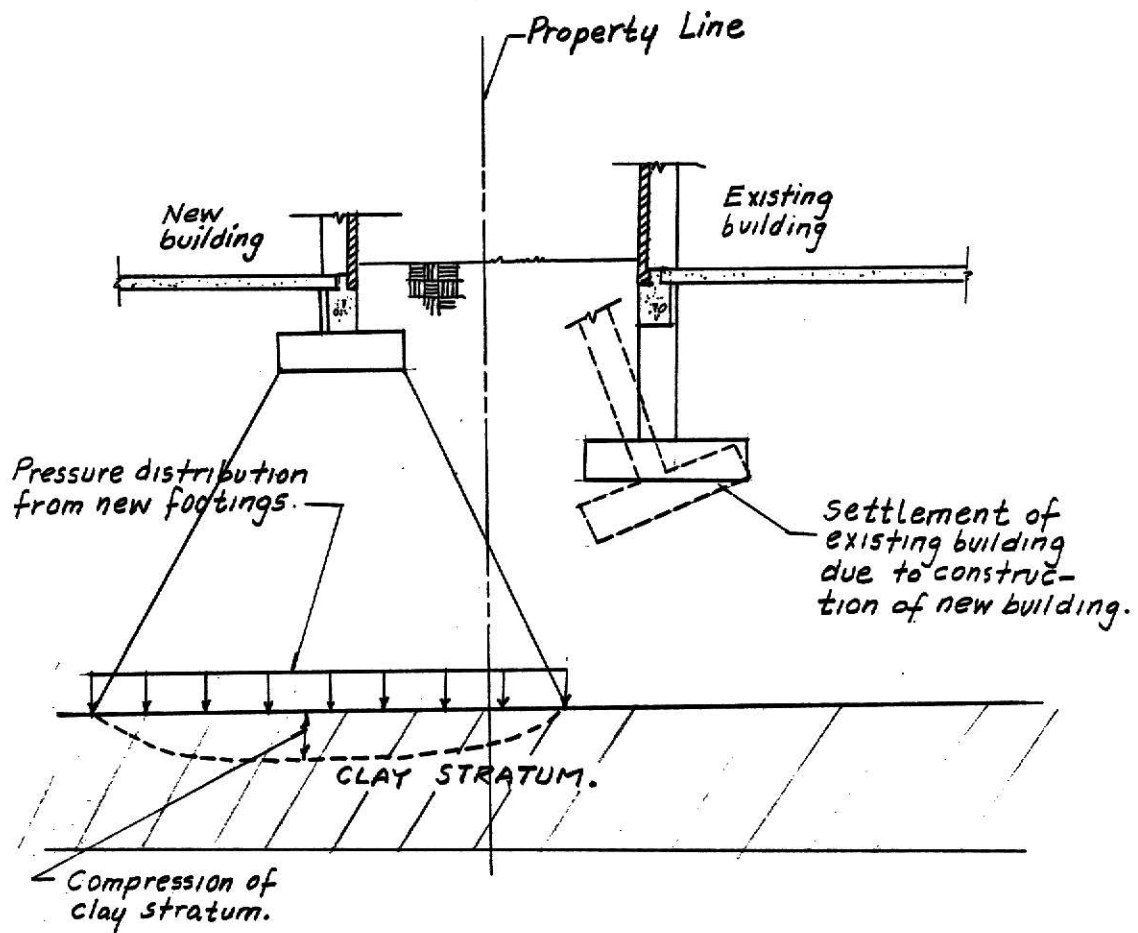Figure 6.2. Potential settlement due to loss of oberburden pressure.

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Bearing pressure (lower footing)

Bearing pressure (upper footing)

potential tilt of lower footing due to influence of upper footing.

Pressures on and under lower footing due to influence of upper footing.

Figure 6.3. Adjacent Footings

Property Line

New building

Existing building

Pressure distribution from new footings.

Settlement of existing building due to construction of new building.

CLAY STRATUM.

Compression of clay stratum.

Figure 6.4. Influence of Construction of New Foundation on Performance of Existing Construction.

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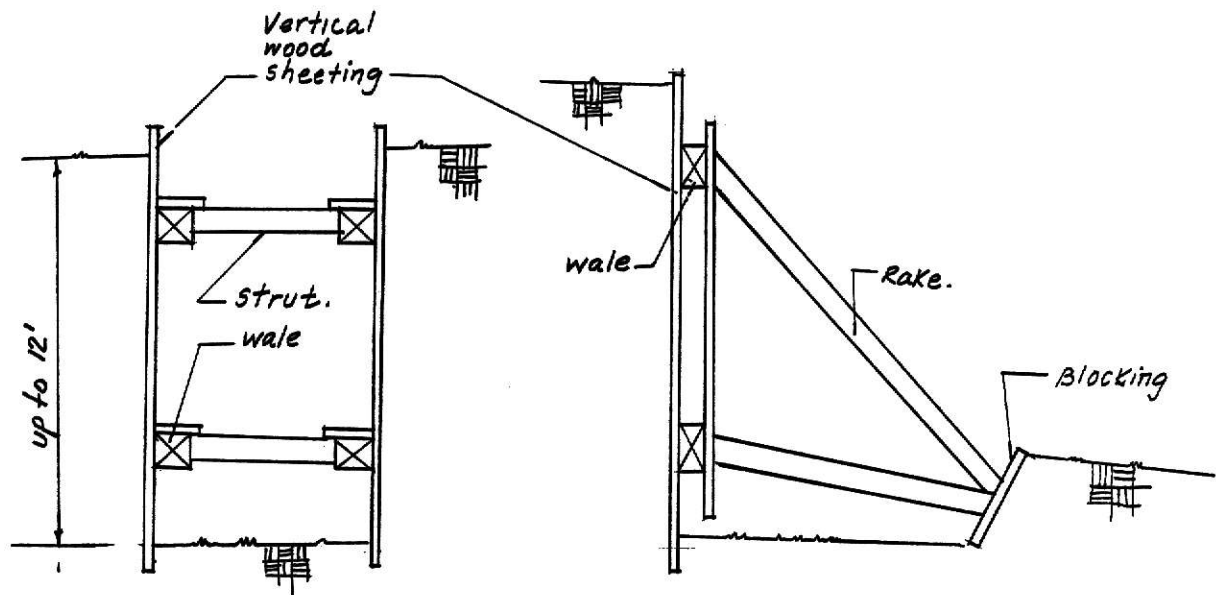
Figure 6.5. Common methods for bracing sides of shallow foundations.
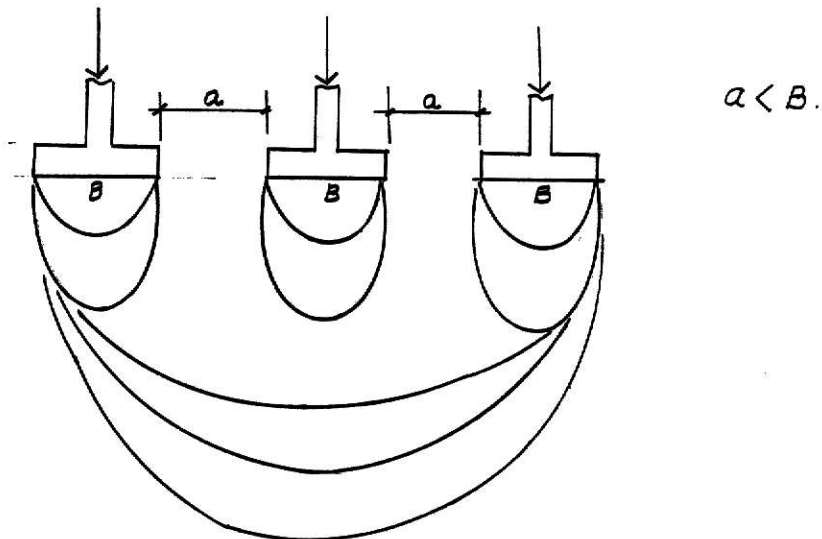
Figure 6.6. Effect of overlapping pressure bulbs.

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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,

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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

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has been periodically revised, the recent major revisions were made in 1963, 1971, 1977 and 1983.

Single Footing   Stepped Footing   Sloped Footing   Wall footing
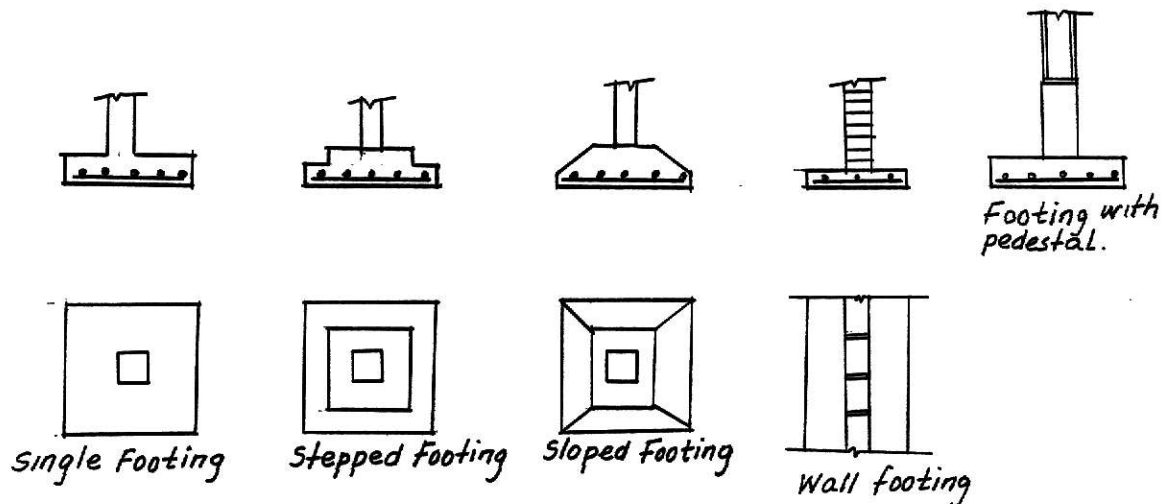
Footing with pedestal.

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 x L or B.

Critical Sections for wide-beam shear.

Critical Sections for punching shear.
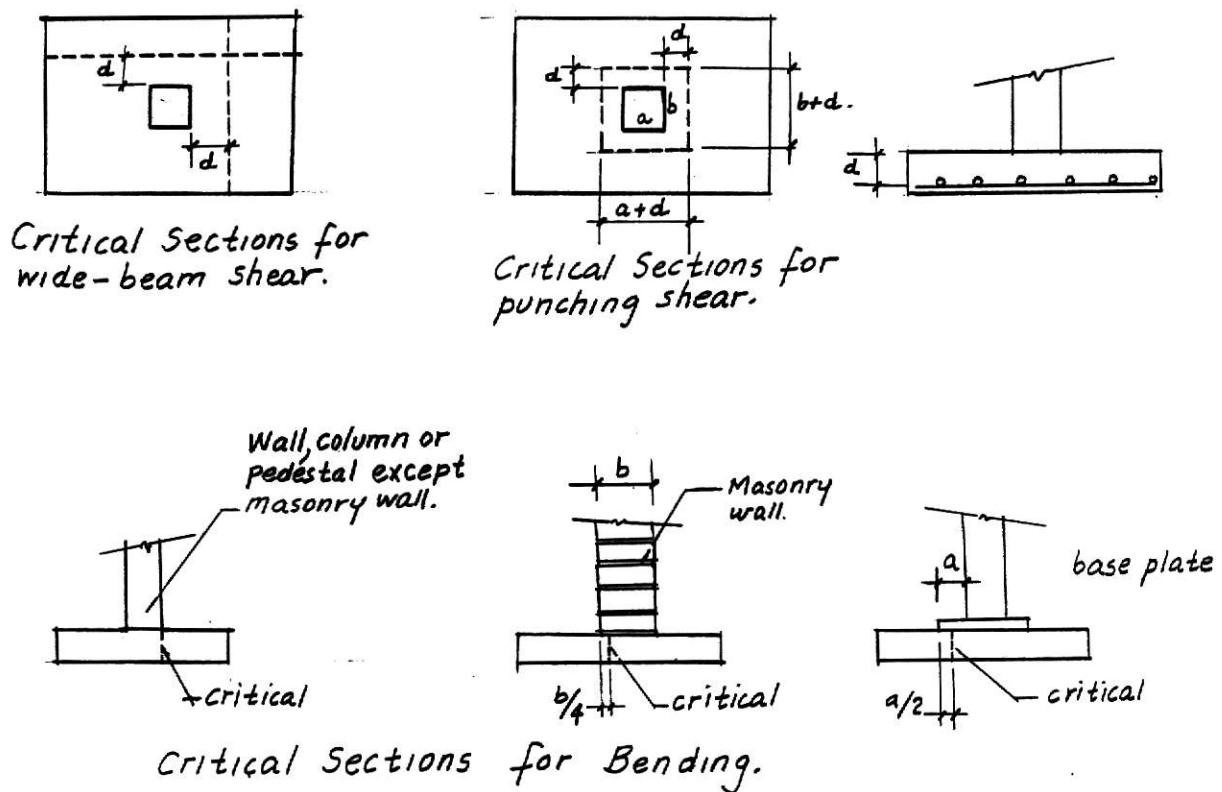
Critical Sections for Bending.

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.

$q_{effective} = q_a - \gamma_o D_f$

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where

$$o = \frac{soil - concrete}{2}$$

The footing dimensions are readjusted to

$$A_{required} = \frac{P_{total}}{q_{effective}}$$

4. Convert the q_a value or the q_e value to an ultimate value q_u

$$q_u = \frac{P_u}{BL}$$

where P_u = 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:

Design factor	ACI Code article	General requirements
Spacing of reinforcement	7.6	Not less than D or 1 in or 1.33 × [max. aggregate size]; not more than 3 × depth of footing or 18 in
Lap splices	12.15.2	Not for bars > No. 11
In tension	12.16	See section in Code
In compression	12.17	See section in Code
Temperature and shrinkage	7.12	$p = 0.002$ for $f_y = 275$ to 345 MPa $= 0.0018\dagger$ for $f_y = 415$ MPa welded-wire fabric
Minimum reinforcement cover	7.7	3 in against earth
Design-methods flexure	10.2	$M_u = \phi A_s f_y (d - a/2)$ $a = A_s f_y / (0.85 f_c' b)$
Maximum reinforcement	10.3.3	$p_b = 0.75 \times$ Eq. (8-3) (in textbook) $p = A_s / bd \le p_b$
Minimum reinforcement	10.5.1	$p \ge 200/f_y$ if footing of variable thickness; for slabs of uniform thickness use shrinkage and temperature percentage
β_1 factor	10.2.7	$\beta_1 = 0.85$ for $f_c' \le 28$ MPa $= 0.85 - 0.05$ for each 7 MPa over 28 MPa
Limits of compression reinforcement	10.9	$0.01 \le A_{st}/A_g \le 0.08$
Modulus of elasticity	8.5	$E = w^{1.5} 44 \sqrt{f_c'}$ MPa $14.1 \le w < 24.4$ kN·m³ $= 4730 \sqrt{f_c'}$ MPa $22 \le w < 23.6$ kN·m³ $= 57\,000 \sqrt{f_c'}$ psi Take $n = E_s/E_c$ to nearest integer > 6
Load factors ϕ	9.3.2	Flexure = 0.90; shear = 0.85; bearing = 0.70; flexure plain concrete = 0.65
Load	9.2	1.4 × dead load; 1.7 × live load
Bearing on concrete	10.16	$q_{brg} \le \psi 0.85 \phi f_c'$ $\psi \le 2$
Shear. wide-beam	11.11 11.3.1.1	$v_u = V_u / bd$ $v_c = 2\phi \sqrt{f_c'}$
Diagonal (punch) tension	11.11.2	$v_u = V_u / bd$ $v_c = (2 + 4/\beta)\phi \sqrt{f_c'} \le 4\phi \sqrt{f_c'}$ $\beta = \dfrac{\text{col. length}}{\text{col. width}}$
Shear reinforcement in footings	11.11.3	
Development length of reinforcement	12.2, 12.6	See equations in text or Code
Grade beams	14.3	
Footings	15.2	General footing considerations
Location of bending moments	15.4.2	See Fig. 8-5
Distribution of reinforcing in rectangular footings	15.4.4	Percent in zone of width $B = 2/[L/B + 1]$
Shear location	15.5	See Fig. 8-4
Transfer of stress at base of column	15.8	At least 4 dowels with total $A_s \ge 0.005 A_g$
Unreinforced pedestals and footings	15.11	$f_c = 0.85 \phi f_c'$ $\phi = 0.70$ $f_t = 5\phi \sqrt{f_c'}$ $\phi = 0.65$
Round columns	15.3	Equivalent square column side, $w' = \sqrt{A_c}$
Minimum edge thickness	15.11.4 15.7	8 in for unreinforced footing; 6 in above reinforcement; 12 in on piles
Maximum tensile stress in unreinforced footings		$f_t \le 5\phi \sqrt{f_c'}$ $\phi = 0.65$
Minimum wall thickness	14.2.7	8 in

† Author recommends using 0.002 for all grades of steel.

Table **7.1** Summary of foundation-member requirements ACI Code.
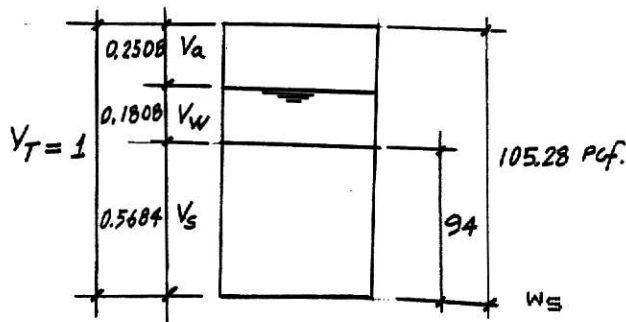
<u>Soil data:</u>

γ_d = 94 Pcf

w_{nat} = 12%

$\phi = 10°$ c = 4 psi c_c = 0.032

Allowable settlement = 1"

A. <u>Physical properties of the soil</u>

$$w = \frac{w_w}{w_s} : \frac{v_w(62.4)}{v_s\,G_s\,(62.4)} \qquad G_s = \frac{w_s}{\frac{w}{1}} = \frac{w_s 1}{v_s\,w}$$

$$0.12 = \frac{v_w}{0.5684\ (2.65)} \qquad v_s = \frac{w_s}{G_s\quad w}$$

$$w_s = v_s\,G_s\quad w$$

$$s = \frac{v_w}{v_v} = \frac{0.1808}{0.4316} \times 100 \qquad v_s = \frac{94}{2.65(62.4)} = 0.5684$$

$$s = 41.92 \qquad v_v = 1 - v_s = 0.4316$$

$$e = \frac{v_v}{v_s} : \frac{0.4316}{0.5684} : \frac{0.7593 \ > \ 0.7}{}$$

Since this value is a little high, we must check the set-tlement.

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γ_{tot} = 94 pcf + 0.1808 (62.4) = 105.28 pcf.

γ_{sat} = 94 + 0.4316 (62.4) = 120.93 pcf.

γ_{sub} = γ_{sat} - γ_w = 120.93 - 62.4 = 58.53 pcf.

for

ϕ = 10° N_c = 9.6 N_q = 2.7 N = 1.2 (from Table 4.1)

B. Bearing Capacity

B.1 $Q_u = cN_c + \gamma D_f N_q + 0.5 B \gamma N_\gamma$

Q_u = 4(144) 9.6 + 105.28 (3) 2.7 + 0.5 B(105.28) 1.2

Q_u = 6382.37 + 63.17 B

$q_a = \dfrac{Q_u}{3}$ 2127.45 + 21.06 B

Assuming

q_a = 2300 psf, for P = L.L + DL = 190 K.

$B^2 = \dfrac{P}{q_a} = \dfrac{190000}{2300}$

B = 9.09'

Trying B = 9.25'

q_a = 2127.45 + 21.06 (9.25) : 2322.25 psf

P_a = 2322.25 (9.25)^2 : 198694.52 > 190000 ok

Trying B = 9¹

q_a = 2316.99

P_a = 2316.99 (9)^2 = 187676.19 < 190000

∴ use 9.25' = B to support P = 190 Kips

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B.2 Checking the settlement

$S_{allow} = 1''$

$$S = \frac{H \, C_c}{1 + e_o} \log \left(1 + \frac{\sigma_i}{\sigma_{body}}\right)$$

Obtained by:

σ_i = 2:1 method

$H = 24''$ $C_c = 0.032$

$\sigma_{body} = \gamma H$

$e_o = 0.7593$
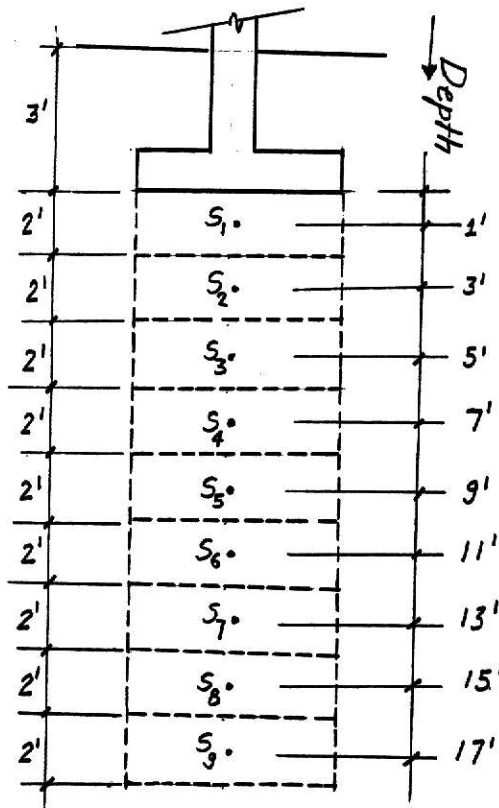
$S_1 = 0.316''$

$S_2 = 0.203''$

$S_3 = 0.138''$

$S_4 = 0.099''$

$S_5 = 0.071''$

$S_6 = 0.052''$

$S_7 = 0.039''$

$S_8 = 0.030''$

$S_9 = 0.023''$

$S_t = 0.971 < 1''$

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 $q_a = 2.32$ ksf

c. Design of the footing

$$L.L = 90K \qquad q_a = 2.32$$

$$DL = 100 \ K$$
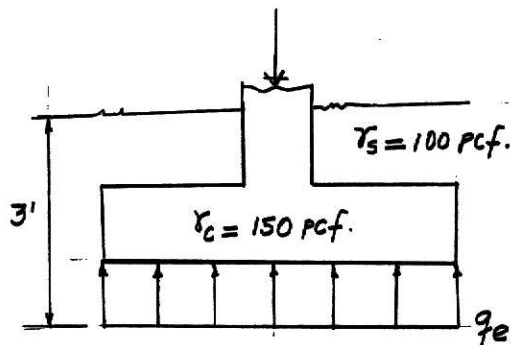
$$\frac{\gamma_s + \gamma_c}{2} = \gamma_o = 125 \ pcf$$

$$q_{effective} = q_a - \gamma_o D_f$$

$$q_{effective} =$$

$$2.32 - (0.125)3$$
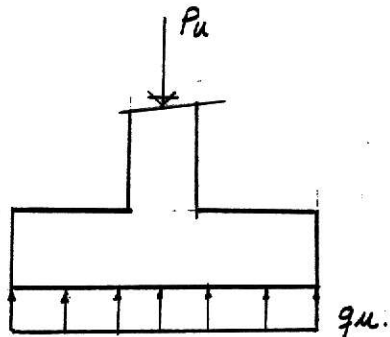
$$= 1.945 \ ksf = q_e$$

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

$$\underline{Use \ B = 10'}$$

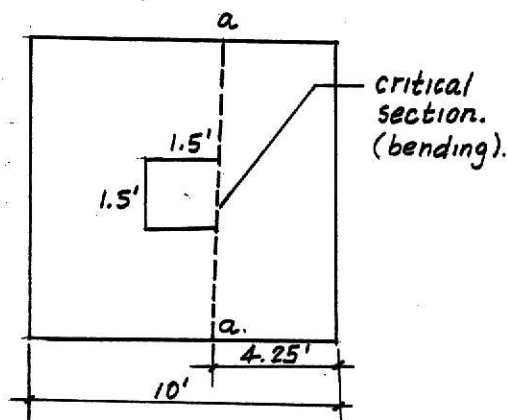## c.1 Design for Bending

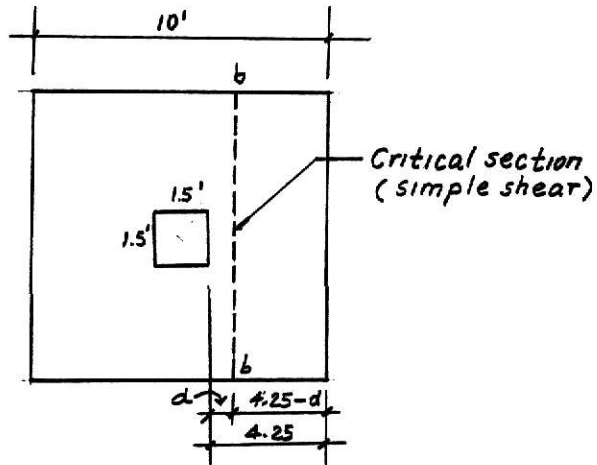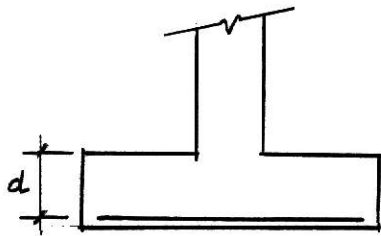$$P_u = 1.4(100) + 1.7(90)$$

$$= 293 \ K$$

$$q_u = \frac{293 \ k}{10 \times 10} \quad 2.93 \ ksf$$

$$M_{a-a} = \frac{2.93(10)(4.25)^2}{2}$$

$$= 264.62 \ k\text{-}foot$$

$$q = \frac{P \ f_y}{f'_c} \qquad f_y = 60 \ ksi$$

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$\phi = 0.9 \quad f'_c : 3 \text{ ksi}$

$\rho = 0.016$

(according to ACI code Art 10.3.3)

$q = \dfrac{0.016(60)}{3}$

$q = 0.32$

$M_u = bd^2 f'_c \, q(1 - 0.59q)$

$M_u = 0.9(10 \times 12 \text{ in}) \, 3 \times$

$(0.032)(1 - 0.59(0.32))d^2$

$M_u = 8.41 \, d^2$

$M_u = M_r$

$8.41 \, d^2 =$

$264.62 \times 12 \text{ k-inch}$

$d = 19.43''$ (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

$V_c = 0.85(2) \sqrt{3000} \quad :$
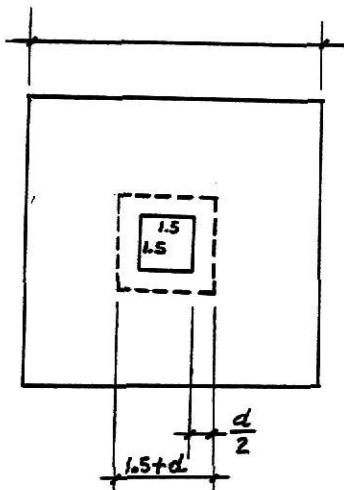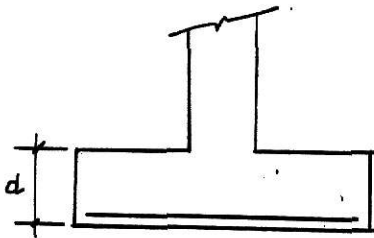
108

$$v_c = 93.11 \text{ psi} = 13.4 \text{ ksf}$$

$$V_c = 13.4(10)d = 134d$$

$$V_{bb} = V_c$$

$$124.525 - 29.3\ d = 134d$$

$$d = 0.762' = 9.15'' \text{ (re-}$$
$$\text{quired for}$$
$$\text{simple}$$
$$\text{shear)}$$

c.3 Design for Diagonal Tension

$$v_c = 4\phi \sqrt{f'c}$$

$$\text{Art 11.11.1 ACI}$$

$$V_c = 4\phi \sqrt{f^1c}\ \ A\ ,$$

$$v_c = 4(0.85)(\sqrt{3000})$$

$$= 186.22 \text{ psi} = 26.82 \text{ ksf}$$

$$V_c = 26.82(4)(1.5 + d)(d)$$

$$V_c = 160.92d + 107.28d^2$$

$$V_u = 2.93\ [10 \times 10$$

$$- (1.5 + d)^2]$$

$$V_u = 2.93\ [100 - (2.25$$

$$+ 3d + d^2]$$

$$V_u = 293 - 6.5925 - 8.79d$$

$$- 2.93d^2$$

$$V_u = 286.4075 - 8.79d$$

$$- 2.93d^2$$

$$V_u = V_c$$

$160.92d + 107.28d^2 = 286.4075 - 8.79d - 293d^2$

$110.85d^2 + 169.71d - 286.4075 = 0$

$d^2 + 1.53d - 2.58 = 0$

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

$d = 1.014'$

$d = 12.17"$

We should use

d = 19.43" *(value required for bending)*.

h = 19.43" + 3" = 22.43" , <u>use h = 22.5"</u>

As required

$$M_u = \phi As \ fy(d - \frac{a}{2}) \qquad a = \frac{AsfY}{0.85f'c \ b}$$

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

$M_u = 54 \ As(19.43 - 0.196 \ As)$

$M_u = 1049.22 \ As - 10.59 \ As^2 = 264.62 \times 12$

$-10.59 \ As^2 + 1049.22 \ As - 3175.44 = 0$

$As^2 - 99.08 \ As + 299.85 = 0$

$$As = \frac{99.08 \overset{+}{_{-}} \sqrt{99.08^2 - 4(1)(299.85)}}{2}$$

$$As = 3.12 \ in^2$$

$$\rho = \frac{As}{bd} = \frac{3.12}{10 \times 12 \times 19.43} = 0.0013$$
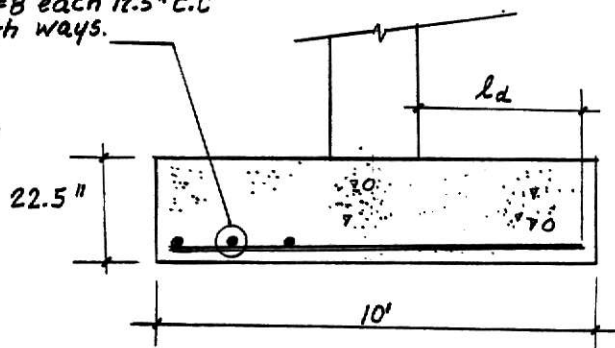
$$\rho_{min.} = \frac{200}{f_y} : \frac{200}{60000} = 0.00\overline{33}$$

We have to use $\rho_{min} = 0.00\overline{33}$; $As_{min} = 0.00\overline{33} \ (120 \times 19.43)$

$$= 7.77 \ in^2.$$

If we use bars #8, $A_s^{\#8} = 0.79 \ in^2$

$$\# \ bars = \frac{7.77}{0.79} : 9.83 = 10 \ bars \ \#8$$

$$spacing : \frac{120" - (2 \times 3")}{2} : 12.67"$$

10#8 each 12.5" c.c
both ways.

ℓ_d

22.5"

10'

Checking Ld

$$Ld_{req} = \frac{0.04 \; A_b f_y}{\sqrt{f'_c}} = \frac{0.04(0.79) \; 60000}{\sqrt{3000}} = 34.62"$$

or

$$Ld_{req} = 0.0004 \; d_b f_y = 0.0004(1)(60000) = 24"$$

$$Ld_{available} : 60" - \frac{18"}{2} - 3" = \underline{48" > 34.62" \; ok}$$

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

effective
surcharge
$$= \gamma df = (\gamma_{sat} - \gamma_w)Df$$

And, the value of γ 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

$$Q_u = CN_c + (\gamma_{sat} - \gamma_w) \; DfNq + 0.5B \; (\gamma_{sat} - \gamma_w)N_\gamma$$

- 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.
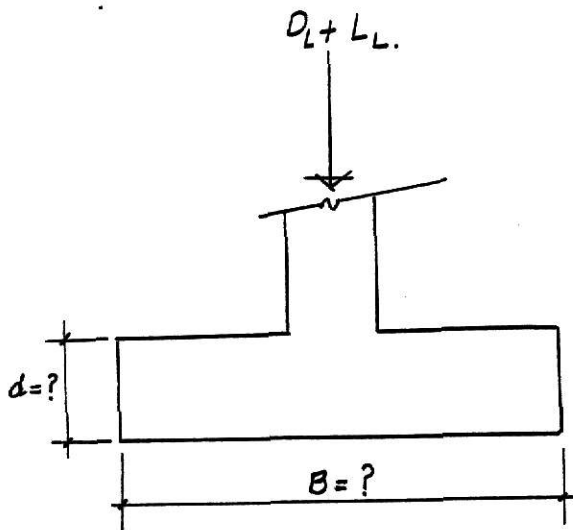

Example 7.2

Spread Footing Design

Given: Soil information: $\phi = 30°$ $C = 0$ $D_f = 3$
$\gamma_{moist} = 120 \; pcf$
water table @ 50'

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Structural data: 12" x 12" column with 4 # 8

D_L = 20K

L_L = 90K

$D_L + L_L.$

f^1c = 3000 psi

f_S = 60 grade

Bearing Capacity

$Q_u = CN_{ct} + \gamma D_f N_q + 0.5\ B\gamma N_\gamma$

Bearing capacity factors (Terzaghi equations)

$\phi,$ deg	N_c	Nq	N
30	37.2	22.5	19.7

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$$Q_u = 3(120)(22.5) + 0.5(B)(120)(19.7)$$

$$Q_u = 8100 + 1182B$$

$$q_a = \frac{Q_u}{3} = 2700 + 394\,B$$

Assuming $q_a = 3000$ psf

$$A_{req} = \frac{P}{q_a} \qquad A_{req} = \frac{(20 + 90)\,KiPs}{3\ ksf} = 36.67\ sf\ ,\ B = 6.06'$$

$$q_a = 2700 + 394(6.06) = 5,087.79 \neq 3000\ assumed$$

Assuming 5090 psf

$$A_{req} = \frac{110}{5.09} = 21.61\ square\ feet \quad B = 4.65'$$

$$q_a = 2700 + 394(4.65) = 4532.1 \neq 5090$$

Assuming $q_a = 4.5$ ksf

$$A_{req} = \frac{110}{4.5} = 24.4\overline{4}\ ,\quad B = 4.94'$$

$$q_a = 2700 + 394(4.94) = 4646.36\ psf = 4.65\ ksf \simeq 4.5\ assumed.$$

Assuming $q_a = 4.6$ ksf

$$A_{req} = \frac{110}{4.6} = 23.91 \quad B = 4.89$$

$$q_a = 2700 + 394(4.89) = 4626.7 = 4.63\ ksf = 4.6$$

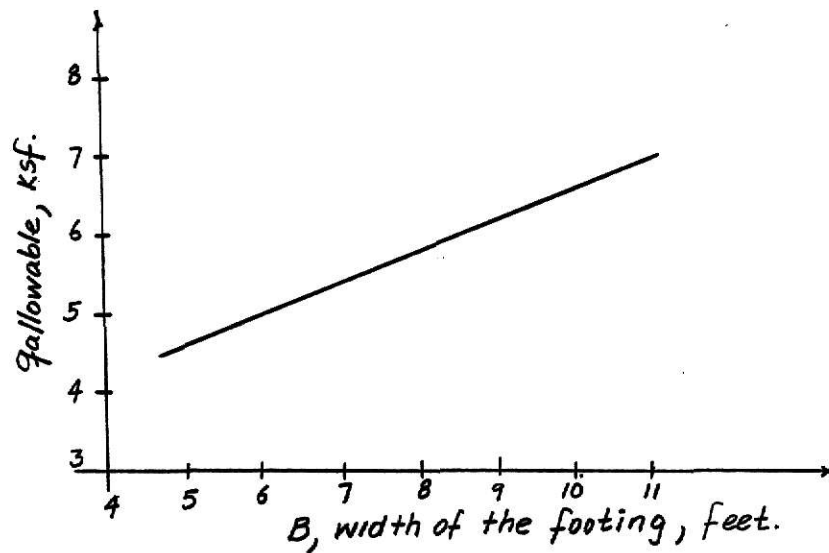$$P_a = 4.62 \times 4.89 = 110.47 \rangle \rangle 110\ ok$$

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

<u>Using B = 5' q_a = 4.67 ksf</u>

B	q_a	$P_{allowable}$
5	4.67	116.75 > 100 ok

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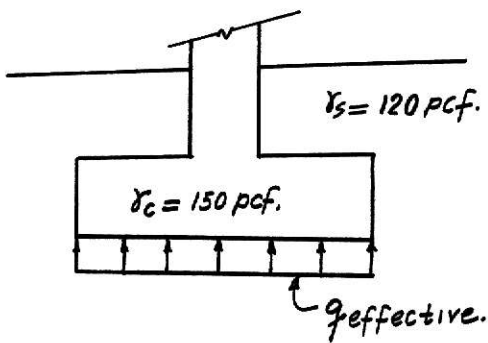| 6 | 5.06 | 182.16 |
7	5.46	267.54
8	5.85	373.76
9	6.25	506.25
10	6.64	664.

B, width of the footing, feet.

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

Design of the footing

$$q_a = 4.67$$

$$\gamma_o = \frac{120 + 150}{2} = 135 \text{ pcf}$$

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$$q_{effective} = q_a - \gamma_o D_f$$

$$= 4.67 - 3(.135)$$

$$= 4.265$$

$$A_{req} = \frac{90 + 20}{4.265} = 25.8$$

$$= 5.08'$$

<u>Use B = 5.25'</u>

Designing for Bending

$$P_u = 1.4(20) + 1.7(90) = 181$$

$$q_u = \frac{P_u}{(5.25)^2} = \frac{181}{5.25^2} = 6.57 \text{ ksf}$$
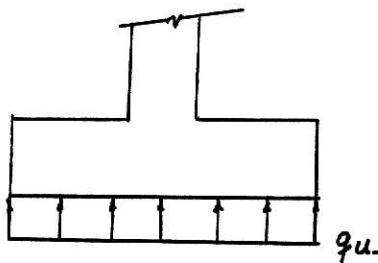
$$M_{aa} = \frac{6.57(5.25)(2.125)^2}{2}$$

$$= 77.87 \text{ k-foot}$$

$$M_u = \phi bd^2 f'c(1 - 0.59 q)$$

$$q = \frac{\rho fy}{f'c}, \quad \phi = 0.9 \quad fy = 60 \text{ ksi}$$

$$f'c = 3 \text{ ksf}$$

$$\rho = 0.016$$

(Art 10.33 ACI Code)

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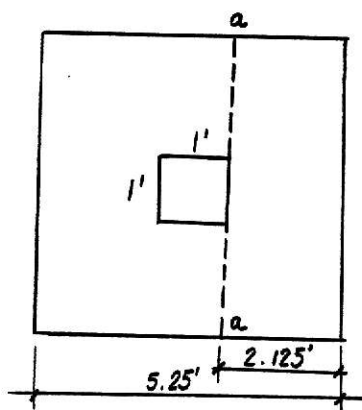
$$q = \frac{0.016(60)}{3}$$

$$q = 0.32$$

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

$$M_u = 44.16 \, d^2$$

$$M_u = M_r$$

$$44.16 \, d^2 = 77.89 \times 12$$

<u>d flexion = 4.6"</u>

Design for Diagonal Tension

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

$$\frac{a}{B} = \frac{1" \text{ (column width)}}{5.25} = 0.19$$

$$\frac{q_u}{V_c} = \frac{6570 \text{ Psf}}{186.22 \text{ Psi}} = 35.28$$

- Using these values from Fig. 7.3 is obtained :

$$\frac{d}{B} = 0.16$$

$$d = 0.16 \times 5.25'$$

$$d = 10.08"$$

Art ACI 11.11.2 $V_{c_{\text{diagonal}}} = 4(0.85)\sqrt{3000} = 186.22 \text{ psi}$

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Plot: Vertical axis labeled *Ratio, d/B* marked from 0 to 0.5 in 0.1 increments. Horizontal axis labeled *Ratio q_n/v_c* marked 0, 50, 100, 150, 200. Curves labeled:
- $a/B = 0$
- $a/B = 0.1$
- $a/B = 0.2$
- $a/B = 0.3$
- $a/B = 0.4$

v_c = shear stress for strength design (lb/sq in.)

q_n = factored net soil pressure (lb/sq ft)

Rectangular footings

Before entering curves, find $B' = B\sqrt{2.5-1}$ and let B' replace B in a/B and d/B ratios
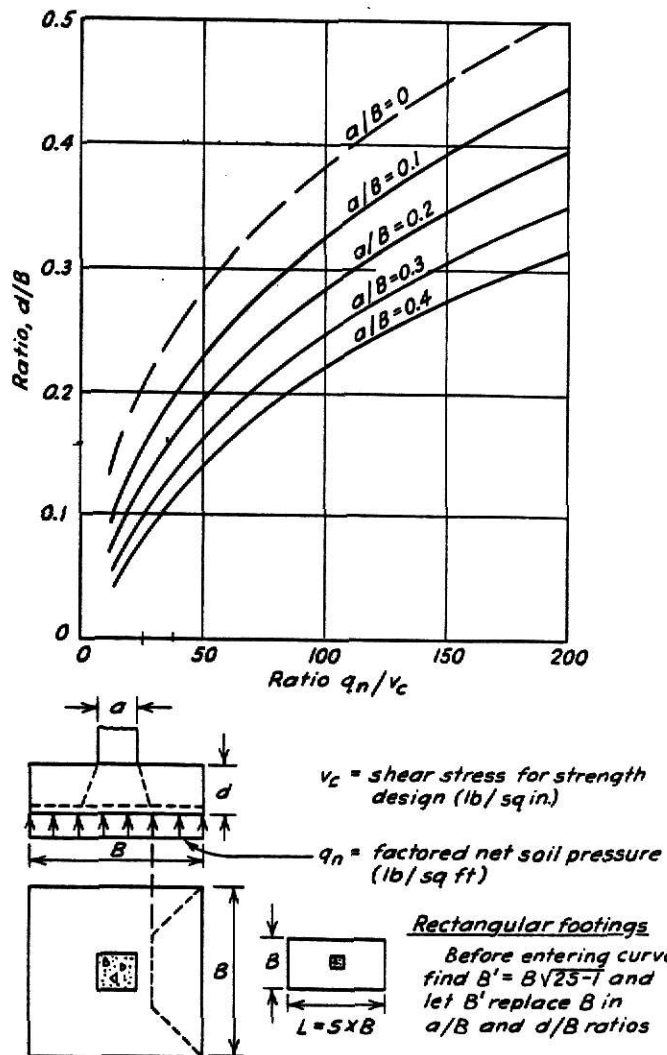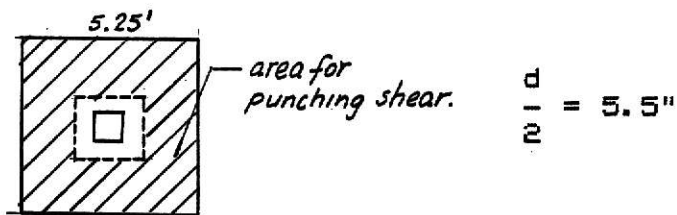
$L = 5 \times B$

Table 7.3. Curves for selecting depth of footing as determined by shear, provided by two-way behavior prevails.

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We should assume d = 11"   t = 11 + 3" = 14"

5.25'

area for punching shear.

$\dfrac{d}{2} = 5.5"$

$$V_u = \frac{6.57 \text{ Ksf} \times 1000/144 \text{ Psi}) [5.25' \times 12 - 23]^2 - 23^2}{4(23)(11)}$$

$V_u = 155.09 \text{ psi} < V_c = 186.22 \text{ psi}$ ok

$d_{required} = 11"$
  for diagonal
  tension
_____

Design for beam shear

63"

d

d | 14.5
25.5"
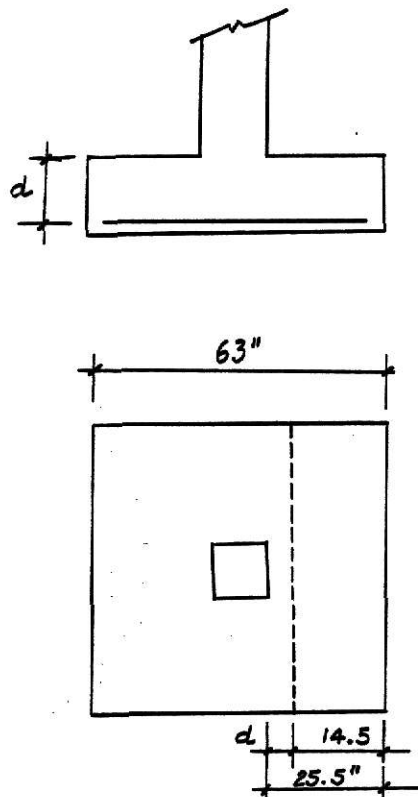
$q_u = 6.57 \text{ Ksf} = 45.63 \text{ psi}$

$V_c = 0.85(2) \ f^1 c$

   Art 11.3.1.1 ACI

$V_c = 0.85(2) \sqrt{3000} = 93.11 \text{ psi}$

$$V_u = \frac{45.63 \text{ Psi} (14.5" \times 63")}{63" (d)}$$

$$= \frac{661.64}{d}$$

$V_c = V_u$

$93.11 = \dfrac{661.64}{d}$, d = 7.11"
                        beam shear
                        _____

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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:
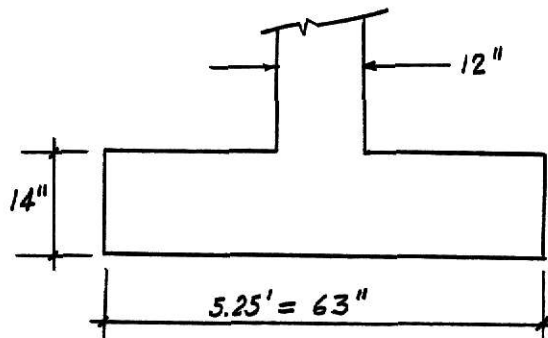

Steel Area Required

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

$$M_u = \phi \, As \, fy \left(d - \frac{a}{2} \right) \text{ where } a = \frac{A_s \, fy}{0.85 \, f'_c b}$$

$$M_u = 0.9 \, As(60) \left[11 - \frac{As(60)}{2(0.85)(3)63} \right]$$

$$M_u = 594 \, A_s - 10.08 \, A_s^2$$

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

$$A_s = \frac{-(-594) \pm \sqrt{(-594)^2 - 4(10.08)(934.68)}}{2(10.08)}$$

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

$$A_s = 1.62 \text{ in}^2$$

$$\rho = \frac{A_s}{bd} = \frac{1.62}{63 \times 11} : 0.0023 \; ; \; \text{use } \rho \text{min!}$$

$$\rho \text{min} = \frac{200}{f_y} = \frac{200}{60000} = 0.00\overline{33}$$

$$A_{s \, min} = 0.00\overline{33} \, (63 \times 11) = 2.31 \text{ in}^2$$

If we use #6 A_s #6 = 0.44 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.

14"

5.25'
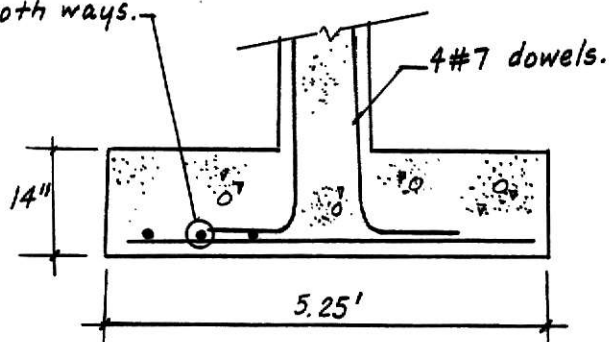
Checking the Development Length (L_d)

$$L_{d_{req}} = \frac{0.04 \ A_b \ fy}{\sqrt{f'c}} = \frac{0.04 \ (.44)(60000)}{\sqrt{3000}} = 19.28"$$

$$L_{d_{req}} = 0.0004 \ d_b fy = 0.004(0.75)(60000) = 18"$$

$$L_{d_{available}} = \frac{25.5" - 3" = 22.5" > 19.28" \ ok}{}$$

Checking the bearing strength

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

$$\sqrt{A_2/A_1} = \sqrt{\frac{63 \times 63}{12 \times 12}} = 5.25 \quad , \quad use \ 2$$

$$f_c = 0.85(0.7)(3)(2) = 3.57 \ Ksi > f'c$$

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

Transfer of Stress at Base of Column

At least four dowels with total $A_s > 0.005 \ Ag$

$$A_{s_{req}} = 0.005(144) = 0.72 \ in^2 \quad Use \ 4 \ \# \ 7$$

Diameter difference $= \underset{\underset{\#8}{C}}{1.00} - 0.875 = 0.125 < 0.25 \quad ok$

$$L_{d_{\substack{compression \\ bars \ required}}} = \frac{0.02(fy)(d_b)}{\sqrt{3000}} = \frac{0.02(60000)(0.875)}{\sqrt{3000}}$$

$$= 16" > 11" \ (available)$$

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$$L_{d_{reg}} = 0.003(f_y)(d_b) = 0.0003(60000)(0.875)$$

$$= 13.1" > 11"$$

We should reduce L_d to L^1d

$$L^1d = L_d \frac{A_{req}}{A_{furnished}} = \frac{0.72}{2.40} \times 16" = 4.8"$$

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:

$$q_{max} = \frac{P}{BL} (1 + \frac{6e}{L})$$

(7.1)

that is,

$$q_a \, > \, q_{max}$$

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

$$\frac{-L}{6} < e < \frac{L}{6} \quad \text{(see Fig. 7.4a and 7.4b)}$$

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
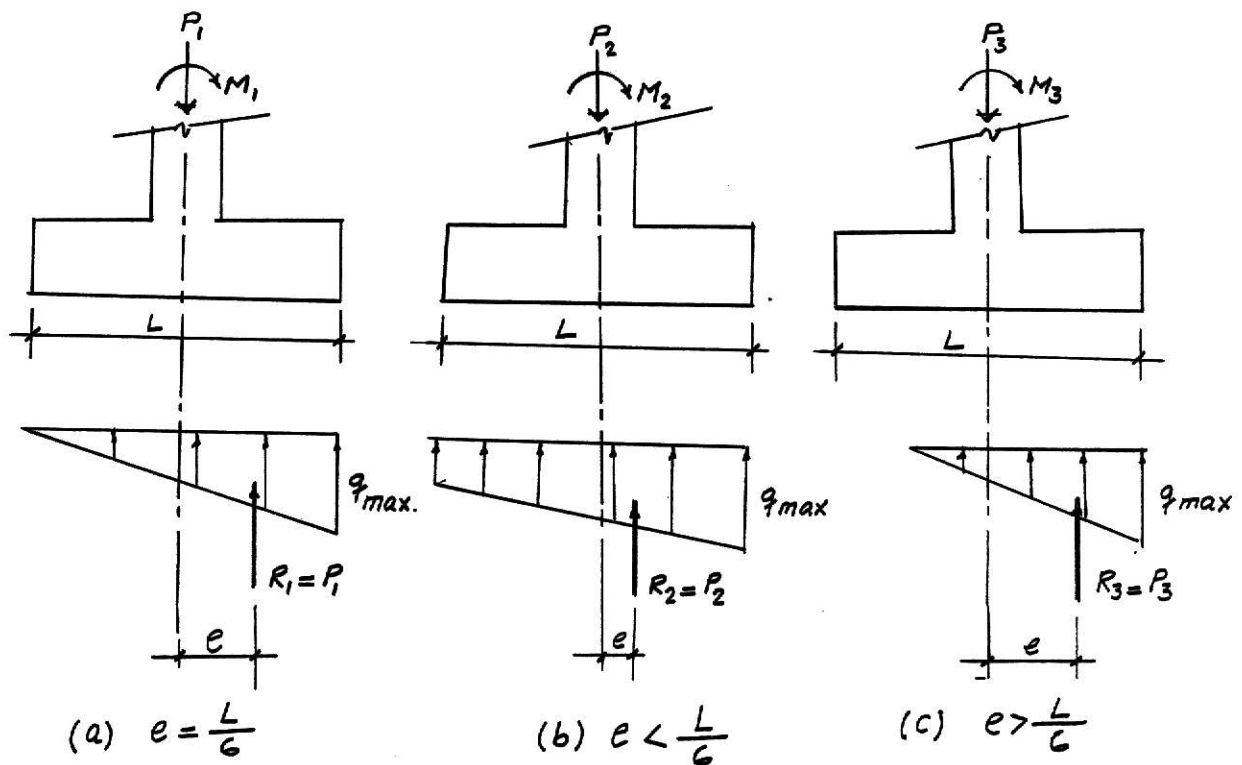subjected to tensile stress is assumed to be unable to carry

Fig 7.4 Pressure distribution under footings eccentrically loaded.

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

    A = BL'

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

$$L' = 3(\frac{L}{2} - e)$$

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

$$R = \frac{q_{max}}{2} BL'$$

Substituting L' into the expression for R is obtained.

$$q_{max} = \frac{2R}{3B(\frac{L}{2} - e)} \ \sqrt{\ } \ q_{allowable} \qquad (7.2)$$

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

$$q_{any \atop Point} = \frac{p}{A} \pm \frac{M_y x}{I_y} \pm \frac{M_x y}{I_x}$$

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 Peck 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.

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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
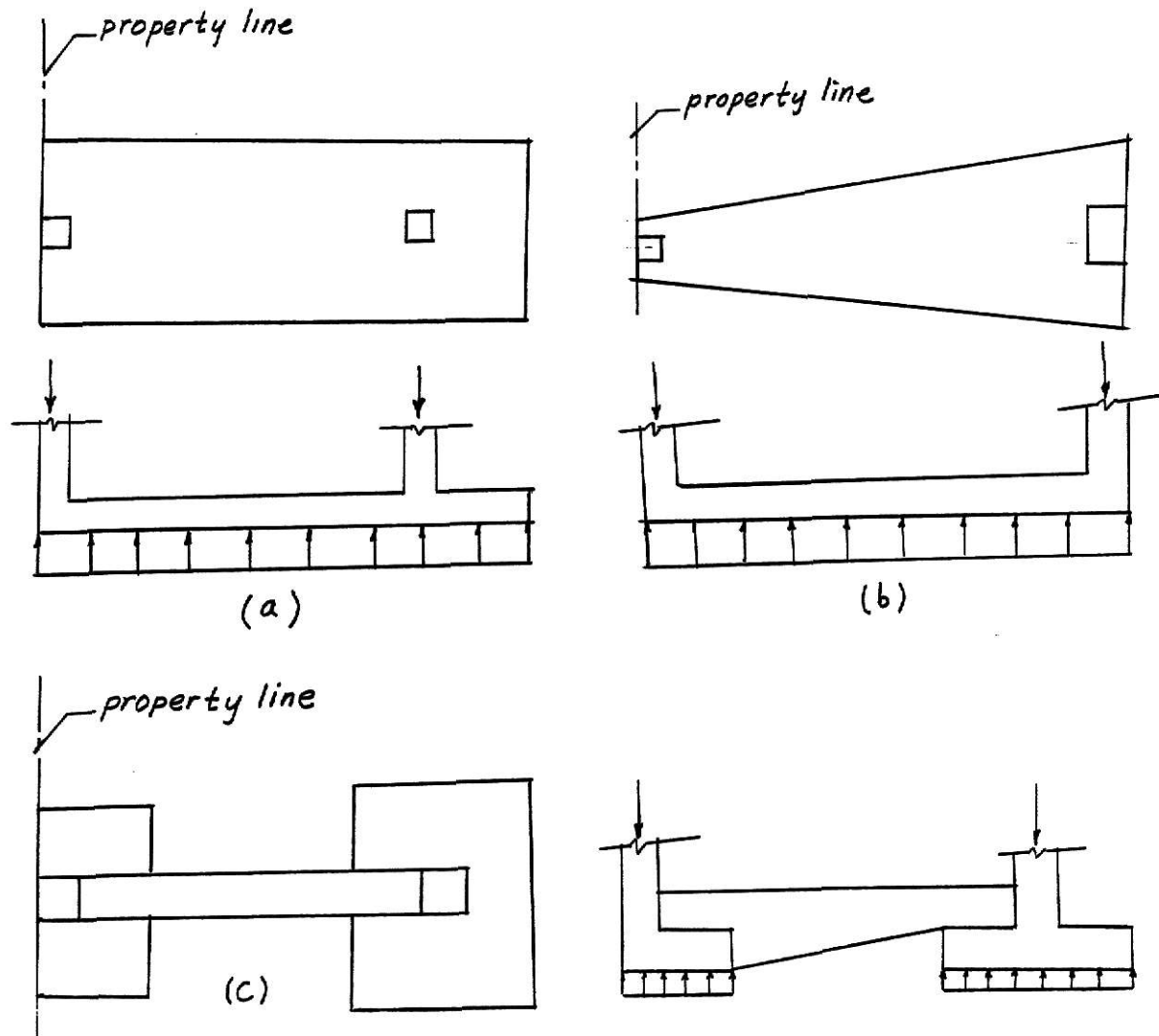
Figure 8.1. Combined footings: (a) Rectangular footing; (b) Trapezoidal footing; (c) Cantilever footing.
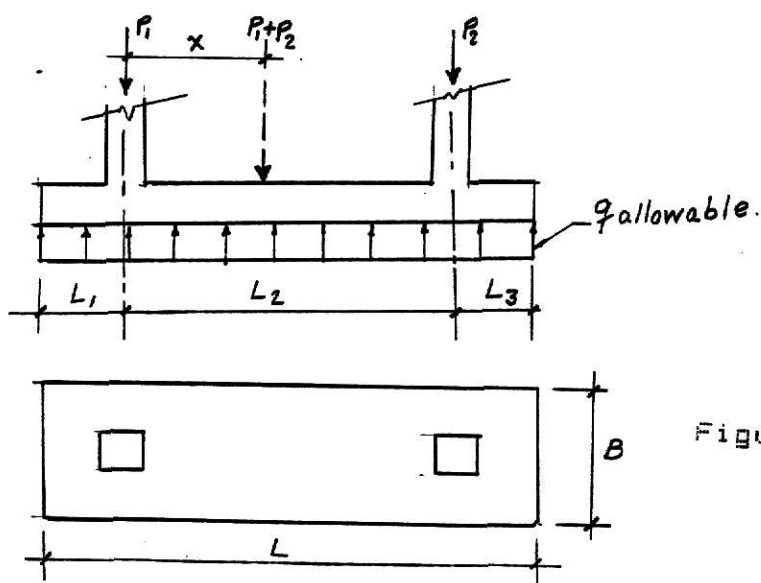

Figure 8.2. Rectangular footing

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

$$L_3 = L - (L_1 + L_2) \tag{8.5}$$

d.   Determine the width of the foundation (B).

$$B = \frac{A}{L} \tag{8.6}$$

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:
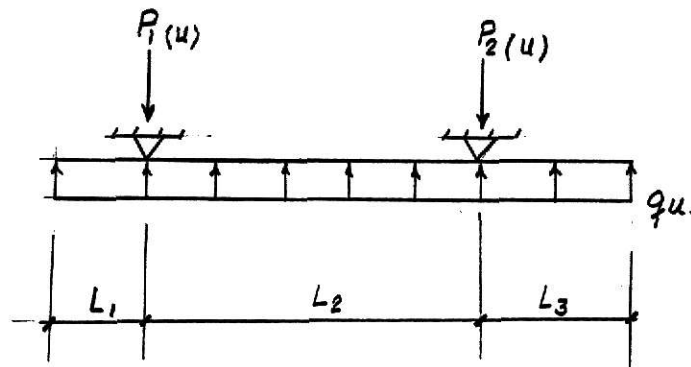
(a) Simple supported beam.

2. Trapezoidal Combined Footings (see Fig. 8.3)
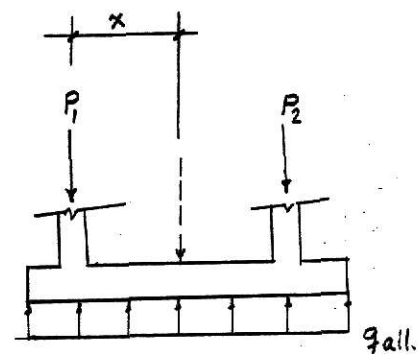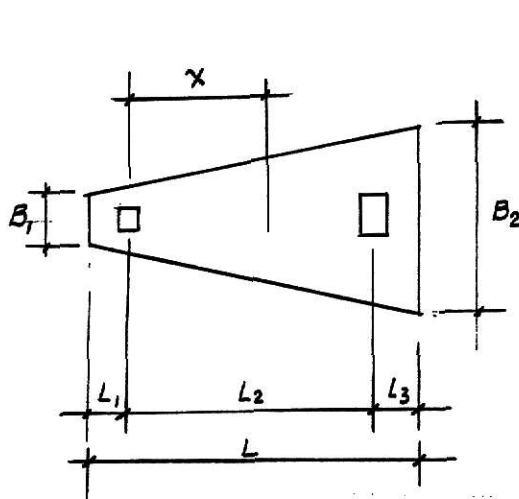
Figure 8.3.   Trapezoidal footing.

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a) Determine the area of the foundation as:

$$A = \frac{P_1 + P_2}{q_{allow}} \qquad (8.7)$$

and

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

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:

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

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

$$\frac{L}{3} < (x + L_1) < \frac{L}{2}$$

3. __Cantilever Footing__ (see Fig. 8.4)

Figure 8.4. Cantilever Footing.
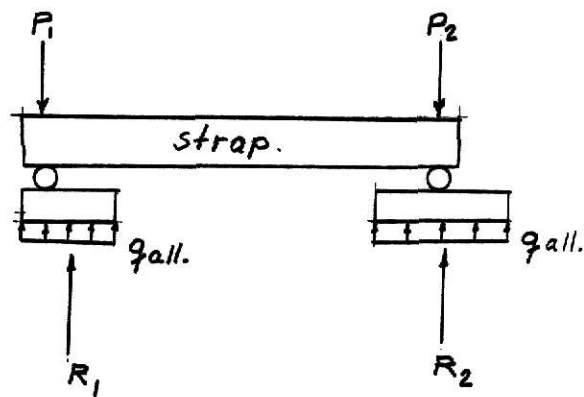
Figure 8.4.   b) Principle of cantilever footing.

a) Fox the dimension B^1 and obtain e and L^1.   Notice that the values of "a" and "s" are normally known.

$$e = \frac{B_1}{2} - a \qquad (8.11)$$

$$L_1 = L - e \qquad (8.12)$$

b) Obtain R_1 taking $\int M$ about $R_2 = 0$, $R_1 = \frac{P_1 \, L}{L_1} \qquad (8.13)$

Obtain R_2 taking $\int M$ about $R_1 = 0$ $\qquad R_2 = \frac{P_2 L - P_1 e}{L} \qquad (8.14)$

check by $F_v = 0$, $\qquad R_1 + R_2 = P_1 + P_2$

c) Obtain B_2 and B_3 as

$$B_2 = \frac{R_1}{q_{allow.} B_1} \quad (8.15) \; ; \; (B_3)^2 = \frac{R_2}{q_{all.}} \quad (8.16)$$

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

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

where

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$$UR = \frac{P_{ultimate\ total}}{P_{working\ total}}$$

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

$$I_{strap}/I_{footing} > 2$$

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

$$q_{allow} = \frac{CN_c + \gamma D_f\ N_q + 1/2\ B\gamma N_\gamma}{F\ S}$$

where c, N_c, γ, D_f, N_q, N_γ, and $F\ S$ are given.
Then, referring to Fig. 8.2,

$$q_{all} = \frac{P_1 + P_2}{B \times L} = \frac{c\ N_c + \gamma D_f\ N_q + 1/2\ B\ \gamma N_\gamma}{F\ S} \qquad (8.17)$$

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.

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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.

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

$$+ \; 1/2 \; \gamma B N_\gamma \, F_{\gamma s} \, F_{\gamma d} \, F_{\gamma i} \tag{8.18}$$

The allowable bearing capacity is given by

$$q_{all \atop (net)} = \frac{q_u - \gamma D_f}{FS}. \quad \text{FS is usually 3}$$

The term B in the preceding equation is the smallest dimension of the mat.
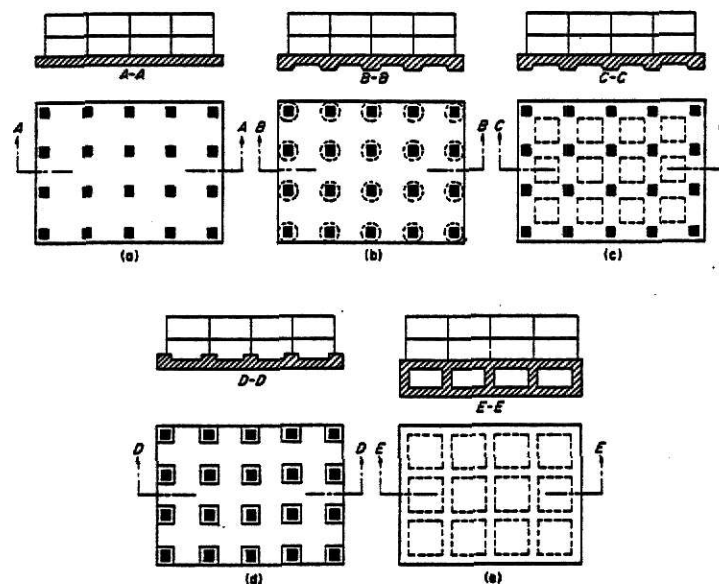
Figure 8.5.   Common types of mat foundations.   (a) Flat plate; (b) plate thickened under columns; (c) beam-and-slab; (d) plate with pedestals; (e) basement walls as part of mat.

For mat foundations placed over sand, the bearing capacity may be obtained from the standard penetration resistance numbers.

$$q_{all} = 11.98 \ N' \ \left(\frac{3.28B + 1}{3.28B}\right)^2 \ F_d\left(\frac{S}{25.4}\right) \ \left(\frac{KN}{m^2}\right) \qquad (8-19)$$

where

N' = corrected standard penetration resistance

B = width (m)

$$F_d = 1 + 0.33 \ \left(\frac{D_f}{B}\right) < 1.33$$

S = settlement in mm

N' = 15 + 1/2 (N-15) if N > 15 (Terzagh 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

$$K_r = \frac{E \ I_b}{E_s \ B^3} \qquad (8.20)$$

where

E = modulus of elasticity of the material used in the structure

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E_s = modulus of elasticity of the soil

B = width of foundation

I_b = moment of inertia of the superstructure per unit of

length at right angles to B.

The term $E\ I_b$ can be computed as

$$E\ I_b = E(I_f + \sum I_b' + \sum \frac{ah^3}{12}) \qquad (8.21)$$

where

EI_b = flexural rigidity of the superstructure and

footing

EI_f = flexural rigidity of the foundation per unit of

length at right angles to B

$E\sum I_b'$ = flexural rigidity of the frame members

$E\sum \frac{ah^3}{12}$ = flexural rigidity of the shear walls

perpendicular to B.

a = shear wall thickness

h = 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

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indicated above because the ACI method takes into account the stiffness of the superstructure.

K_r	Differential settlement
0	$0.35 \times \Delta H$ for long base, $\dfrac{B}{L} - 0$
	$0.35 \times \Delta H$ for square mats, $\dfrac{B}{L} = 1$
0.5	$0.1 \times \Delta H$
> 0.5	Rigid mat, no differential settlement

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:

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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 β is equal to

$$= \sqrt[4]{\frac{BK}{4 E I}} \qquad (8.22)$$

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,

$$Q_T = \sum_{i=1}^{n} Q_i \qquad (8.24)$$

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

$$q = \frac{Q}{A} + \frac{My\ x}{Iy} + \frac{Mx\ y}{Ix} \qquad (8.24)$$

where

A = B x L

$$I_x = \frac{BL^3}{12} = \text{moment of inertia about the x axis}$$

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$$Iy = \frac{LB^3}{12} = \text{moment of inertia about the y axis}$$

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

M_x = moment of Q_T about the x axis = Q_T ey

My = moment of Qy about the y axis = Q_T e_x

e_x ey = load eccentricities that can be found as explained before

3. Compare the soil pressure values with the net allowable soil pressure. Remember q_{anY} $\langle$ $q_{allowable}$ net.
Point

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:

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

(8.25)

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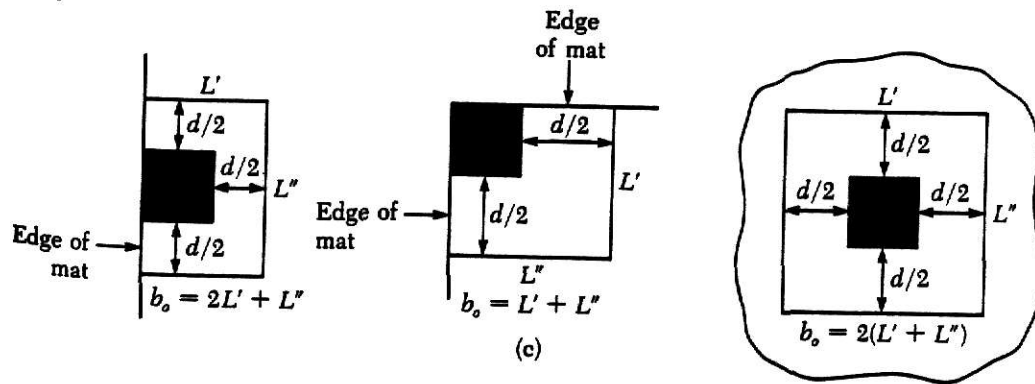
Figure 8.6. Critical sections for punching shear.

(a)

(b)
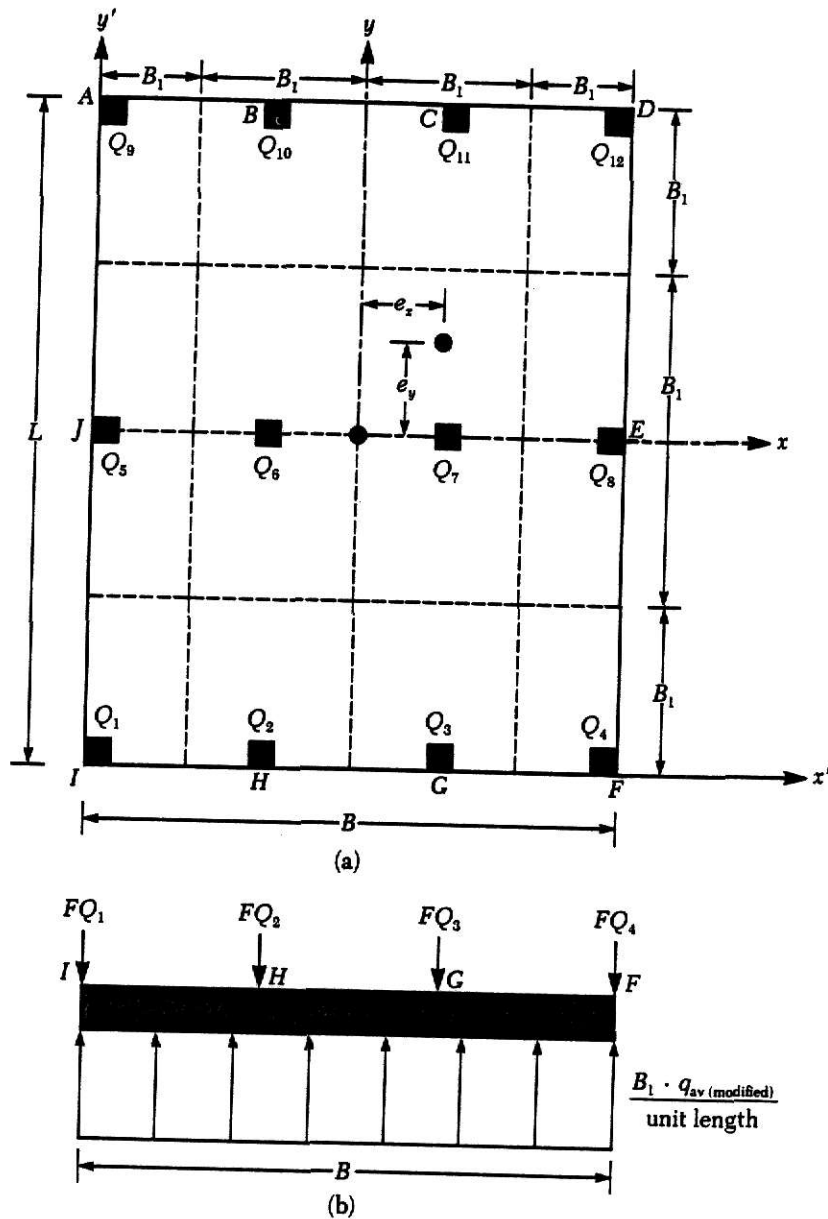
Figure 8.7. Conventional rigid mat foundation design.

$$q_{av} = q_{av} \left(\frac{\text{average load}}{\text{soil reaction}} \right) \qquad (8.26)$$
$$\text{(adjusted)}$$

$$\begin{matrix} \text{(column load} \\ \text{modification } F = \dfrac{\text{average load}}{\text{column loads on the strip}} \qquad (8.27) \\ \text{factor)} \end{matrix}$$

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

$$Q_{\text{modified}}{}_i = F \; Q_i$$

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).
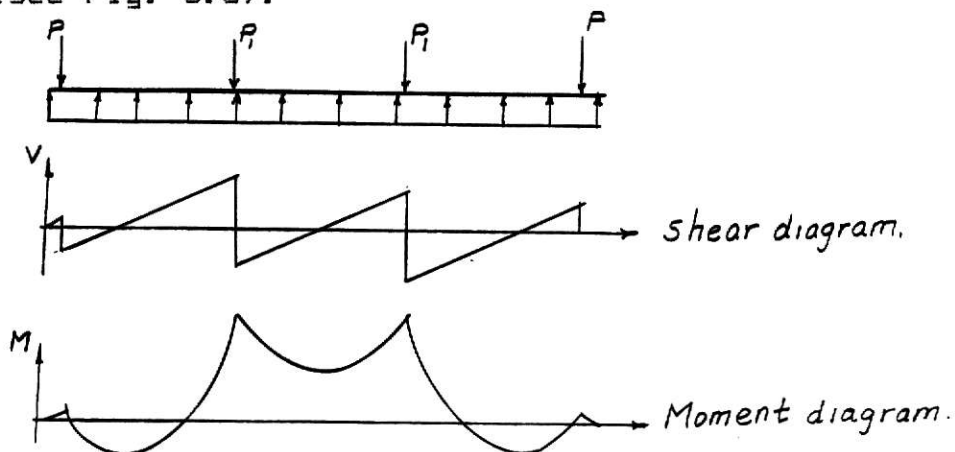
Figure 8.8. Typical shear and moment diagrams for a strip.

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 those earth pressures in order to design such structures safely and economically.
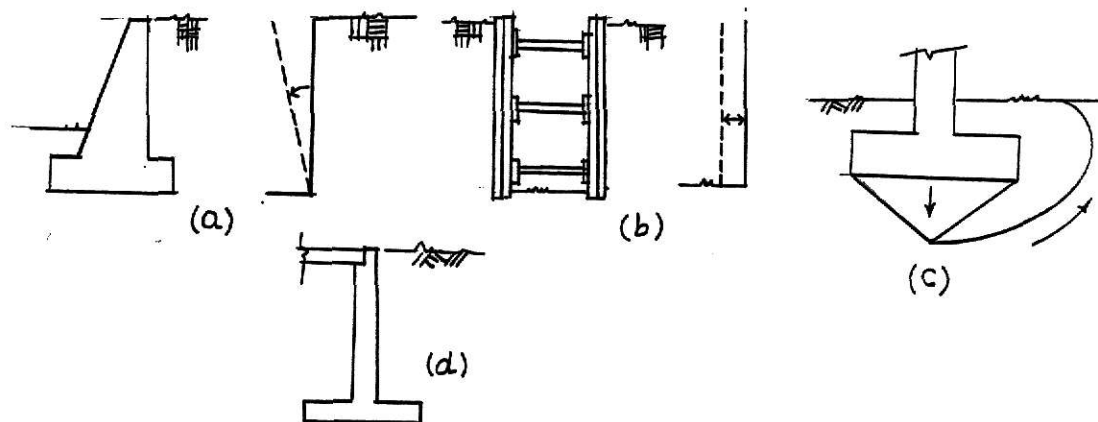
Figure 9.1. Civil Engineering structures with different types of displacement caused by earth pressures. (a) Retaining wall; (b) bracing of cut; (c) footing; (d) basement wall.

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

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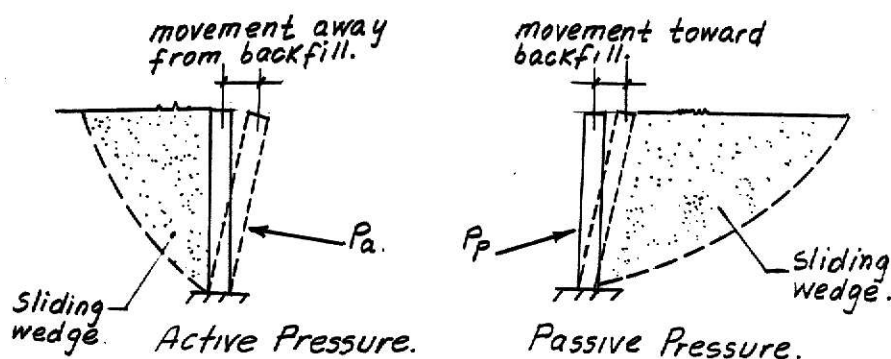
In this chapter attention will be given to the different
theories available to determine active, passive and at rest
earth pressures.

As a way of illustration, the Mohr's circle can be used
to understand the concepts mentioned above. Suppose <u>a soil</u>
<u>deposit with a horizontal surface and no shearing stresses on</u>
<u>horizontal or vertical surface</u>, as shown in Fig. 9.3. If an
element of soil which originally was subjected to a vertical
stress ($\emptyset$1) and to a horizontal stress ($\emptyset$2) is stretched in
the horizontal direction, it will behave as a specimen in the
triaxial machine where the confining stress is decreased and
the axial stress is remained constant. The minimum stress
reached before the soil fails is ($\emptyset$3) and is the active
pressure mentioned. On the other hand, if the soil is
compressed in the horizontal direction, it will behave as a
soil specimen in the triaxial machine where the horizontal
stress is increased and the vertical stress is kept constant.
The maximum value that the horizontal stress can reach is
($\emptyset$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
faillure 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 vircles represent a
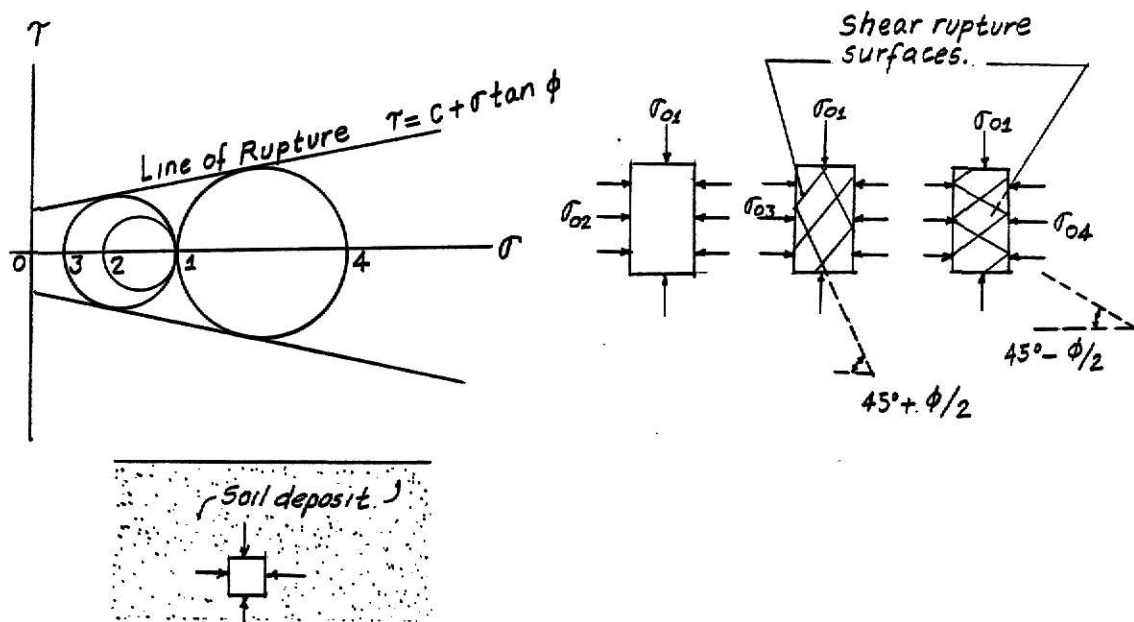
Figure 9.3. Passive, Active, at Rest Pressure.

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:

$$\sigma_{\emptyset 3 \; (active)} = \sigma_{\emptyset 1} \tan^2 \left(45 - \frac{\phi}{2}\right) - 2c \tan\left(45 - \frac{\phi}{2}\right) \quad (9.1)$$

or

$$= \sigma_{\emptyset 1} K_a - 2c\sqrt{K_a}$$

$$\sigma_{\emptyset 4 \; Passive} = \sigma_{\emptyset 1} \tan^2\left(45 + \frac{\phi}{2}\right) + 2c \tan\left(45 + \frac{\phi}{2}\right) \quad (9.2)$$

$$= \sigma_{\emptyset 1} K_p + 2c\sqrt{K_p}$$

where K_a and Kp are the active coefficient earth pressure and

passive coefficient earth pressure respectively.

Also,

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

$\phi =$ angle of internal friction

$C =$ cohesion

The coefficient for earth pressure at rest (K_0) can be approximated by

$$K_0 = 1 - \sin\phi \tag{9.3}$$

for normally conolidated soils. For over-consolidated soils and compacted soils, K_0 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

Soil	K
Compacted clay, hand tamped	1.0 to 2.0
Compacted clay, machine tamped	2.0 to 6.0
Clay, overconsolidated	1.0 to 4.0
Sand, loosely damp	0.5
Sand, compacted	1.0 to 1.5

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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
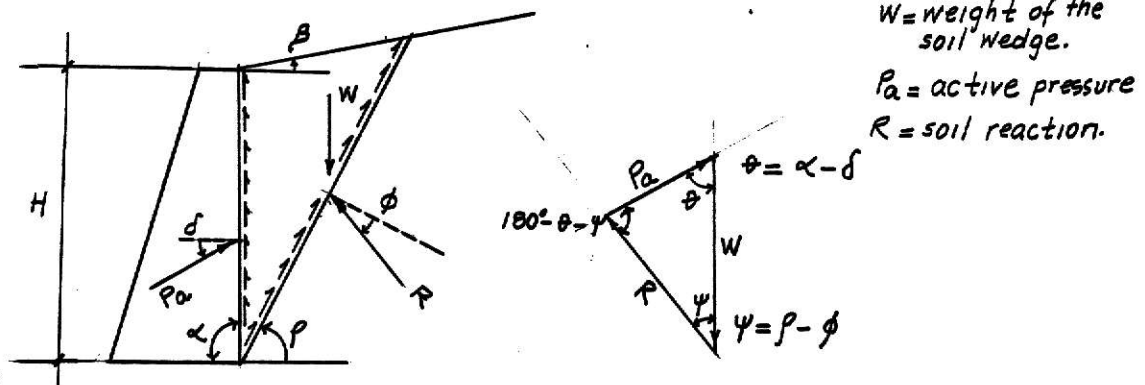
Figure 9.4

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 (ρ) then, applying the concept of the first

derivative in function of ρ is equal to zero, that is

$$\frac{dPa}{d\rho} = 0$$

the active earth pressure becomes

$$Pa = \frac{\gamma H^2}{2} \times$$

(9.4)

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

or

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

(9.5)

Note that the value of K_a depends on α, β, δ and ϕ which are:

α = backwall inclination

β = surface inclination

δ = friction angle between soil and wall

ϕ = friction angle

Passive earth pressure can be obtained solving the triangle of forces shown in Fig. 9.5
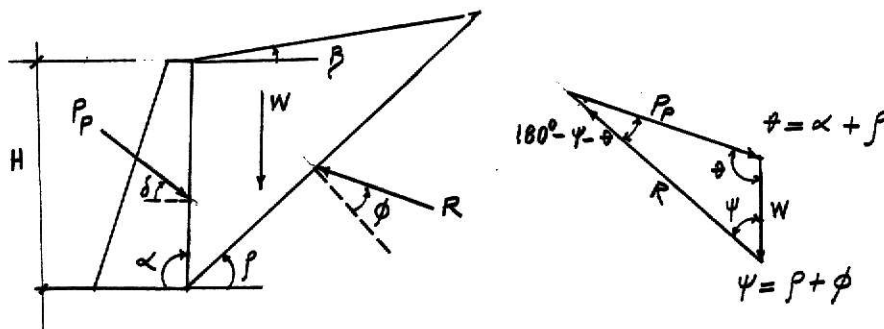
Figure 9.5. Coulomb's Passive Pressure.

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If $\dfrac{dPp}{d\rho} = 0$, $Pp = \dfrac{\gamma H^2}{2}$

$$\dfrac{\sin(\alpha - \phi)}{\sin^2 \alpha \sin(\alpha + \delta)[1 - \sqrt{\dfrac{\sin(\phi + \delta)\sin(\phi - \beta)}{\sin(\alpha + \delta)\sin(\alpha + \beta)}}]^2} \qquad (9.6)$$

or

$$P_\rho = \dfrac{\gamma H^2}{2} Kp \qquad (9.7)$$

For a smooth vertical wall with horizontal backfill and right-angle wall that is, $\delta = 0$, $\beta = 0$, $\alpha = 90°$, the equation 9.4 and 9.6 become:

$$P_a = \dfrac{\gamma H^2}{2} \tan^2 (45 - \dfrac{\phi}{2})$$

$$Pp = \dfrac{\gamma H^2}{2} \tan^2 (45 + \dfrac{\phi}{2})$$

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

```
     ALPHA =90.        BETA = 0.

 δ    φ=26   28     30     32     34     36     38     40
 0    0.390 0.361 0.333 0.307 0.283 0.260 0.238 0.217
16    0.349 0.324 0.300 0.278 0.257 0.237 0.218 0.201
17    0.348 0.323 0.299 0.277 0.256 0.237 0.218 0.200
20    0.345 0.320 0.297 0.276 0.255 0.235 0.217 0.199
22    0.343 0.319 0.296 0.275 0.254 0.235 0.217 0.199

     ALPHA =90.        BETA = 5.

 δ    φ=26   28     30     32     34     36     38     40
 0    0.414 0.382 0.352 0.323 0.297 0.272 0.249 0.227
16    0.373 0.345 0.319 0.295 0.272 0.250 0.229 0.210
17    0.372 0.344 0.318 0.294 0.271 0.249 0.229 0.210
20    0.370 0.342 0.316 0.292 0.270 0.248 0.228 0.209
22    0.369 0.341 0.316 0.292 0.269 0.248 0.228 0.209

     ALPHA =90.        BETA =10.

 δ    φ=26   28     30     32     34     36     38     40
 0    0.443 0.407 0.374 0.343 0.314 0.286 0.261 0.238
16    0.404 0.372 0.342 0.315 0.289 0.265 0.242 0.221
17    0.404 0.371 0.342 0.314 0.288 0.264 0.242 0.221
20    0.402 0.370 0.340 0.313 0.287 0.263 0.241 0.220
22    0.401 0.369 0.340 0.312 0.287 0.263 0.241 0.220
```

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

```
     ALPHA =90.        BETA = 0.

 δ    φ=26    28      30      32      34      36      38      40
 0    2.561  2.770  3.000  3.255  3.537  3.852  4.204  4.599
16    4.195  4.652  5.174  5.775  6.469  7.279  8.230  9.356
17    4.346  4.830  5.385  6.025  6.767  7.636  8.662  9.882
20    4.857  5.436  6.105  6.886  7.804  8.892 10.194 11.771
22    5.253  5.910  6.675  7.574  8.641  9.919 11.466 13.364

     ALPHA =90.        BETA = 5.

 δ    φ=26    28      30      32      34      36      38      40
 0    2.943  3.203  3.492  3.815  4.177  4.585  5.046  5.572
16    5.250  5.878  6.609  7.464  8.474  9.678 11.128 12.894
17    5.475  6.146  6.929  7.850  8.942 10.251 11.836 13.781
20    6.249  7.074  8.049  9.212 10.613 12.321 14.433 17.083
22    6.864  7.820  8.960 10.334 12.011 14.083 16.685 20.011

     ALPHA =90.        BETA =10.

 δ    φ=26    28      30      32      34      36      38      40
 0    3.385  3.713  4.080  4.496  4.968  5.507  6.125  6.841
16    6.652  7.545  8.605  9.876 11.417 13.309 15.665 18.647
17    6.992  7.956  9.105 10.492 12.183 14.274 16.899 20.254
20    8.186  9.414 10.903 12.733 15.014 17.903 21.636 26.569
22    9.164 10.625 12.421 14.659 17.497 21.164 26.013 32.602
```

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

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

[†] 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. 9.6.
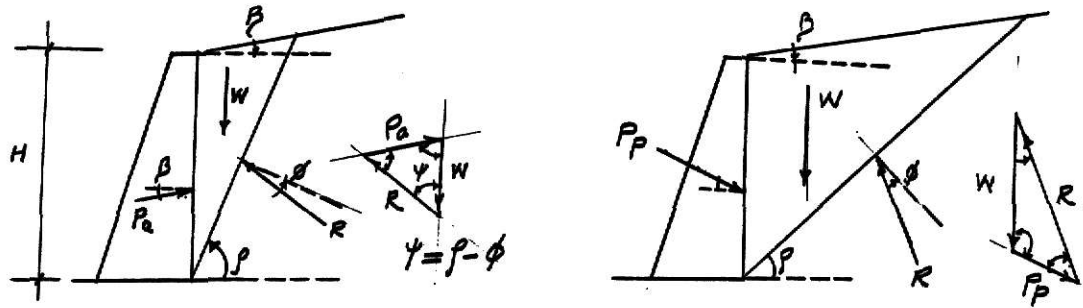
Figure 9.6. Rankine's Principle for Active and Passive Earth Pressures.

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

$$\frac{dP_a}{d\rho} = 0 \text{ and } \frac{dP_p}{d\rho} = 0, \text{ is obtained.}$$

$$P_a = \frac{\gamma H^2}{2} \cos\beta \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} \qquad (9.8)$$

or

$$P_a = \frac{\gamma H^2}{2} Ka_R \qquad (9.9)$$

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

or

$$P_p = \frac{\gamma H^2}{2} K_{pR}$$

Values K_{aR} and K_{pR} 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 $\beta > \phi$

β	$\phi=$ 26	28	30	32	34	36	38	40	42
0	0.3905	0.3610	0.3333	0.3073	0.2827	0.2596	0.2379	0.2174	0.1982
5	0.3959	0.3656	0.3372	0.3105	0.2855	0.2620	0.2399	0.2192	0.1997
10	0.4134	0.3802	0.3495	0.3210	0.2944	0.2696	0.2464	0.2247	0.2044
15	0.4480	0.4086	0.3730	0.3405	0.3108	0.2834	0.2581	0.2346	0.2129
20	0.5152	0.4605	0.4142	0.3739	0.3381	0.3060	0.2769	0.2504	0.2262
25	0.6999	0.5727	0.4936	0.4336	0.3847	0.3431	0.3070	0.2750	0.2465
30	0.0000	0.0000	0.8660	0.5741	0.4776	0.4105	0.3582	0.3151	0.2784
35	0.0000	0.0000	0.0000	0.0000	0.0000	0.5971	0.4677	0.3906	0.3340
40	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.7660	0.4668

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

β	$\phi=$ 26	28	30	32	34	36	38	40	42
0	2.5611	2.7698	3.0000	3.2546	3.5371	3.8518	4.2037	4.5989	5.0447
5	2.5070	2.7145	2.9431	3.1957	3.4757	3.7875	4.1360	4.5272	4.9684
10	2.3463	2.5507	2.7748	3.0216	3.2946	3.5979	3.9365	4.3161	4.7437
15	2.0826	2.2836	2.5017	2.7401	3.0024	3.2925	3.6154	3.9766	4.3827
20	1.7141	1.9175	2.1318	2.3618	2.6116	2.8857	3.1888	3.5262	3.9044
25	1.1736	1.4343	1.6641	1.8942	2.1352	2.3938	2.6758	2.9867	3.3328
30	0.0000	0.0000	0.8660	1.3064	1.5705	1.8269	2.0937	2.3802	2.6940
35	0.0000	0.0000	0.0000	0.0000	0.0000	1.1238	1.4347	1.7177	2.0088
40	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.7660	1.2570

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 $K_p = K_a = 1$ for water and its distribution is linear with depth.   Then, water pressure is another force to
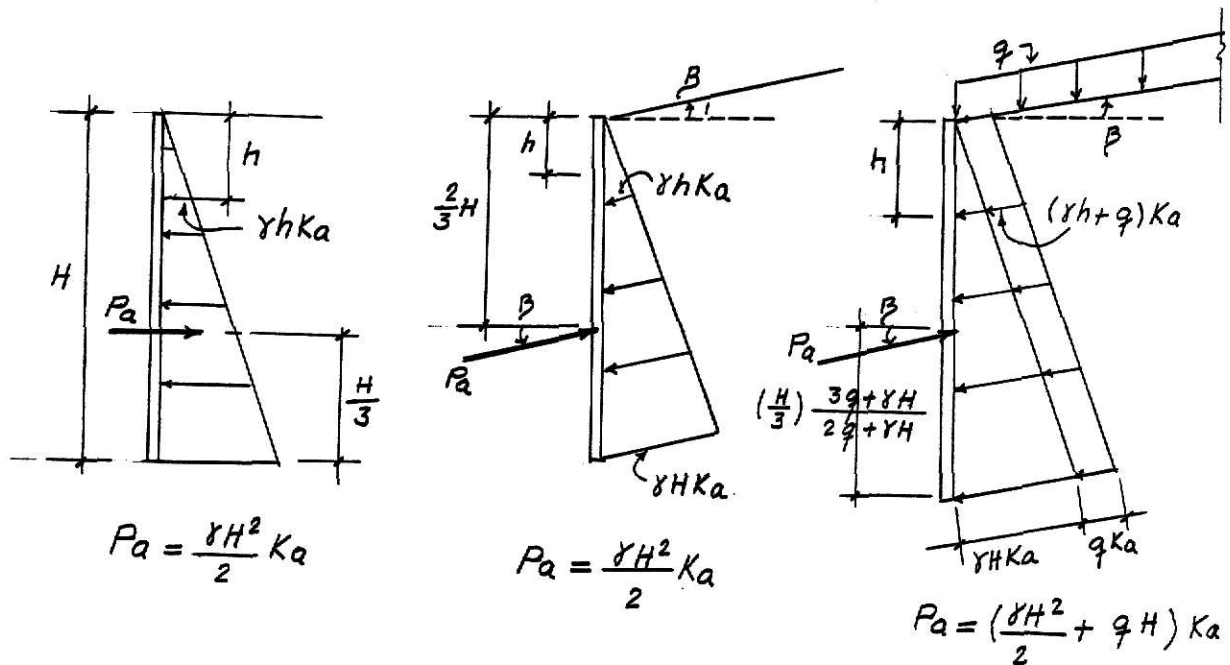
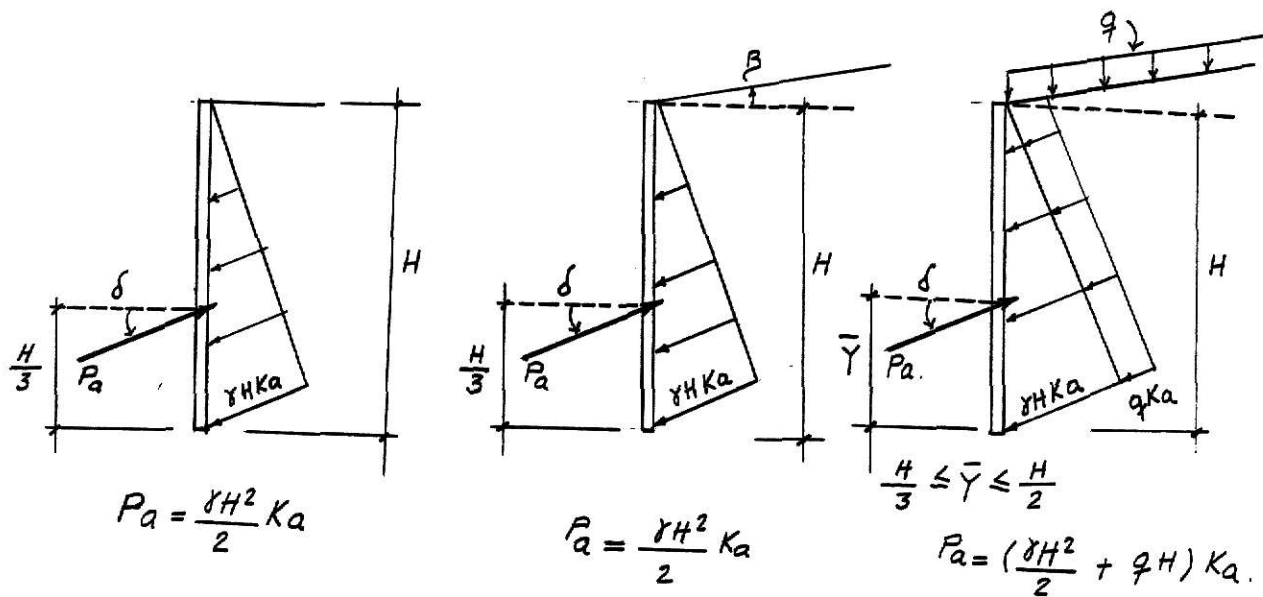Figure 9.7a.   Pressure Diagrams for Active Rankine Earth Pressure

Figure 9.7b.   Pressure Diagrams for Active Coulomb's Earth
Pressure.

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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 Rankind theory take into account the cohesion value (c) of the soil directly. The coefficients given include the friction angle ϕ and the weight of the backfill γ 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.

$$P_a = \frac{\gamma H^2}{2} Ka - 2CH \sqrt{Ka} \qquad (9.12)$$

$$Pp = \frac{\gamma H^2}{2} Kp + 2CH \sqrt{Kp} \qquad (9.13)$$

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).
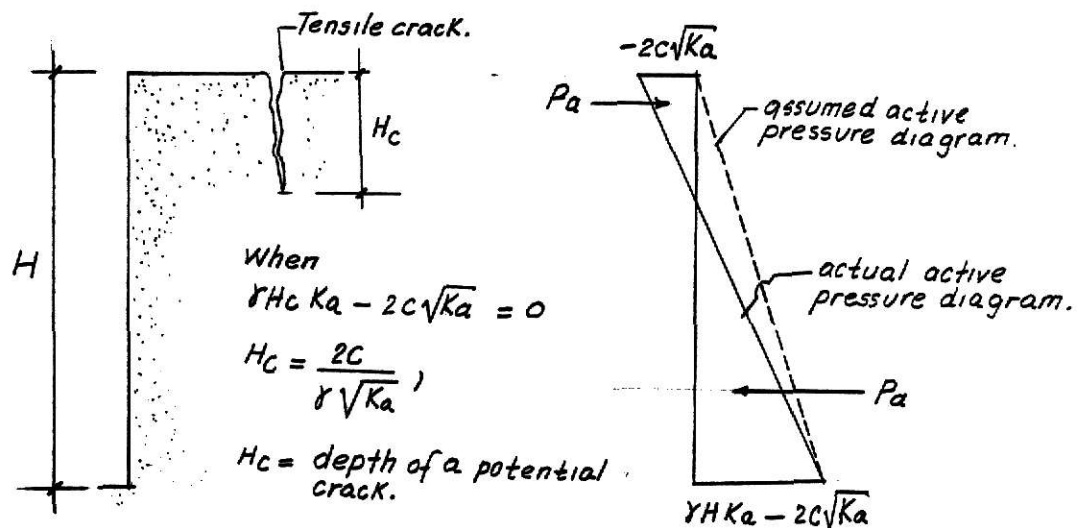
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Figure 9.8. Assumed and Actual Pressure Diagram for Soil with Cohesion.

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

$$\sigma_H = IMq \tag{9.14}$$

where

σ_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

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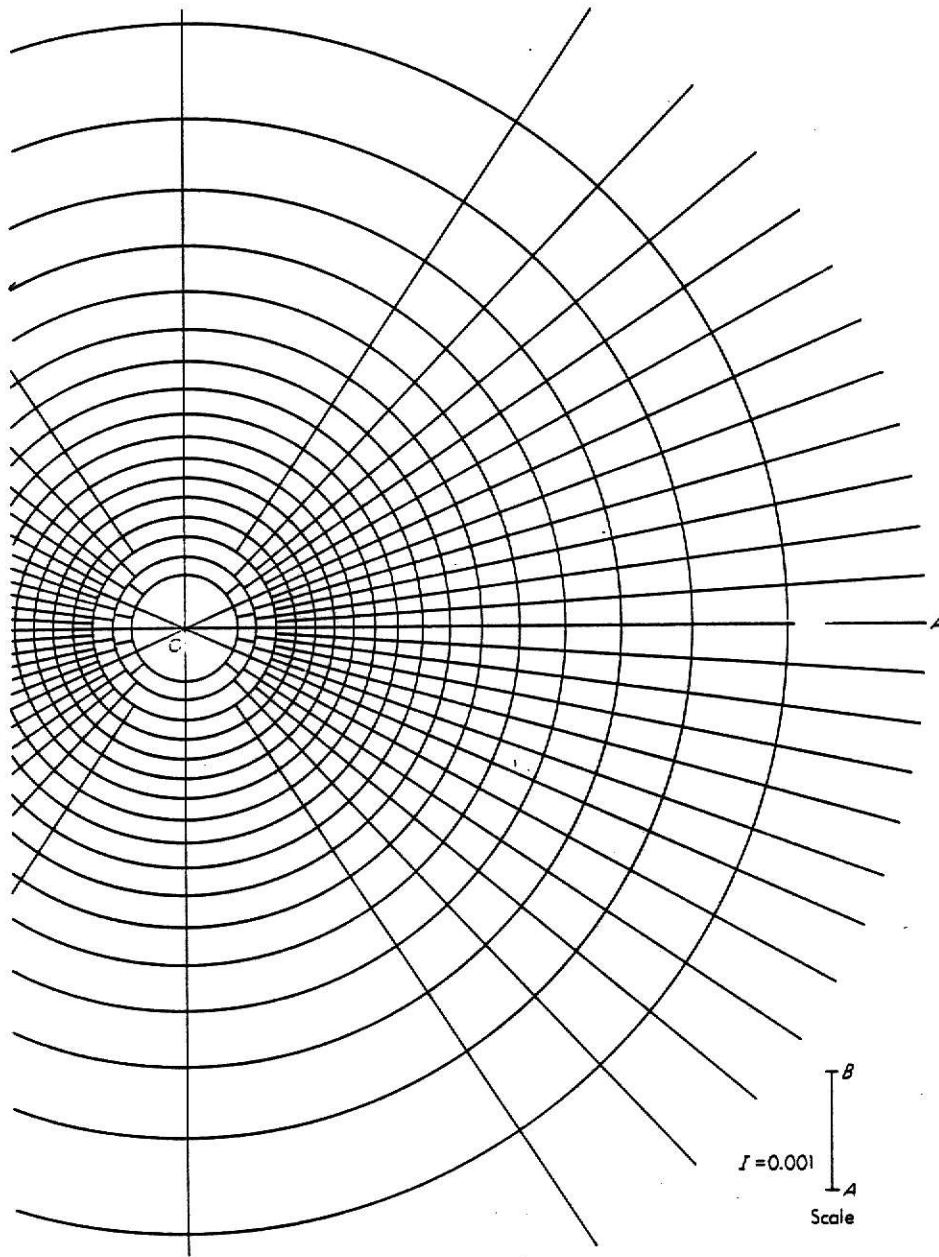
Figure 9.9.   Influence chart for computing lateral   pressure
at   point 0 for any type of loading in the influence field   (μ
= 0.5).   (Newark, 1942).

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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.

$$\text{(a)}$$

$m > 0.4$

$\sigma_h = \dfrac{1.77V}{H^2} \dfrac{m^2 n^2}{(m^2+n^2)^3}$

$m \le 0.4$

$\sigma_h = \dfrac{0.28V}{H^2} \dfrac{n^2}{(0.16+n^2)^3}$

for $m > 0.4$

$\sigma_h = \dfrac{4}{\pi} \dfrac{q}{H} \left(\dfrac{m^2 n}{(m^2+n^2)^2} \right)$

for $m \le 0.4$

$\sigma_h = \dfrac{q}{H} \dfrac{0.203 n}{(0.16+n^2)^2}$
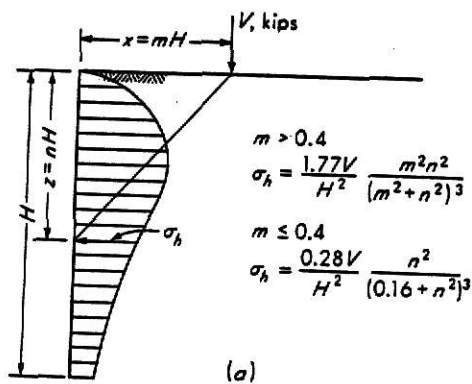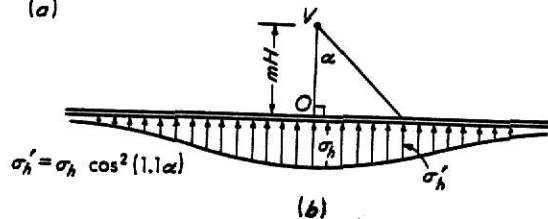
(C) Lateral pressure against rigid wall due to a line load and $\mu = 0.5$

$\sigma_h' = \sigma_h \cos^2(1.1\alpha)$

(b)

Fig 9.10. (a) Lateral pressure against rigid wall due to a point load and $\mu = 0.5$;

(b) Lateral pressure at points along the wall on each side of a perpendicular from the concentrated load.
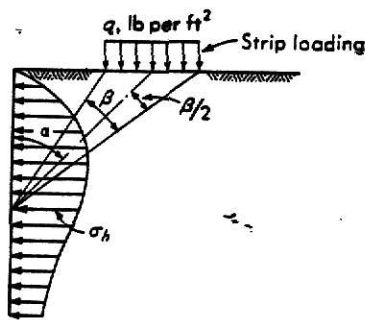
(d) Lateral pressure against rigid wall due to a strip load.
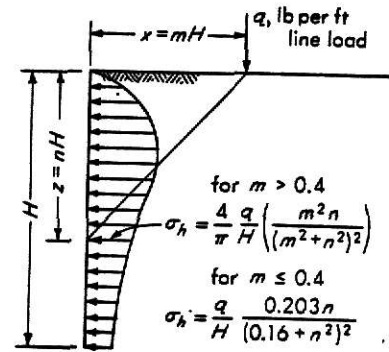Note that at a wall depth, this equation can compute (−) pressure for certain β and α combinations.

$\sigma_h = \dfrac{2q}{\pi} (\beta - \sin\beta \cos 2\alpha)$

$$P_i = \left(\dfrac{\sigma_i + \sigma_{i+1}}{2} \right) \dfrac{H}{n}$$

$$P_T = \sum_{i=1}^{n} P_i$$
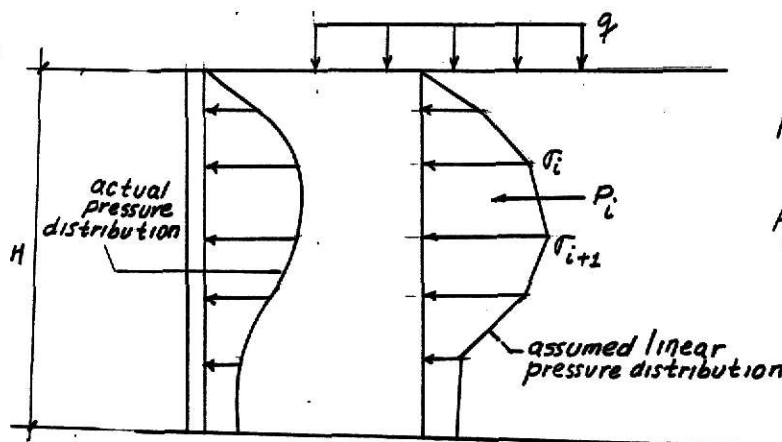
$n =$ number of segments.

$P_T =$ total force.

Figure 9.11.   Assumed linear pressure distribution.

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(a)

(b)

(c)
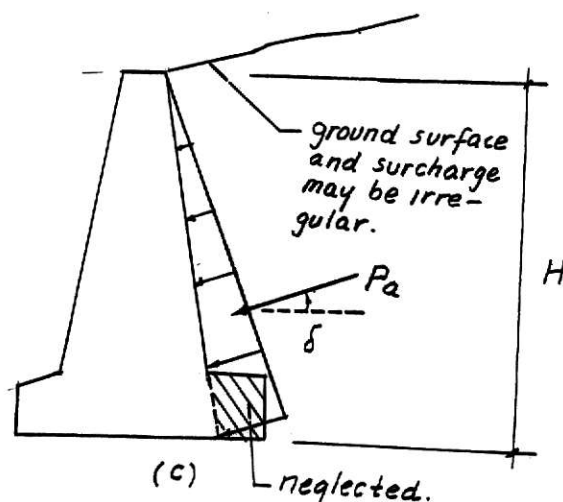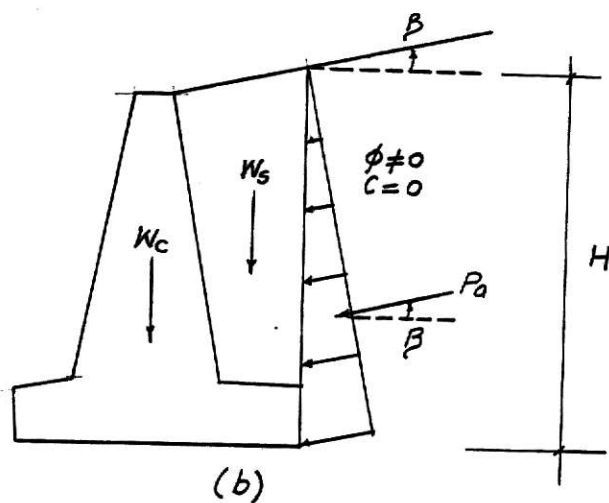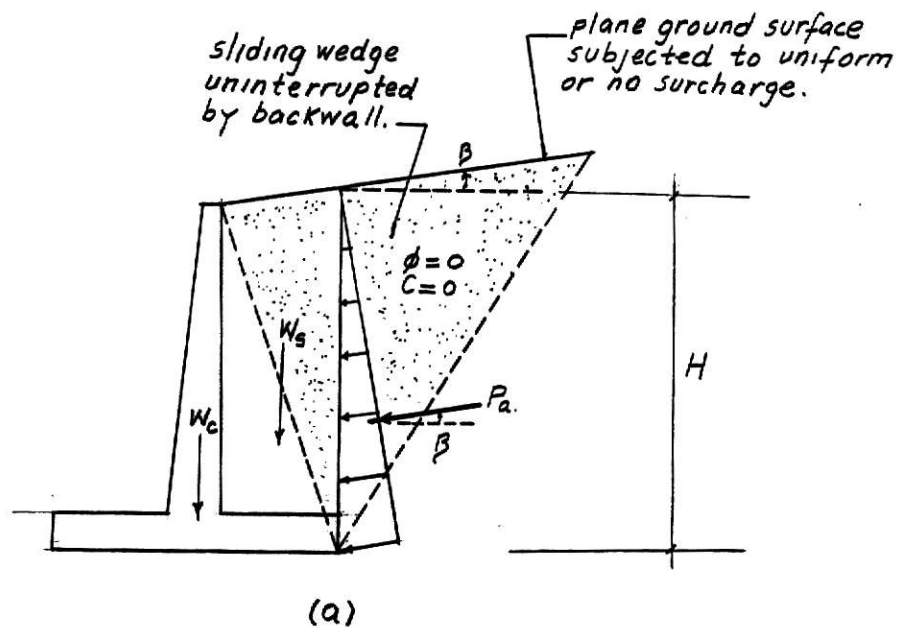
Figure 9.12. Requirements to use the Rankine Theory and Coulomb's Theory; (a) Cantilever wall, Rankine analysis; (b) Gravity wall, Rankine analysis; (c) Gravity wall, Coulomb analysis.

CHAPTER X

RETAINING WALLS

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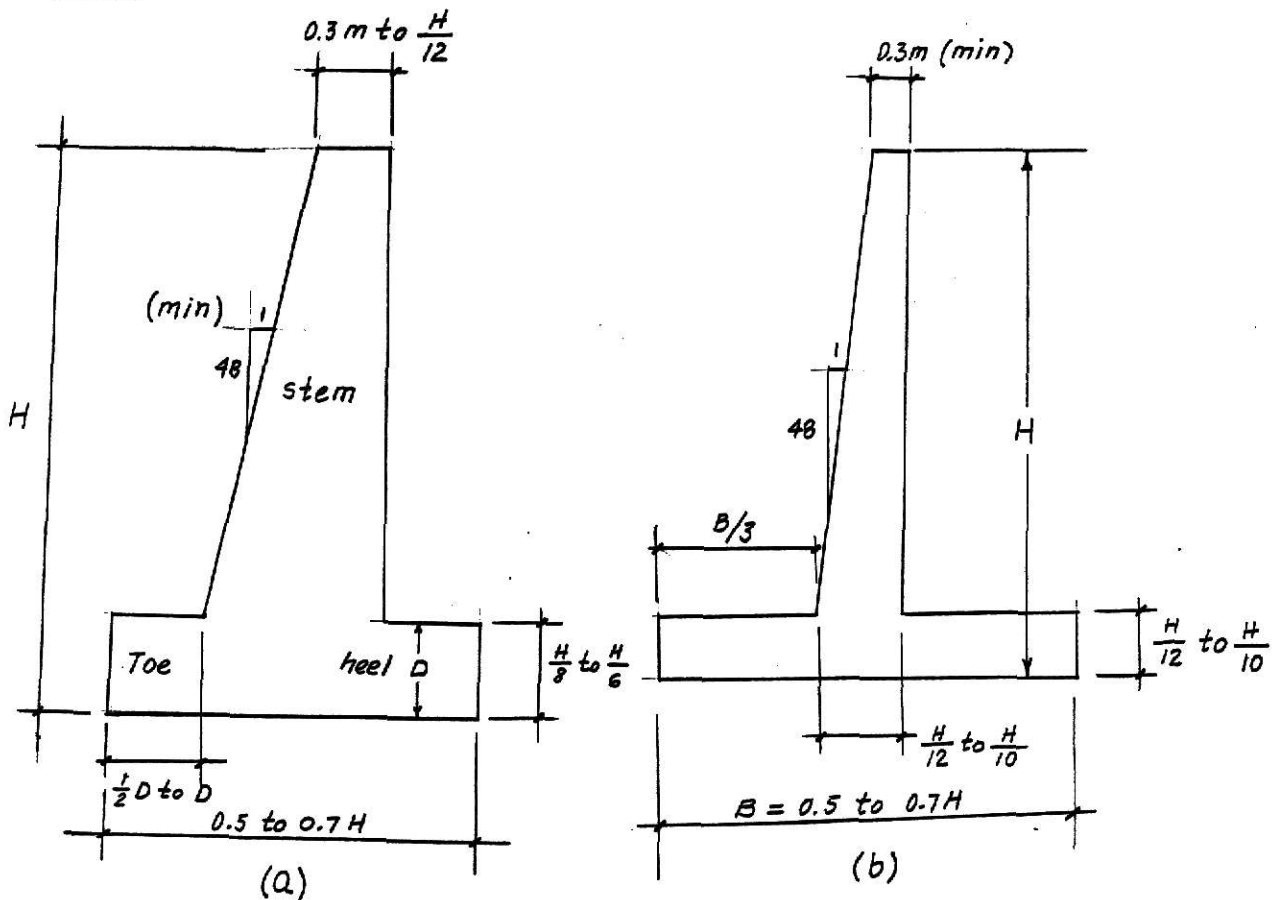
Figure 10.1. Approximate dimensions for (a) Gravity wall (b) Cantilever wall.

b. Retaining Wall Design

The following steps are suggested to design retaining walls:

1.   Obtain soil properties ϕ, c and γ for both backfill and base soil. Cohesionless backfill are recommended in the Rankine zone (see Fig. 10.2).
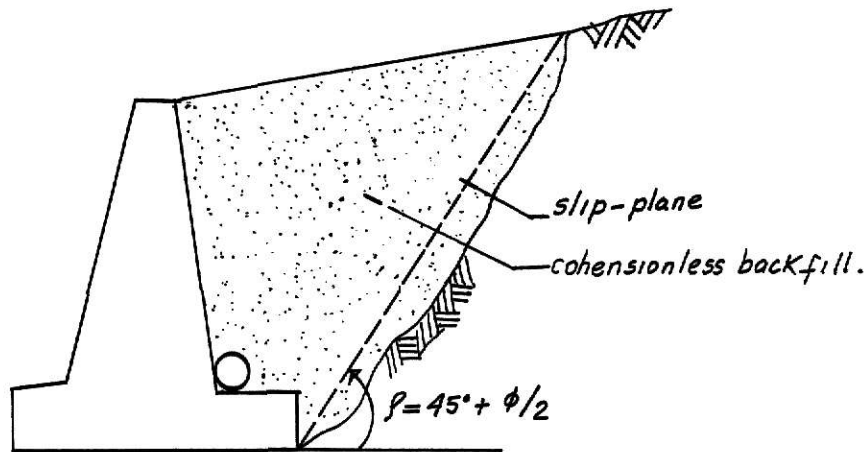
Figure 10.2. Backfill Rankine Zone.

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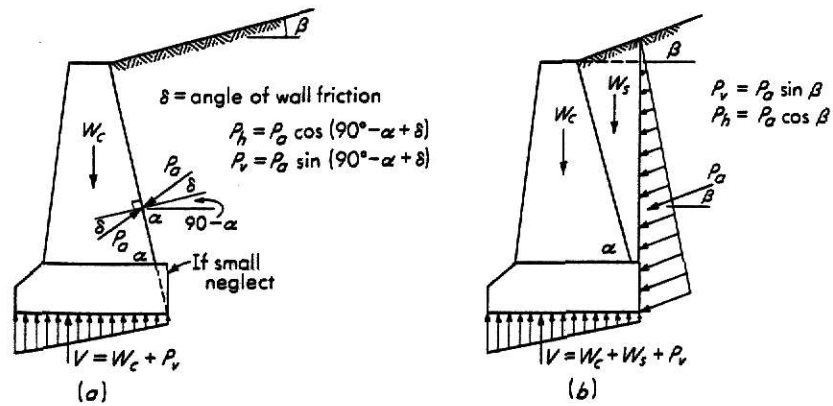
Figure 10.3. Forces on gravity wall (a) Coulomb analysis; (b) Rankine analysis.
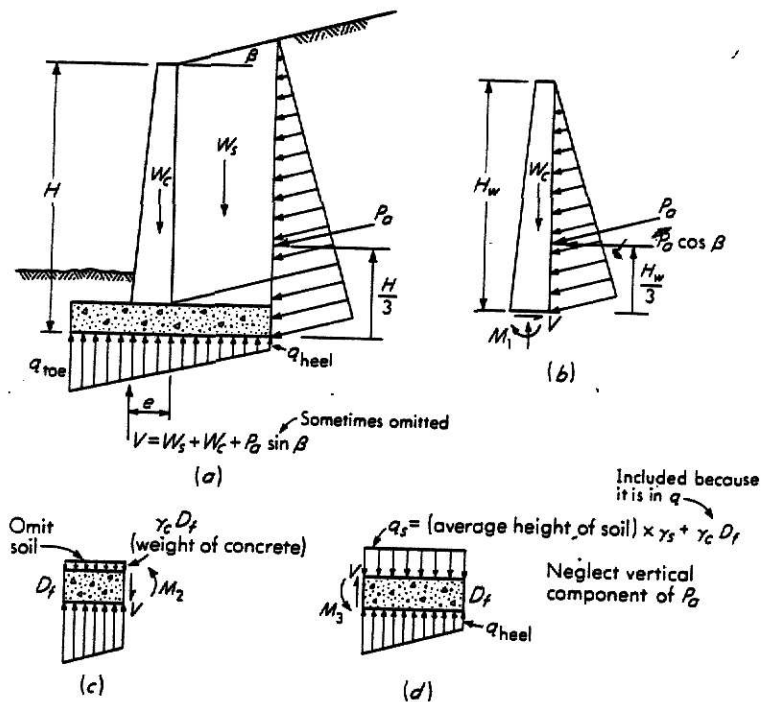
Figure 10.3a. Forces on cantilever wall. (a) Entire unit; free bodies for; (b) stem; (c) toe; (d) heel. Note that $M_1 + M_2 + M_3 = 0.0$.

The safety factor against overturning about the toe is given as

$$FS = \frac{\sum M_R}{\sum M_O} \quad \text{standard practice requires} \qquad \begin{matrix} 1.5 < FS < 2 \\ \text{overturning} \end{matrix}$$

where

$\sum M_O$ = sum of the moments of forces tending to overturn about point A ($Pa_{Horizontal}$).

$\sum M_R$ = sum of the moments of forces tending to resist overturning about point A (W_c, W_s, $Pa_{vertical}$).

5.2. Check for sliding failure along its base.

The safety factor against sliding is given as

$$FS_{sliding} = \frac{\sum F_R'}{\sum F_d} \quad \text{standard practice requires}$$

$$F.S_{sliding} \cong 2.5$$

where

$\sum F_R'$ = sum of the horizontal resisting forces

$\sum F_d$ = sum of the horizontal driving forces

or

$$FS_{sliding} = \frac{(\sum v) \tan \phi + BC}{Pa_{horizontal}}$$

where ϕ 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

$$q_{base} = \frac{V}{A} + \frac{(\sum M) x}{I_z} \leq q_{allowable}$$

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where $q_{allowable}$ may be obtained as indicated for continuous footings. The safety factor against bearing capacity is given as

$$FS \text{ (bearing capacity)} = \frac{q_u}{q_a} \text{ usually } \frac{q_u}{q_a} \simeq 3$$

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.

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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.
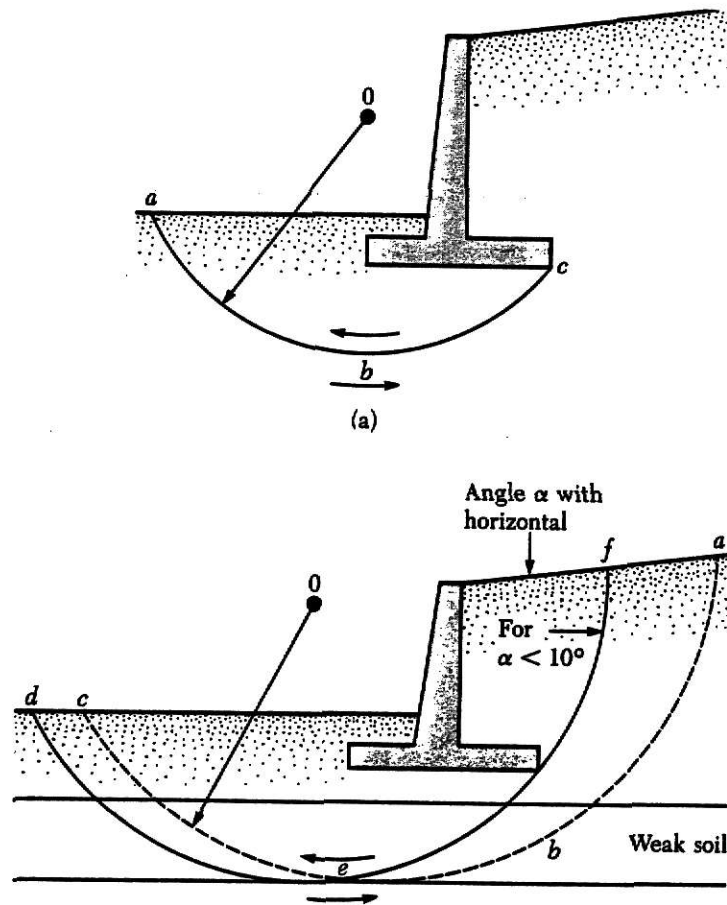
Figure 10.4. (a) Shallow shear failure; (b) deep shear failure.

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)

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

c.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).

c.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.)

Weepholes should be 100 mm or larger to avoid plugging. Note that the discharge is onto the toe where the soil pressure is largest.

Backfill with free draining soil

Granular material of size to avoid plugging weepholes

Drain pipe covered with granular material. Cut hole in counterfort if required

If weepholes are used with a counterfort wall at least one weephole should be located between counterforts

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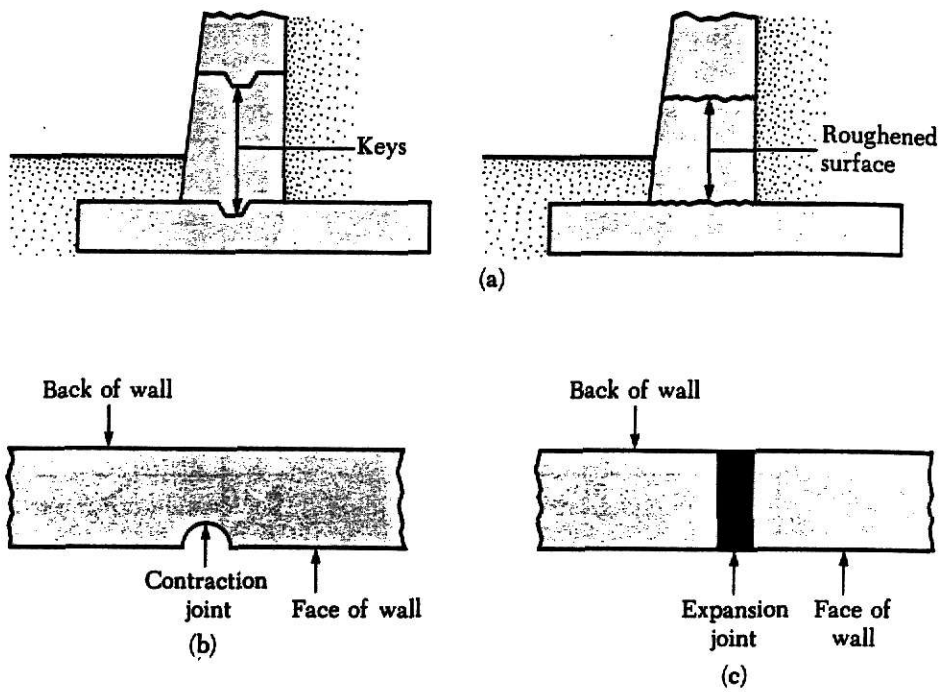
Figure 10.6.(a) Construction joints; (b) contraction joint; (c) expansion.

Example 10.1

Design a gravity retaining wall for the conditions given below.

Elevation

Plan View.

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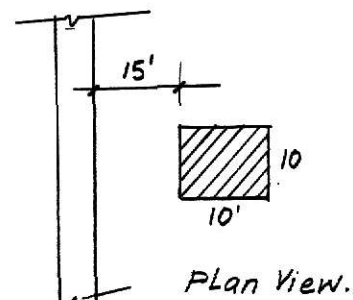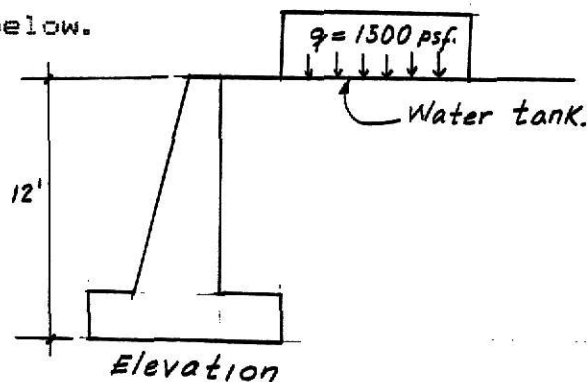
Soil Data

Backfill	Base soil
γ 110 pcf	100 pcf
ϕ 30°	15°
c = 0	c = 500 psf

$\gamma_c = 130$ pcf.

Tentative dimensions (use Fig. 10.1)
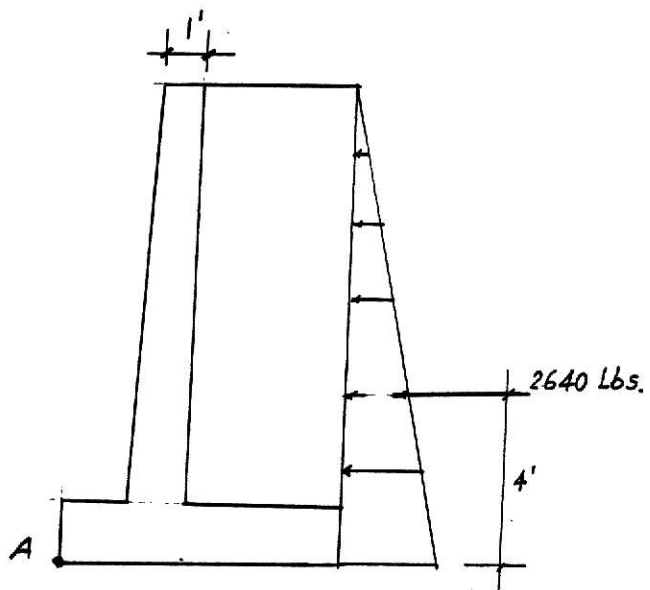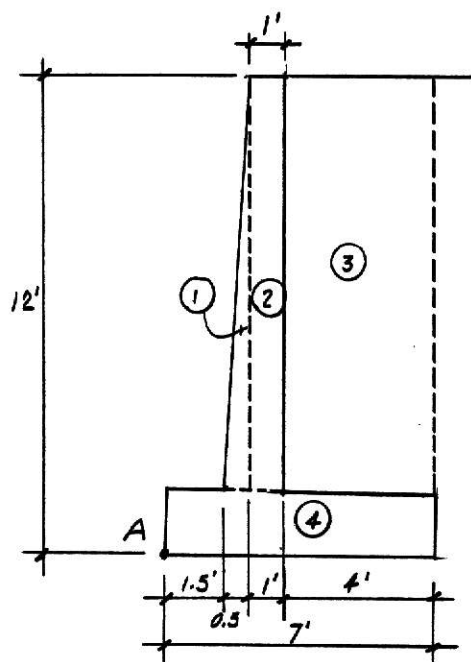
Earth Pressure   (use Rankine theory)

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

$Ka = 0.3333$ For $\phi = 30°$ $\beta = 0$ From Table 3.3

$P_a = 2640$ lbs/ft

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<u>Forces in Retaining Wall</u> (see Fig. 10.)

Part	Weight(pounds)	Arm to Point A (feet)	M(+) (pound-feet)
(1)	$\left(\dfrac{0.5 \times 10.5)}{2}\right) = 342$	1.83	+ 625.9
(2)	$130(1 \times 10.5) = 1365$	2.5	+ 3412.5
(3)	$110(4 \times 10.5) = 4620$	5	+ 23100
(4)	$130(1,5 \times 7) = 1365$	3.5	+ 4777.5
	$\sum V = 7692$		$\sum M_R$ + 31915.9

	P_a	2640	4	- 10560
				$= \sum M_{D_1}$

<u>Pressure due to surcharge</u> (use Newmark Chart, Fig. 9.9)

Elevation	Scale, feet	Relative dimensions in terms of AB		M
- 3'	AB = 3	Side B = $\dfrac{10}{3}$ = 3.33AB,	$\emptyset A = \dfrac{15}{3}$ = 5AB	4
- 6'	AB = 7	B = 1.67 AB	$\emptyset A = 2.54B$	28
- 9'	AB = 9	B = 1.1 AB	$\emptyset A = 1.67AB$	32
- 12'	AB = 12	B = 0.83AB	$\emptyset A = 1.25AB$	30

$\emptyset = 2IM\emptyset$ (for rigid walls)

$\emptyset = 2(0.001)M \times 1500$

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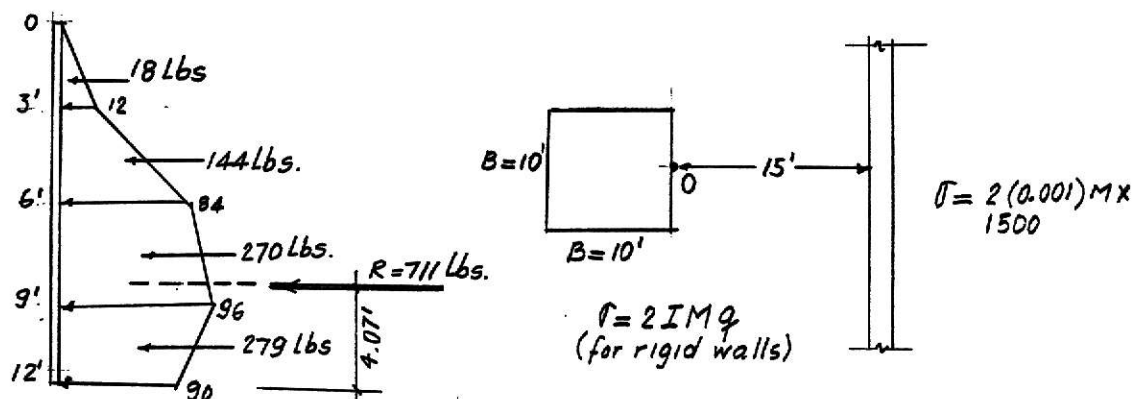
Assuming that the resultant forces except the 18#s force act in the middle of each pressure diagram, the total resultant is located at:

$$\bar{y} = \frac{18(10) + 144(7.5) + 270(4.5) + 279(1.5)}{18 + 144 + 270 + 279} = \frac{2893.5}{711}$$

$$= 4.07$$

$P_{surcharge}$ 711   arm 4.07   Moment - 2893.8 = $\sum M_{O2}$

$$\sum M_{O_T} = -10560 - 2893.8 = 13453.8 \curvearrowright$$

Stability Check

$$FS_{overturning} = \frac{\sum M_R}{\sum M_{O_T}} = \frac{31915.9}{13453.8} = 2.37 > 2 \quad ok$$

$$FS_{sliding} = \frac{\sum F_R'}{\sum F_d} = \frac{7692 \tan 15° + (7 \times 500)}{2640 + 711} = \frac{5561.27}{3351}$$

$$= 1.66 > 1.5 \quad ok$$

Bearing Capacity Check
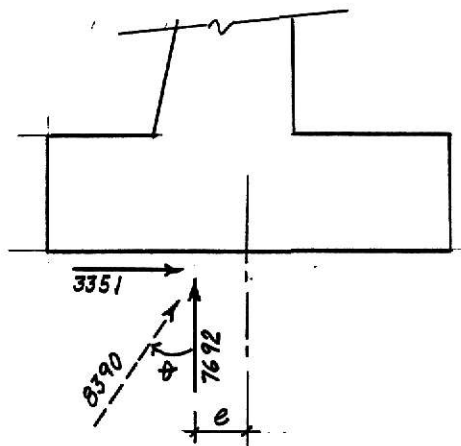
$$q_{toe} = \frac{\sum V}{B} \left(1 + \frac{6e}{B}\right)$$

$$q_{toe} = \frac{7692}{7}\left(1 + \frac{6(1.10)}{7}\right)$$

$$= 2134.92 \text{ psf}$$

$$q_{heel} = \frac{\sum V}{B} \left(1 - \frac{6e}{B}\right)$$

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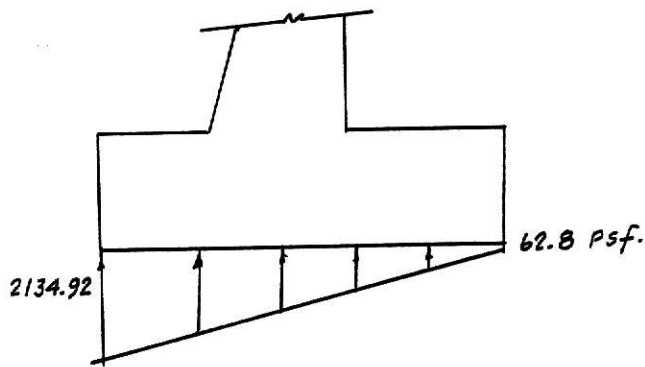
$$q_{heel} = \frac{7692}{7} \times$$

$$\left(1 - \frac{6(1.10)}{7}\right)$$

$$= 62.8 \text{ psf}$$

$$R = \sqrt{3351^2 + 7692^2} = 8390$$

$$\theta = \tan^{-1} \frac{3351}{7692} = 23.54°$$

$$e = 7/2 - \frac{31915.9 - 13453.8}{7692} = 1.10' < B/6 = 1.17 \text{ (only compression under base)}$$

$$e = B/2 - \frac{\sum M_R - \sum M_O}{\sum V}$$

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

$$+ 1/2 \, B^l \gamma N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}$$

$\phi = 15°$ $N_c = 11$

            $N_q = 3.9$ (from Table 4.2)

            $N_\gamma = 2.2$

$F_{cs} = F_{qs} = F_s =$ (for continuous footings)

$B' = B - 2e = 7 - 2(1.1) = 4.8$ (eccentrically loaded footings)

$$F_{cd} = 1 + 0.4 \left(\frac{D_f}{B}\right) \text{ for } D_f/B < 1$$

Assuming $D_f = 3'$ $\frac{D_f}{B} = 3/7 = 0.43 < 1$

$$F_{cd} = 1 + 0.4(3/7) = 1.17$$

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$$F_{qd} = 1 + 2 \tan \phi \, (1 - \sin \phi)^2 \frac{D_f}{B}, \quad \frac{D_f}{B} \leq 1$$

$$F_{qd} = 1 + 2 \tan 15° \, (1 - \sin 15°)^2 \; 3/7 = 1.29$$

$$F_d = 1$$

$$F_{ci} = F_{qi} = (1 - \frac{B°}{90°})^2$$

$$F_{ci} = F_{qi} = (2 - \frac{23.54°}{90°})^2 = 0.55$$

$$F_{\gamma i} = (1 - \frac{B°}{\phi})^2 = (1 - \frac{23.54°}{15°})^2 = 0.32$$

$$q_u = 500(11)(1.17)(0.55) + 100(3)(1.29) \; 0.55 \; (3.9)$$

$$+ \; 1/2 \; (4.8 \; 100 \; (1.1)(0.32)$$

$$q_u = 4453.85 \text{ psf}$$

$$q_a = \frac{4453.85}{3} = 1484.6 < 2134.92$$

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 stratums 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

$$P_{all} = Af_{all} \qquad (11.1)$$

where

P_{all} = allowable design load

A   = cross-sectional area of pile at cap

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f_{all} = allowable stress of the material being used (code value)

Below are presented formulas to obtain the allowable design load based on the structural strength.

Pile Type	P_{all}	
Timber piles	P_{all} = Ap fa	Ap: pile cross-sectional area fa: allowable stress for type of timber used
Precase concrete	P_{all} = Ag (0.33f'c − 0.27fe)	Aq: gross concrete area f'c: concrete compressive strength (35-55 mPa) fe: effective pretress after losses (5 mPa)
Cast in place piles	P_{all} = Ac fc + Asfs	Ac, Ac = area of, respectively, concrete and steel fc, fc = allowable material stresses
Steel piles	P_{all} = Ap fs	Ap = cross-sectional area of pile at cap fs: allowable steel stress

Table II.1

2. <u>Pile Capacity - Bearing Capacity Equations</u>

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 stratums penetrated by the pile, that is,

$$Q_u = Qp + Q_s \qquad\qquad (11.2)$$

where

Q_u = ultimate bearing capacity of the pile

Q_p = point-capacity contribution

Q_s = skin resistance contribution

The allowable bearing capacity is given by:

$$Q_{all} = \frac{Q_u + Q_s}{FS} \qquad (11.3)$$

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).
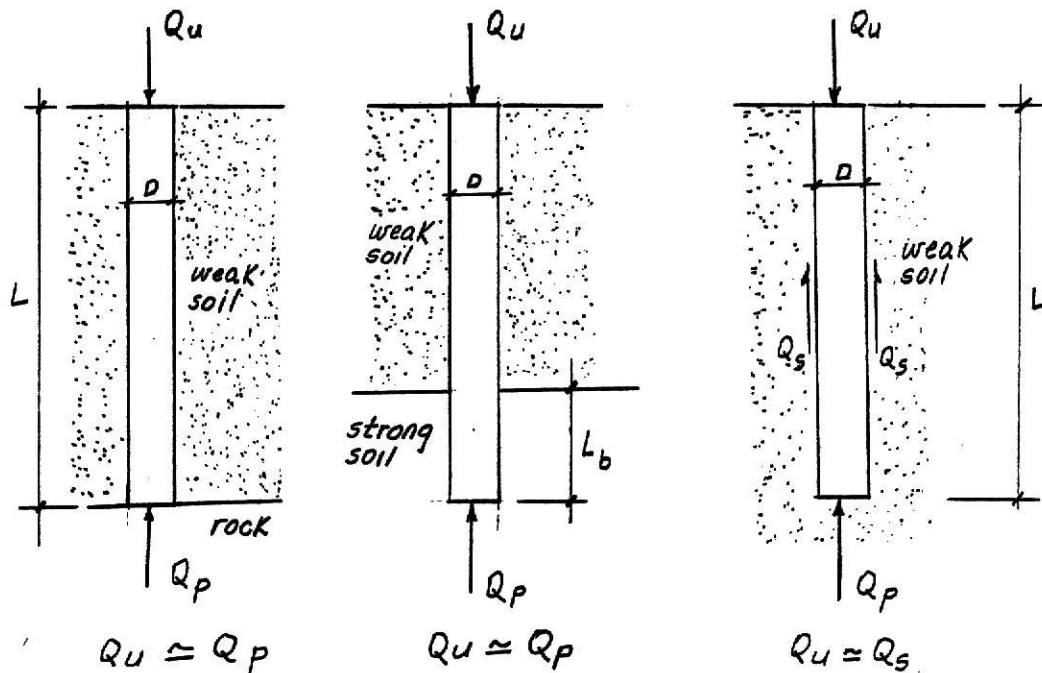
Figure 11.1.   (a) and; (b) Point bearing piles; (c) friction piles.

The expression to obtain Q_s and Q_p are presented below for cohesionless, cohesive and ϕ-c soils.

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Point Bearing Capacity

The ultimate point bearing capacity can be given as

$$q_p = A_p \, q_p \qquad (11.4)$$

where

$$q_p = cN_c^* + q'N_q^* \qquad (11.5)$$

and

A_p = area of pile at the pile tip

C = cohesion of the supporting material

q' = effective vertical stress at the level of pile tip

N_c^*, N_q^* = bearing capacity factors

q_p = 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 ϕ, determine $\left(\dfrac{L_b}{D}\right)_{crit}$ from Fig. 11.3.

b) Determine $\dfrac{L_b}{D}$, if $\dfrac{L_b}{D} > 0.5\left(\dfrac{L_b}{D}\right)$ and $\phi < 30°$ obtain

N_c^*, N_q^* from Fig. 11.2 taking the maximum values ($N'c$, Nq').

If $\dfrac{L_b}{D} < 0.5 \left(\dfrac{L_b}{D}\right)_{crit}$, then

$$N_c^* = N_c + (N'c - N_c)\left[\frac{\left(\dfrac{L_b}{D}\right)}{0.5\left(\dfrac{L_b}{D}\right)_{crit}}\right] \qquad (11.6)$$

$$N_a^* = N_q + (N_q^1 - N_q) \left[\frac{\left(\dfrac{L_b}{D}\right)}{0.5\left(\dfrac{L_b}{D}\right)_{crit}} \right] \tag{11.7}$$

c) $\phi \gtrless 30°$ take maximum values (N'_c, N_q)

For sand, $c = 0$ equation (11.5) becomes

$$Q_p = A_p \, q' \, N_q^* \; \underset{=}{<} \; A_p \, q_L \tag{11.8}$$

$$q_L = 50 \, n_q^* \, \tan \left(\frac{Kn}{m^2}\right) \overline{\text{(limiting point resistance)}} \tag{11.9}$$

For saturated clays, undrained conditions ($\phi = 0$).

$$Q_p = 9 \, C_u \, A_p \tag{11.10}$$

where

$q = N_c^*$ from Fig. 11.2

c_u = cohesion from unconfined compression test

The Vesic's method is used as follows:

$$Q_p = A_p(cN_c^* + \sigma'_o \, N_\sigma^*) \tag{11.11}$$

where

$$\sigma'_o = \left(\frac{1 + 2 K_o}{3}\right) q'$$

$K_o = 1 - \sin\phi$

N_c^*, N_σ^* = bearing capacity factors given in Table 11.1.

In Table 11.1,

$$I_{rr} = \frac{I_r}{1 + I_r \Delta} \tag{11.12}$$

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$$I_r = \text{rigidity index} = \frac{E_s p}{2(1 + \mu_s)(c + q' + \tan\phi)}$$

$$= \frac{G_s}{c + q' \tan\phi}$$

where

G_s is the shear modulus of the soil

and

Δ is the average of volumatic strain in the plastic

Zone below the pile point.

Where there is not volume change (dense sand or saturated

clay)

Δ = 0 and the equation 11.12 becomes $I_{rr} = I_r$.

Recommended values of I_r are shown below.

Soil	I_r
Sand (D_r = 0.5 - 0.8)	75-150
Silt and clay (drained conditions)	50-100
Clays (Undrained condition)	100-200

It can be noticed that the Vesic's method requires many work

in the laboratory to obtain ϕ, c, E_s, μ and γ.

Another method to obtain point bearing capacity is by

means of the standard penetration test. Meyerhof (1976)

suggests to obtain the value of Qp as:

$$Q_p = A_p (38N) \frac{L_b}{D} \le 380 \, N(A_p) \, KN \qquad (11.13)$$

where

N = statistical average of the SPT numbers in a zone about 8D above to 3D below the pile point.

$A_p, \dfrac{L_b}{D}$ as defined before (see Fig. 11.1)
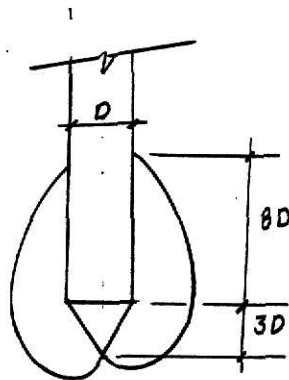
Figure 11.2

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:

$$Q_s = \sum p \, \Delta Lf \qquad (11.14)$$
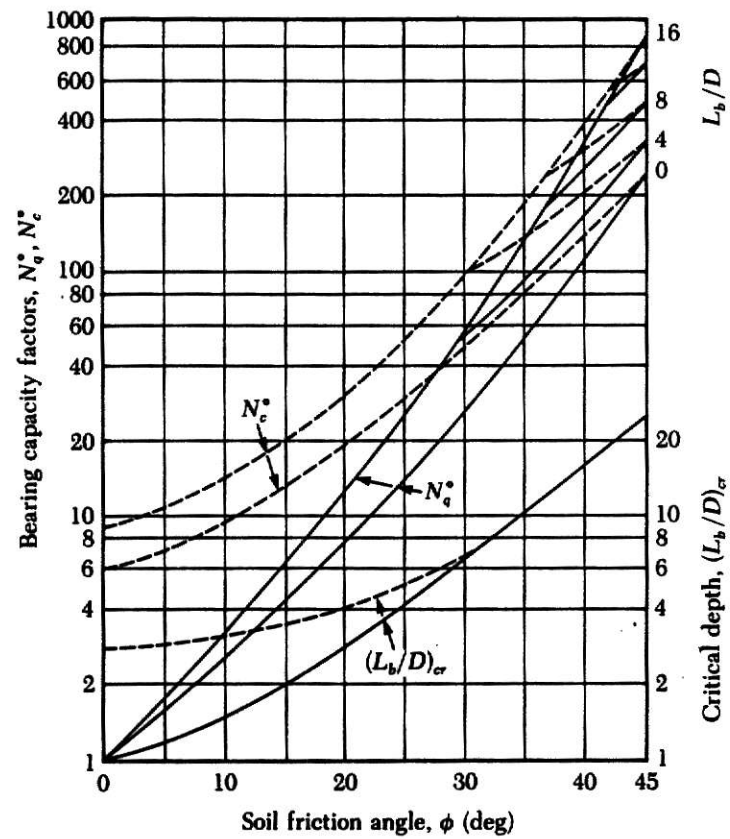
where

p = perimeter of pile section

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Figure 11.3 Critical embedment ratio and bearing capacity factors for various soil friction angles (after Meyerhof, 1976)

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φ	10	20	40	60	80	100	200	300	400	500
0	6.97	7.90	8.82	9.36	9.75	10.04	10.97	11.51	11.89	12.19
	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
1	7.34	8.37	9.42	10.04	10.49	10.83	11.92	12.57	13.03	13.39
	1.13	1.15	1.16	1.18	1.18	1.19	1.21	1.22	1.23	1.23
2	7.72	8.87	10.06	10.77	11.28	11.69	12.96	13.73	14.28	14.71
	1.27	1.31	1.35	1.38	1.39	1.41	1.45	1.48	1.50	1.51
3	8.12	9.40	10.74	11.55	12.14	12.61	14.10	15.00	15.66	16.18
	1.43	1.49	1.56	1.61	1.64	1.66	1.74	1.79	1.82	1.85
4	8.54	9.96	11.47	12.40	13.07	13.61	15.34	16.40	17.18	17.80
	1.60	1.70	1.80	1.87	1.91	1.95	2.07	2.15	2.20	2.24
5	8.99	10.56	12.25	13.30	14.07	14.69	16.69	17.94	18.86	19.59
	1.79	1.92	2.07	2.16	2.23	2.28	2.46	2.57	2.65	2.71
6	9.45	11.19	13.08	14.26	15.14	15.85	18.17	19.62	20.70	21.56
	1.99	2.18	2.37	2.50	2.59	2.67	2.91	3.06	3.18	3.27
7	9.94	11.85	13.96	15.30	16.30	17.10	19.77	21.46	22.71	23.73
	2.22	2.46	2.71	2.88	3.00	3.10	3.43	3.63	3.79	3.91
8	10.45	12.55	14.90	16.41	17.54	18.45	21.51	23.46	24.93	26.11
	2.47	2.76	3.09	3.31	3.46	3.59	4.02	4.30	4.50	4.67
9	10.99	13.29	15.91	17.59	18.87	19.90	23.39	25.64	27.35	28.73
	2.74	3.11	3.52	3.79	3.99	4.15	4.70	5.06	5.33	5.55
10	11.55	14.08	16.97	18.86	20.29	21.46	25.43	28.02	29.99	31.59
	3.04	3.48	3.99	4.32	4.58	4.78	5.48	5.94	6.29	6.57
11	12.14	14.90	18.10	20.20	21.81	23.13	27.64	30.61	32.87	34.73
	3.36	3.90	4.52	4.93	5.24	5.50	6.37	6.95	7.39	7.75
12	12.76	15.77	19.30	21.64	23.44	24.92	30.03	33.41	36.02	38.16
	3.71	4.35	5.10	5.60	5.98	6.30	7.38	8.10	8.66	9.11
13	13.41	16.69	20.57	23.17	25.18	26.84	32.60	36.46	39.44	41.89
	4.09	4.85	5.75	6.35	6.81	7.20	8.53	9.42	10.10	10.67
14	14.08	17.65	21.92	24.80	27.04	28.89	35.38	39.75	43.15	45.96
	4.51	5.40	6.47	7.18	7.74	8.22	9.82	10.91	11.76	12.46
15	14.79	18.66	23.35	26.53	29.02	31.08	38.37	43.32	47.18	50.39
	4.96	6.00	7.26	8.11	8.78	9.33	11.28	12.61	13.64	14.50
16	15.53	19.73	24.86	28.37	31.13	33.43	41.58	47.17	51.55	55.20
	5.45	6.66	8.13	9.14	9.93	10.58	12.92	14.53	15.78	16.83
17	16.30	20.85	26.46	30.33	33.37	35.92	45.04	51.32	56.27	60.42
	5.98	7.37	9.09	10.27	11.20	11.98	14.77	16.69	18.20	19.47
18	17.11	22.03	28.15	32.40	35.76	38.59	48.74	55.80	61.38	66.07
	6.56	8.16	10.15	11.53	12.62	13.54	16.84	19.13	20.94	22.47
19	17.95	23.26	29.93	34.59	38.30	41.42	52.71	60.61	66.89	72.18
	7.18	9.01	11.31	12.91	14.19	15.26	19.15	21.87	24.03	25.85
20	18.83	24.56	31.81	36.92	40.99	44.43	56.97	65.79	72.82	78.78
	7.85	9.94	12.58	14.44	15.92	17.17	21.73	24.94	27.51	29.67
21	19.75	25.92	33.80	39.38	43.85	47.64	61.51	71.34	79.22	85.90
	8.58	10.95	13.97	16.12	17.83	19.29	24.61	28.39	31.41	33.97
22	20.71	27.35	35.89	41.98	46.88	51.04	66.37	77.30	86.09	93.57
	9.37	12.05	15.50	17.96	19.94	21.62	27.82	32.23	35.78	38.81
23	21.71	28.84	38.09	44.73	50.08	54.66	71.56	83.68	93.47	101.83
	10.21	13.24	17.17	19.99	22.26	24.20	31.37	36.52	40.68	44.22
24	22.75	30.41	40.41	47.63	53.48	58.49	77.09	90.51	101.39	110.70
	11.13	14.54	18.99	22.21	24.81	27.04	35.32	41.30	46.14	50.29
25	23.84	32.05	42.85	50.69	57.07	62.54	82.98	97.81	109.88	120.23
	12.12	15.95	20.98	24.64	27.61	30.16	39.70	46.61	52.24	57.06
26	24.98	33.77	45.42	53.93	60.87	66.84	89.25	105.61	118.96	130.44
	13.18	17.47	23.15	27.30	30.69	33.60	44.53	52.51	59.02	64.62
27	26.16	35.57	48.13	57.34	64.88	71.39	95.02	113.92	128.67	141.39
	14.33	19.12	25.52	30.21	34.06	37.37	49.88	59.05	66.56	73.04
28	27.40	37.45	50.96	60.93	69.12	76.20	103.01	122.79	139.04	153.10
	15.57	20.91	28.10	33.40	37.75	41.51	55.77	66.29	74.93	82.40
29	28.69	39.42	53.95	64.71	73.58	81.28	110.54	132.23	150.11	165.61
	16.90	22.85	30.90	36.87	41.79	46.05	62.27	74.30	84.21	92.80
30	30.03	41.49	57.08	68.69	78.30	86.64	118.53	142.27	161.91	178.96
	18.24	24.95	33.95	40.66	46.21	51.02	69.43	83.14	94.48	104.33
31	31.43	43.64	60.37	72.88	83.27	92.31	126.99	152.95	174.49	193.23
	19.86	27.22	37.27	44.79	51.03	56.46	77.31	92.90	105.84	117.11
32	32.89	45.90	63.82	77.29	88.50	98.28	135.96	164.29	187.87	208.43
	21.55	29.68	40.88	49.30	56.30	62.41	85.96	103.66	118.39	131.24
33	34.41	48.26	67.44	81.92	94.01	104.58	145.46	176.33	202.09	224.62
	23.34	32.34	44.80	54.20	62.05	68.92	95.46	115.51	132.24	146.87
34	35.99	50.72	71.24	86.80	99.82	111.22	155.51	189.11	217.21	241.84
	25.28	35.21	49.05	59.54	68.33	76.02	105.90	128.55	147.51	164.12
35	37.65	53.30	75.22	91.91	105.92	118.22	166.14	202.64	233.27	260.15
	27.36	38.32	53.67	65.36	75.17	83.78	117.33	142.89	164.33	183.16
36	39.37	55.99	79.39	97.29	112.34	125.59	177.38	216.98	250.30	279.60
	29.60	41.68	58.68	71.69	82.62	92.24	129.87	158.65	182.85	204.14
37	41.17	58.81	83.77	102.94	119.10	133.34	189.25	232.17	268.36	300.26
	32.02	45.31	64.13	78.57	90.75	101.48	143.61	175.95	203.23	227.26
38	43.04	61.75	88.36	108.86	126.20	141.50	201.78	248.23	287.50	322.17
	34.63	49.24	70.03	86.05	99.60	111.56	158.65	194.94	225.62	252.71
39	44.99	64.83	93.17	115.09	133.66	150.09	215.01	265.23	307.78	345.41
	37.44	53.50	76.45	94.20	109.24	122.54	175.11	215.78	250.23	280.71
40	47.03	68.04	98.21	121.62	141.51	159.13	228.97	283.19	329.24	370.04
	40.47	58.10	83.40	103.05	119.74	134.52	193.13	238.62	277.26	311.50
41	49.16	71.41	103.49	128.48	149.75	168.63	243.69	302.17	351.95	396.12
	43.74	63.07	90.96	112.68	131.18	147.59	212.84	263.67	306.94	345.34
42	51.38	74.92	109.02	135.68	158.41	178.62	259.22	322.22	375.97	423.74
	47.27	68.46	99.16	123.16	143.64	161.83	234.40	291.13	339.52	382.53
43	53.70	78.60	114.82	143.23	167.51	189.13	275.59	343.40	401.36	452.96
	51.08	74.30	108.08	134.56	157.21	177.36	257.99	321.22	375.28	423.39
44	56.13	82.45	120.91	151.16	177.07	200.17	292.85	365.75	428.21	483.88
	55.20	80.62	117.76	146.97	172.00	194.31	283.80	354.20	414.51	468.26
45	58.66	86.48	127.28	159.48	187.12	211.79	311.04	389.35	456.57	516.58
	59.66	87.48	128.28	160.48	188.12	212.79	312.03	390.35	457.57	517.58
46	61.30	90.70	133.97	168.22	197.67	224.00	330.20	414.26	486.54	551.16
	64.48	94.92	139.73	175.20	205.70	232.96	342.94	429.98	504.82	571.74
47	64.07	95.12	140.99	177.40	208.77	236.85	350.41	440.54	518.20	587.72
	69.71	103.00	152.19	191.24	224.88	254.99	376.77	473.42	556.70	631.25
48	66.97	99.75	148.35	187.04	220.43	250.36	371.70	468.28	551.64	626.36
	75.38	111.78	165.76	208.73	245.81	279.06	413.82	521.08	613.65	696.64
49	70.01	104.60	156.09	197.17	232.70	264.58	394.15	497.56	586.96	667.21
	81.54	121.33	180.56	227.82	268.69	305.37	454.42	573.38	676.22	768.53
50	73.19	109.70	164.21	207.83	245.60	279.55	417.82	528.46	624.28	710.39
	88.23	131.73	196.70	248.66	293.70	334.15	498.94	630.80	744.99	847.61

Table 11.2. Bearing capacity factors for deep foundations, N_c and N_q.

L = pile length

f = unit friction resistance at any depth

For sand "f" is expressed as

$$f = k \; \sigma_v' \; \tan \delta \qquad (11.15)$$

where

K = earth pressure coefficient

σ_v' = effective vertical stress at dept under study

δ = soil-pile friction angle

K varies with depth, the following values are recommended.

K	Pile installation
1. $1 - \sin \phi = K_O$	bored piles
2. K_O - lower limit 1.4 K_O - upper limit	Low-displacement drivven piles
3. K_O - lower limit 1.8 K_O upper limit	High displacement driven piles

---

Taken "K" as a constant and "δ" as a constant which ranges of 0.5 ϕ to 0.8 ϕ, the only term in equation 11.15 that varies with depth is σ'_v. In sand, that variation is from 15 to 20 times the pile diameter, therefore, knowing that f is a function of σ_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,

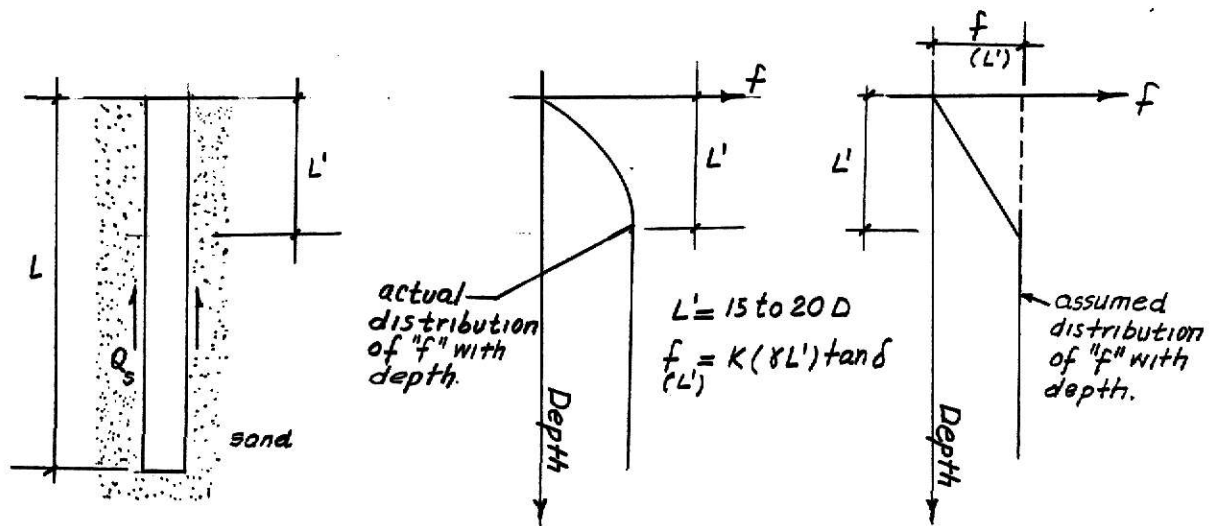$$Q_s = p \; L' \; f_{av_{0-15D}} + p \; (L - L') \; f_{av_{15D \; to \; L}} \qquad (11.16)$$

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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 λ method, the β method and, the α 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".

λ Method: Proposed by Vijayvergiya and Focht (1972) obtains the value of f as:

$$fs = \lambda(\bar{\sigma}_v + 2 \, c_u) \tag{11.17}$$

where

λ = value obtained from Fig. 11.5

$\bar{\sigma}_v$ = average effective vertical stress for the entire embedment length

c_u = average undrained shear strength ($\phi = 0$)

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$\bar{\sigma}_v$ and c_u can be obtained as shown in Fig. 11.6 for layered soils.

$\propto$ Method: Proposed by Tomlinson (1971) suggests to compute the unit skin resistance as:

$$f = \propto c_u \qquad\qquad (11.18)$$

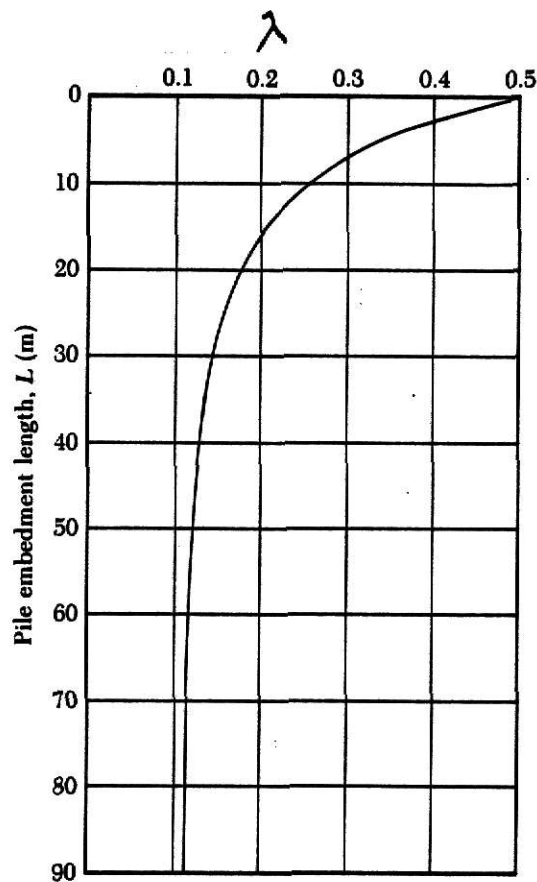where   is an empirical factor that can be obtained from Fig.

Figure 11.5.   Variation of λ with pile embedment length (redrawn after McClelland, 1974).
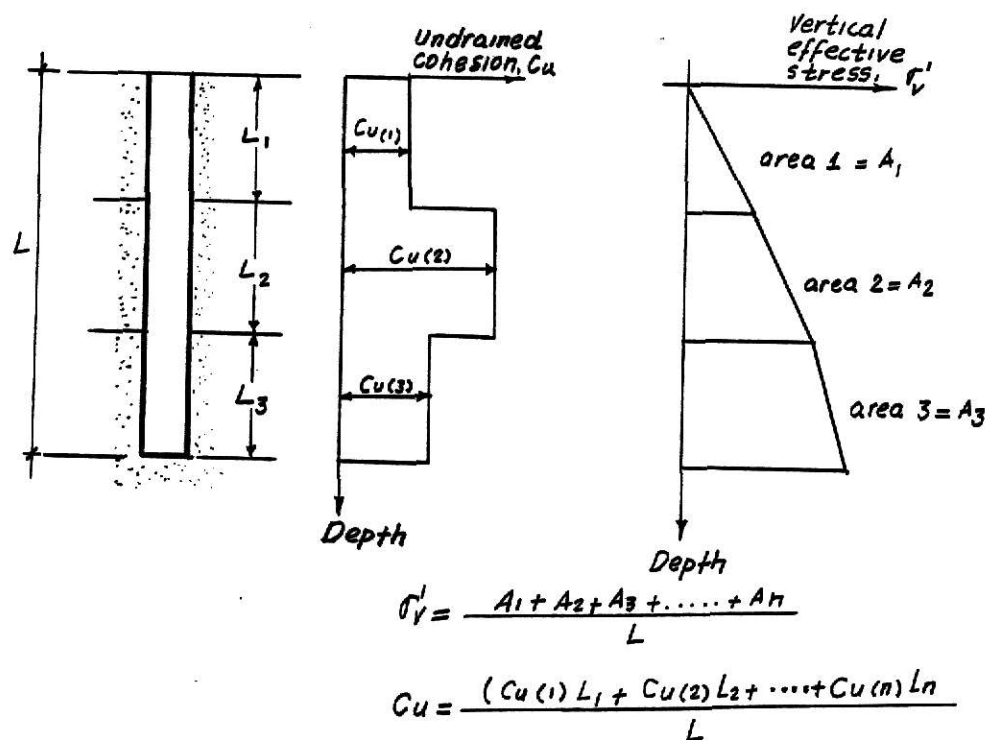
$$\sigma_v^{\prime} = \frac{A_1 + A_2 + A_3 + \ldots + A_n}{L}$$

$$Cu = \frac{(Cu_{(1)} L_1 + Cu_{(2)} L_2 + \ldots + Cu_{(n)} L_n}{L}$$

Figure 11.6 Application of λ method in layered soils.

11.7 and c_u as defined before.   The skin resistance is given by

$$Q_s = \sum \alpha c_u \Delta L \, (p) \qquad\qquad (11.19)$$

β Method: This   method   expresses   the   value   of   unit frictional skin as

$$f = \beta \sigma^{\prime}_v \qquad\qquad (11.20)$$

where

$\qquad \beta = K \tan \phi_R$

ϕ_R = drained friction angle of remodeled clay

K   = earth pressure coefficient

$\sigma^{\prime}_v$ = vertical effective stress

The value of K can be taken as K_o or

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$$K = 1 - \sin \phi_R \quad \text{(normally consolidated clay)} \qquad (11.21)$$

$$K = (1 - \sin \phi_R) \sqrt{\text{OCR}} \quad \text{(overconsolidated clay)} \quad (11.22)$$

plugging the values of K into equation (11.20), is obtained

$$f = (1 - \sin \phi_R) \tan \phi_R \sigma'_v \quad \text{(normally con-} \qquad (11.24) \atop \text{solidated clays)}$$

$$f = (1 - \sin \phi_R) \tan \phi_R \sqrt{\text{OCR}} \; \sigma'_v \quad \text{(over-} \qquad (11.25) \atop \text{consolidated clays)}$$

Again, Q_S is given by

$$Q_S = \int f \cdot P \, \Delta L$$

Goodman (1980) proposed the following equation for piles resting on rocks:
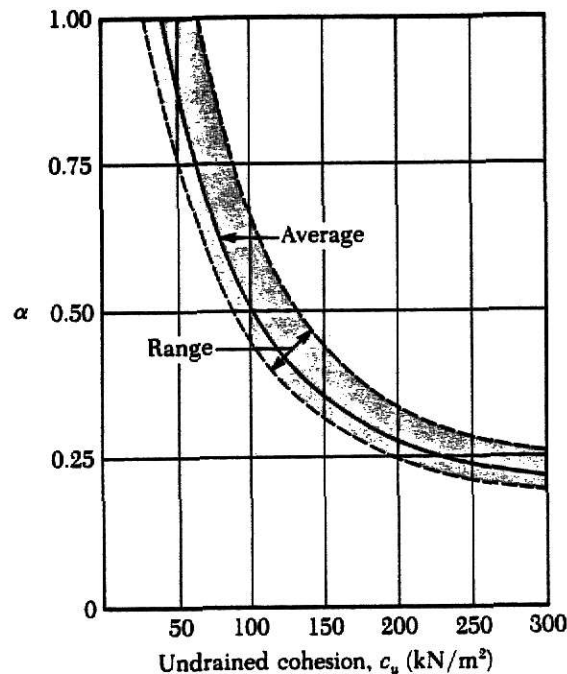
$$qp = q_u (N\phi + 1) \qquad (11.26)$$

where

Figure 11.7. Variation of α with C_u.

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$$N = \tan^2(45 + \frac{\phi}{2})$$

q_u = unconfined compression strength of rock

ϕ = drained angle of friction

then, the allowable point capacity is expressed as:

$$Qp_{allowable,} = \frac{[q_u(N_\phi + 1)]Ap}{FS}, \quad FS \geq 3 \tag{11.27}$$

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,

$$n = \frac{Qg_{(u)}}{\sum Q_u} \qquad (11.28)$$

where

n = pile group efficiency

$Qg_{(u)}$ = 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 expresscan be used (see Fig. 11.10):

$$n = \frac{2(m + n - 2)s + 4D}{p \ m \ n} \qquad where, \qquad (11.29)$$

number of piles = mxn

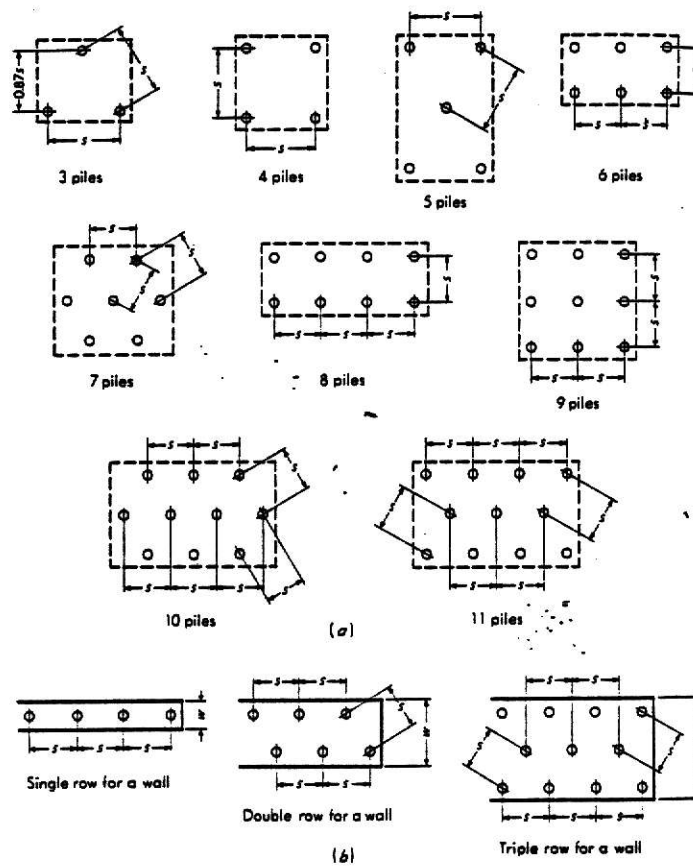p = perimeter of cross section of each pile

D = pile diameter

s = pile spacing

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Figure 11.8. Typical pile group patterns; (a) for single footings; (b) for foundation walls.
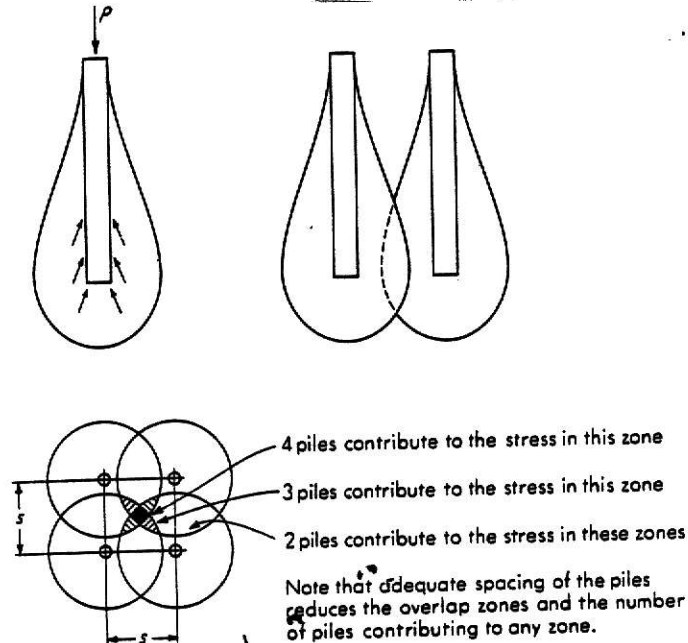

4 piles contribute to the stress in this zone

3 piles contribute to the stress in this zone

2 piles contribute to the stress in these zones

Note that adequate spacing of the piles reduces the overlap zones and the number of piles contributing to any zone.

Figure 11.9. Stresses surrounding a friction pile and the summing effects.
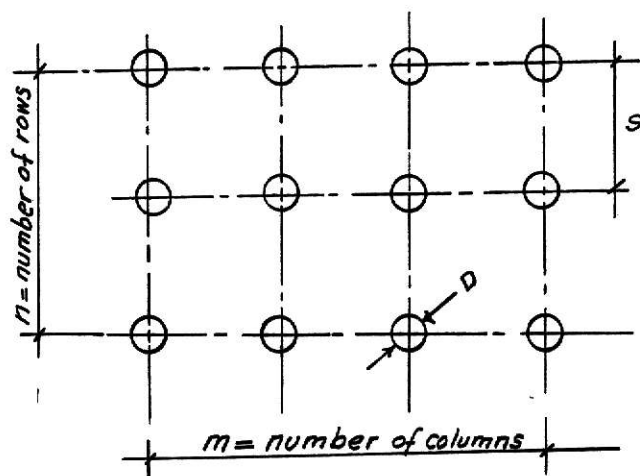
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Figure 11.10.   Pile group.

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

$$Q_u = p L f_{av}$$

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)

$$Q_p = \quad C_{u_{(P)}} \quad A_p \ , \quad C_{u_{(P)}} : \text{cohesion at pile tip}$$

Equation (11.19)

$$Q_s = \int \alpha p \; c_u \; \Delta L$$

then

$$Q_u = mn(9 \; c_{u_{(P)}} \; Ap + [\int \alpha \; p \; c_u \; \Delta L]) \qquad (11.30)$$

2.   Determine pile bearing as if the piles behave as a group, that is,

$$Qp = Lg \; Bg \; C_{u_{(P)}} \; N_C*$$

where the value N_C* can be obtained from Fig. 11.11

and

$$Lg = (m - 1)s + D$$
$$Bg = (n - 1)s + D$$

and

$$Q_s = \int Pg \; c_u \; L = \int 2(Lg + Bg)c_u \; \Delta L$$

then

$$Qg_{(u)} = Lg \; Bg \; C_{u_{(u)}} \; N_C* + \int 2(Lg + Bg)c_u \; \Delta L \qquad (11.31)$$

3.   Compare the two values $Qg_{(u)}$ and $\int Q_u$ 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, $Qg_{(u)} = \int Q_u$ 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
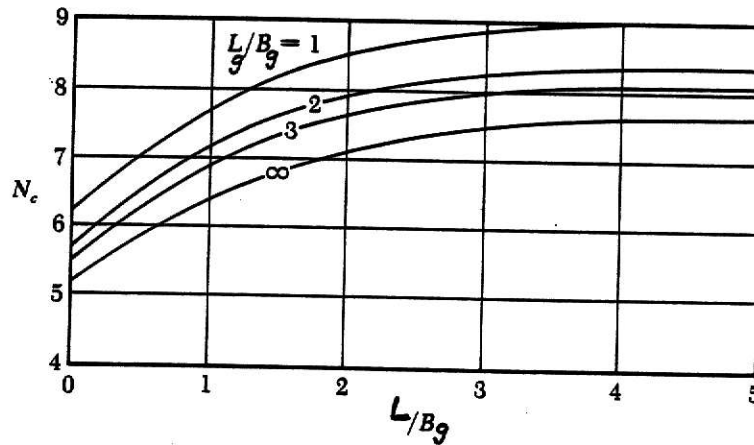
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Figure 11.11. Variation of N, with L/B and H/B-based on Bjerrum and Eide's equation.

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

$Q_T = Q_T - \gamma$ 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.
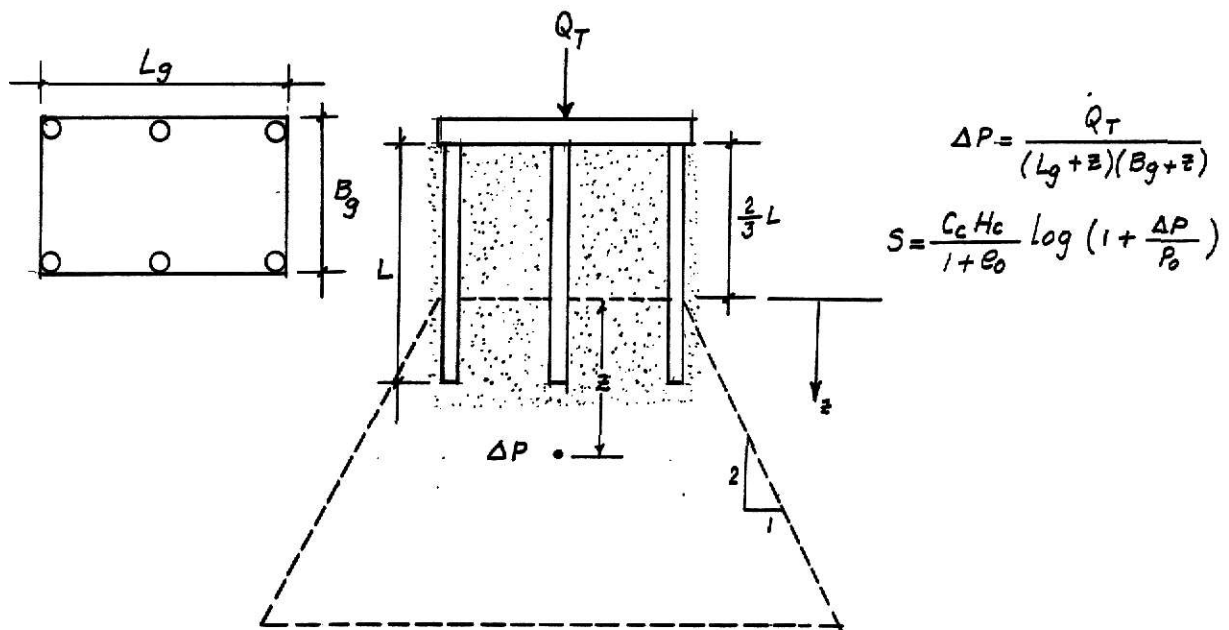
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Figure 11.12. Induced-stresses Pile Group.

The elastic settlement for a pile group may be obtained by the equation proposed by Vesic (1969).

$$Sg_{(e)} = \sqrt{\frac{Bg}{D\, S_e}} \qquad\qquad (11.33)$$

where

$Sg_{(e)}$ = elastic settlement of a group of piles

Bg = 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

$$S_e = \frac{(Q_{wp} + \xi\, Q_{ws})L}{A_p\, E_p} + \frac{Q_{wp}}{A_p}\, \frac{D}{E_s}\, (1 - \mu_s^2)\, I_{wp} +$$

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$$\left(\frac{Q_{ws}}{P\,L}\right)\frac{D}{E_s}\,(1-\mu_s^{2})\,I_{ws} \qquad\qquad (11.33)$$

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

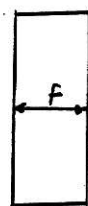L = length of pile

E_p = Young's modulus of the pile material

E_s = Young's modulus of the soil
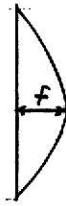
D = pile diameter

μ_s = Poisson's ratio of soil

p = pile perimeter

ξ = 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 $I_{ws} = 2 + 0.35\sqrt{\dfrac{L}{D}}$

g. **Pile Driving Formulas and Pile Load Tests**

As already indicated, dynamic formulas or pile load

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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

$$Q_u = \frac{EW_R h}{S + C} \cdot \frac{W_R + n^2 W_p}{W_R + W_p} \qquad (11.34)$$

where

E = hammer efficiency (given below)

Hammer type	E
Single-double acting hammers	0.7-0.85
Diesel hammers	0.8-0.9
Drop hammers	0.7-0.9

W_R = weight of the ram (specified value)

h = height of fall of the ram

s = penetration of pile per hammer below (usually based on the average obtained from the last few driving blows).

c = 0.254 cm or 0.1 in dependent on units of s and h

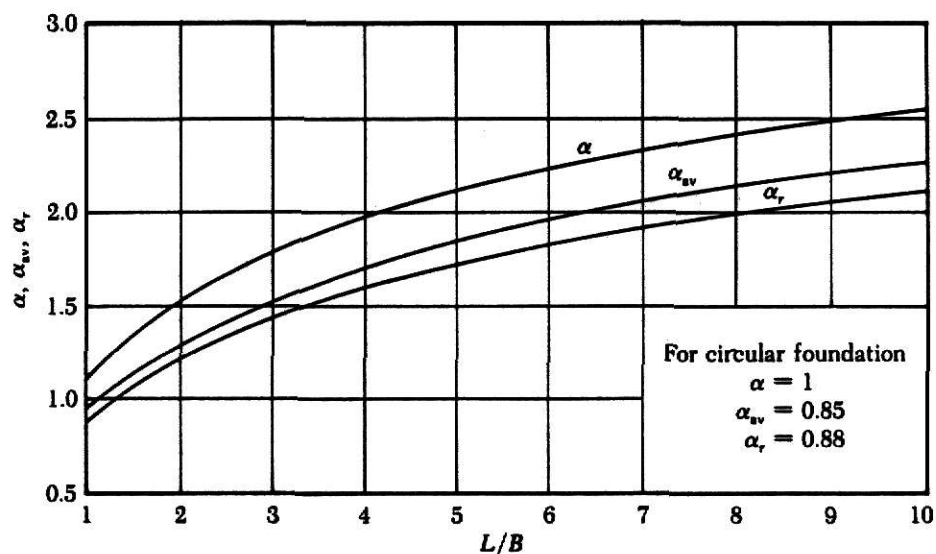n = coefficient of restitution between ram and pile cap

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Figure 11.13.   Values of a, a$_{av}$, and a$_r$.

Pile material	n
Cast iron hammer and concrete pile (without cap)	0.4-0.5
Wood cushion on steel piles	0.3-0.4
Wooden piles	0.25-0.3

Wp = 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.
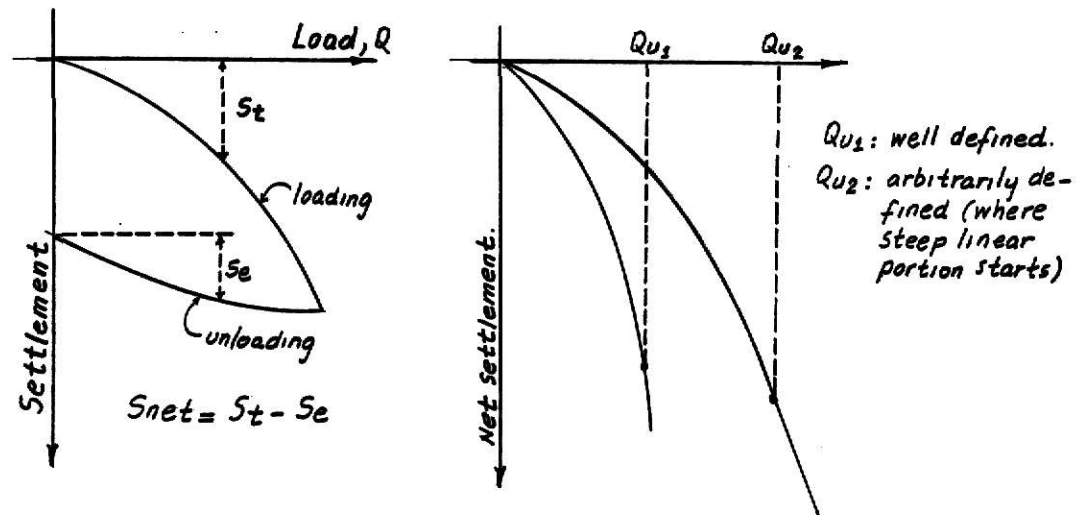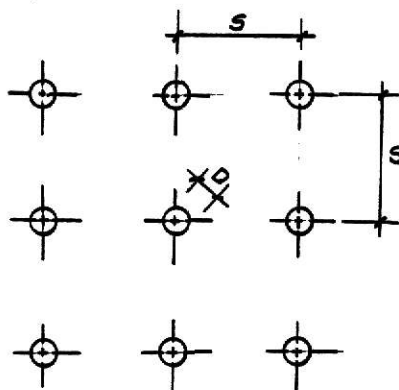

Figure 11.14. Pile-load Test Curve.

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

$C_u = 95.8$ K_n/m^2

$m = 3 \quad n=3$

1. Piles working individually

$$\sum Q_u = mm \left(9\, c_u\, A_p + \sum \alpha\, p\, c_u\, \Delta L\right) \qquad (11.30)$$

$\alpha = 0.53$ from Fig. 11.7 for $c_u = 9.58 \dfrac{K_n}{m^2}$

$$A_p = \frac{\pi (0.406)^2}{4} = 0.1295\ m^2$$

$$p = \pi(0.406) = 1.2755\ m$$

$\Delta L = 18.5\ m$ (the stratum is homogeneous)

$$\sum Q_u = 3 \times 3[(9 \times 95.8 \times 0.1295)+(0.53 \times 1.2755 \times 95.8 \times 18.5)]$$

$$\sum Q_u = 11787.81\ KN$$

2. Piles working as a group

$$Q_{g(u)} = L_g\, B_g\, c_u\, N_c^* + \sum 2(L_g + B_g)\, c_u\, L \qquad (11.31)$$

$$L_g = (m-1)s + D = (3-1)0.850 + 0.406 = 2.106\ m$$

$$B_g = (n-1)s + D = (3-1)0.850 + 0.406 = 2.106\ m$$

$$\frac{L_g}{B_g} = 1 \qquad \frac{L}{B_g} = \frac{18.5\ m}{2.106\ m} = 8.78 \qquad N_c^* = 9$$

$$Q_{g(u)} = (2.106 \times 2.10c \times 95.8 \times 9) + 2(2.106 + 2.106)\,95.8 \times 18.5$$

$$Q_{g(u)} = 18753.92\ KN$$

3. $Q_{g(u)} > \sum Q_u$ use $\sum Q_u = 11787.81$

$$Q_a = \frac{11787.81}{3} = \underline{3929.27\ KN}$$

If the group of piles is embedded in sand, what is the efficiency of the group?

$$n = \frac{2(m + n - 2)s + 4D}{P\, m\, n} \qquad \text{Eq. 11.29}$$

$$n = \frac{2(3 + 3 - 2)0.850 + 4(0.406)}{1.2755(3)(3)} = 0.74$$

then the efficiency is 74%.

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CLASS OUTLINE

Text - Braja M. Das -- Principles of Foundation Engineering,
Brooks/Cole Engineering Division, 1984

Week	Topics	Chapter	Article
1	Introduction	1	1.1
	Soil Mechanics Review	1	1.4, 1.8, 1.14, 1.15, 1.17, 1.18, 1.19, 1.20, 1.21, 1.22
2	Subsurface Exploration	2	2.1, 2.2, 2.3, 2.4, 2.9, 2.12
3	Bearing Capacity of Shallow Foundations	3	3.1, 3.2, 3.3, 3.4, 3.5
4	Bearing Capacity of Shallow Foundations	3	3.6, 3.7, 3.8
5	Settlement of Shallow Foundations	3	3.9, 3.10, 3.20, 3.21
6	Factor to be Considered in Foundation Design	Notes given by instructor	
	Spread Footing Design	Appendix A	A.1, A.2, A.3, A.4, A.5, A.6, A.7
7	Special Footings	4	4.1, 4.2, 4.3, 4.4, 4.6
8	Lateral Earth Pressure	5	5.1, 5.2, 5.3, 5.4
9	Lateral Earth Pressure	5	5.5, 5.6, 5.7, 5.9, 5.10
10	Retaining Walls	5	5.12, 5.13, 5.14, 5.15

11	Retaining Walls	5 and work-out examples	5.16, 5.17, 5.18
12	Pile Foundations	8	8.1, 8.2, 8.3, 8.4, 8.6, 8.8
13	Pile Foundations	8	8.9, 8.12, 8.14, 8.15, 8.16, 8.17
14	Review (examples of the topics studied)		

NOTES ON FOUNDATION ENGINEERING

by

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AN ABSTRACT OF A MASTER'S REPORT

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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.