GEOTECHNICAL ENGINEERING OVERVIEW

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PDH Now, LLC
857 East Park Avenue
Tallahassee, FL 32301

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1 SOIL MECHANICS REVIEW

This chapter presents a brief review of the basic geotechnical properties of soils needed for foundation analysis and design. Topics such as grain-size distribution, consistency and plasticity of soils, soil permeability, effective stresses, consolidation, and shear strength will be briefly discussed.

1.1 Natural Soil Deposits

In most geotechnical engineering evaluations and analyses, the engineer assumes the soils to be homogeneous and isotropic, which is just an idealization. Such idealization requires some knowledge of the geological process by which the soil deposit at the site was formed. Most soils are formed by the weathering of various rocks. Weathering may be mechanical, in which, rocks are broken into smaller pieces that maintain the same chemical composition of the main rock or chemical, in which, the rock may change to something entirely different.

Soil produced by weathering may stay in its original place (residual soils) or may be transported to other places (transported soils). Some examples of transported soils are gravity transported deposits, lacustrine (lake) deposits, Alluvial or fluvial, deposited by running water, glacial, deposited by glaciers, and Aeolian, deposited by wind.

1.2 Size Limits of Soils

Figure 1.1 provides size limits of the main soil types usually encountered by the geotechnical engineer such as boulders, cobbles, gravel, sand, silt, and clay using four internationally recognized standards. The Unified Soil Classification System (USCS) is the most widely used for foundation analysis and design purposes in the states. The particle size, 0.075 mm in the ASTM, USCS, and AASHTO (0.06 mm in BS) is of particular importantance, since it separates the granular/cohesionless soils from the cohesive soils.

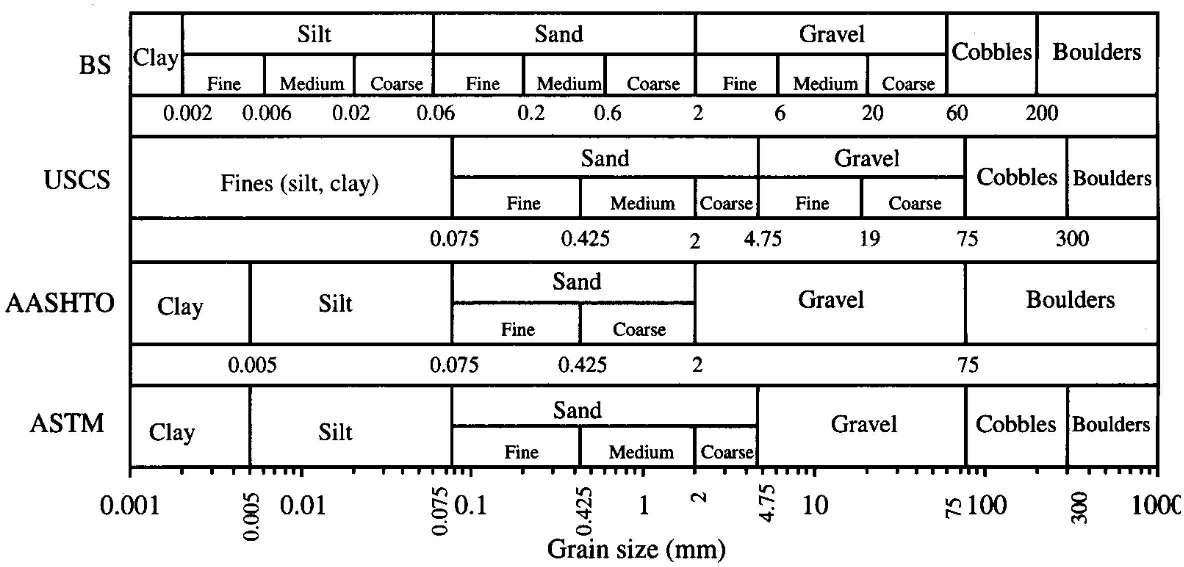


Figure 1.1 Classification of particle size in the BS 1377, USCS, AASHTO, and ASTM engineering soil classification systems.

1.3 Basic Definitions (Weight-Volume Relationships)

The soil matrix consists of solids and voids. Voids may be filled with either water or air. Solids, water, and air are known as the three-phase system. The soil is fully saturated if all voids are filled with water and is dry if all voids are filled with air. Figure 1.2 shows an engineering representation of the soil's three-phase system.

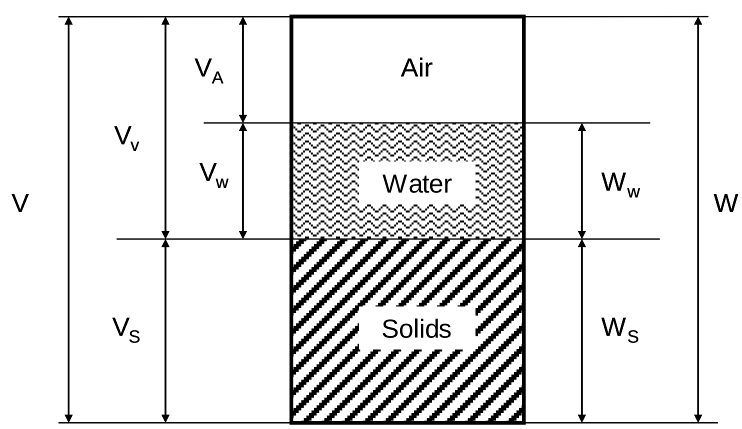


Figure 1.2 Phase diagram

The following summarizes the basic definitions and relationships:

Void Ratio (e): $e = \frac{V_v}{V_S}$

Porosity (n): $n = \frac{V_v}{V}$

Degree of Saturation (S): $S = \frac{V_w}{V_v}$

Moisture Content (w): $w = \frac{W_w}{W_s}$

Unit Weight (γ): $\gamma = \frac{W}{V}$ & $\gamma_{dry} = \frac{W_{dry}}{V}$ & $\gamma_s = \frac{W_s}{V_s}$

Specific Gravity (G_s): $G_s = \frac{\gamma_s}{\gamma_w} = \frac{W_s}{V_s \gamma_w}$.

Table 1.1 provides typical values for the specific gravity of most common soil types.

Table 1.1 Typical specific gravity of some soils

Type of Soil	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

1.4 Basic Relations

Some of the presented definitions may be measured in the laboratory such as moisture content and unit weight. Others are very hard to measure experimentally such as void ratio. In order to estimate some of the basic properties using those measured in the laboratory, the following relations may be used:

$$n = \frac{e}{1+e}$$

$$w = \frac{S\,e}{G_s}$$

$$\gamma = \frac{\gamma_w G_s (1+w)}{1+e}$$

$$\gamma_{sat} = \frac{\gamma_w (G_s + e)}{1+e}$$

$$\gamma_{dry} = \frac{\gamma_w G_s}{1+e}$$

$$\gamma_{dry} = \frac{\gamma}{1+w}$$

$$\gamma_{sub} = \gamma_{sat} - \gamma_w$$

where: γ_{sub} or $\gamma^{/}$ is the submerged unit weight of soil, γ_{dry} is he dry unit weight of soil, and γ_{sat} is the saturated unit weight of soil. γ_w is the water unit weight = 62.4 pcf = 9.81 kN/m^3.

1.5 Relative Density

Relative density is an important measure of granular soil compactness in the field. It is calculated using either one of the following relations:

$$D_r(\%) = \frac{e_{max} - e_f}{e_{max} - e_{min}} \times 100$$

$$D_r(\%) = \left(\frac{\gamma_d - \gamma_{d(min)}}{\gamma_{d(max)} - \gamma_{d(min)}}\right)\left(\frac{\gamma_{d(max)}}{\gamma_d}\right) \times 100$$

Table 1.2 may be used to classify granular soils according to their in-field relative density:

Table 1.2 Denseness of granular soils

Relative density, D_r(%)	Description
0–20	Very loose
20–40	Loose
40–60	Medium
60–80	Dense
80–100	Very dense

1.6 Grain Size Distribution

The grain-size distribution of granular soils is generally explored using Sieve Analysis utilizing a stack of progressively finer sieves and measuring the amount of dry soil retained on each sieve.

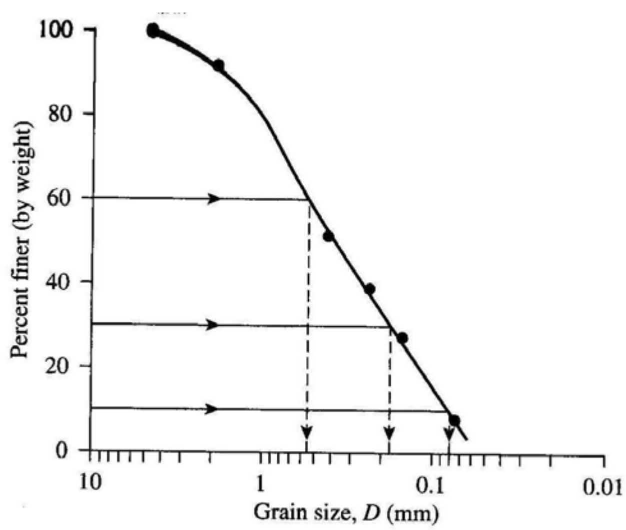
The percent passing (finer) is plotted against the corresponding sieve opening (equivalent soil diameter) to provide the grain size distribution curve shown on Figure 1.3.

The shape of the curve, hence, the grain-size distribution of the soil may be evaluated using the Uniformity Coefficient (C_u) and the Coefficient of Gradation (C_c):

$$C_u = \frac{D_{60}}{D_{10}}$$

$$C_c = \frac{D_{30}^2}{(D_{60})(D_{10})}$$

For well-graded sand, $C_u > 6$ and $1 < C_c < 3$. For well-graded gravel, $C_u > 4$ and $1 < C_c < 3$.



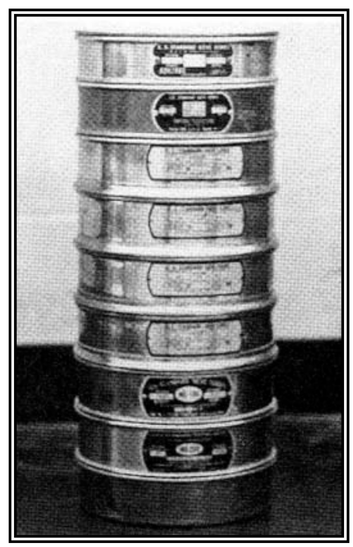


Figure 1.3 A typical grain-size distribution and a typical stack of sieves

The grain-size distribution of cohesive soils (silts and clays) is generally studied using the principle of sedimentation of soil particles in water utilizing a specially manufactures hydrometer that is capable of measuring the amount of soils still in suspension at any given time.

The largest soil particles still in suspension at time (t) can be expressed using Stokes' law as follows:

$$D = \sqrt{\frac{18\eta}{(G_s - 1)\gamma_w}}\sqrt{\frac{L}{t}}$$

Figure 1.4 shows examples of typical curves for well-graded and poorly-graded (uniform) soils.

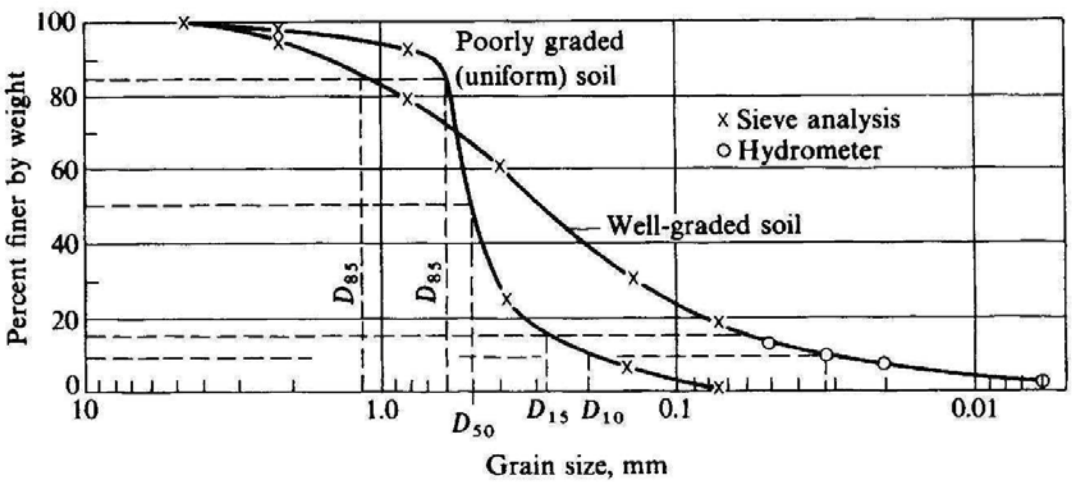


Figure 1.4 Typical grain-size distribution curves

1.7 Consistency of cohesive Soils (Atterberg Limits)

Moisture content has a great effect on the strength and compressibility characteristics of clayey soils. Adding an extensive amount of water to a clayey soil may turn it into semi liquid state loosing most of the shearing resistance. Atterberg defined the Shrinkage Limit (SL), Plastic Limit (PL), and Liquid Limit (LL), which are specific limits that define transition stages from solid to semi-solid, semi-solid to plastic, and plastic to liquid, respectively. The liquid limit may be determined using Casagrande apparatus (Figure 1.5). The liquid limit is the moisture content at which the groove made with the standard tool closes for 1 inch after 25 blows. Figure 1.5 also shows the glass board used to estimate the plastic limit. The plastic limit is the moisture content at which the soil threads starts to crumble at about 1/8 inch diameter.

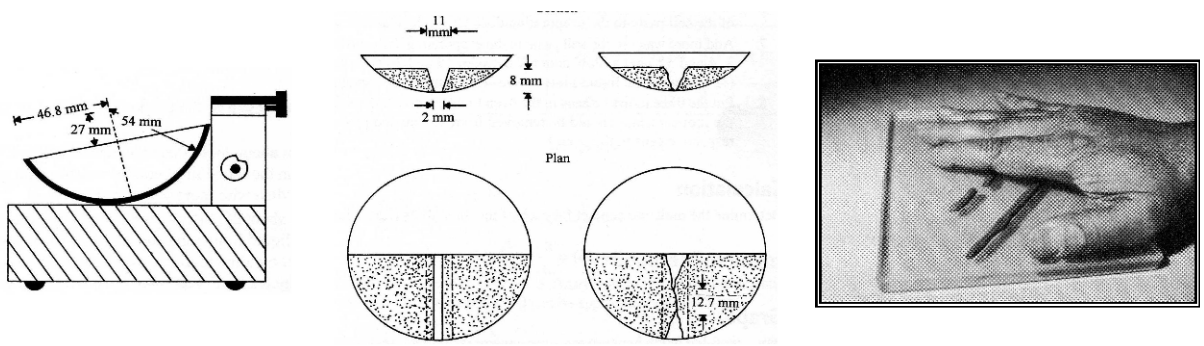


Figure 1.5 Laboratory estimation of liquid and plastic limits

The Plasticity Index, which is an indicator of clay content, is defined as follows:

$$PI = LL - PL$$

The plasticity index is used along with the liquid limit to classify cohesive soils using the plasticity chart shown on Figure 1.6.

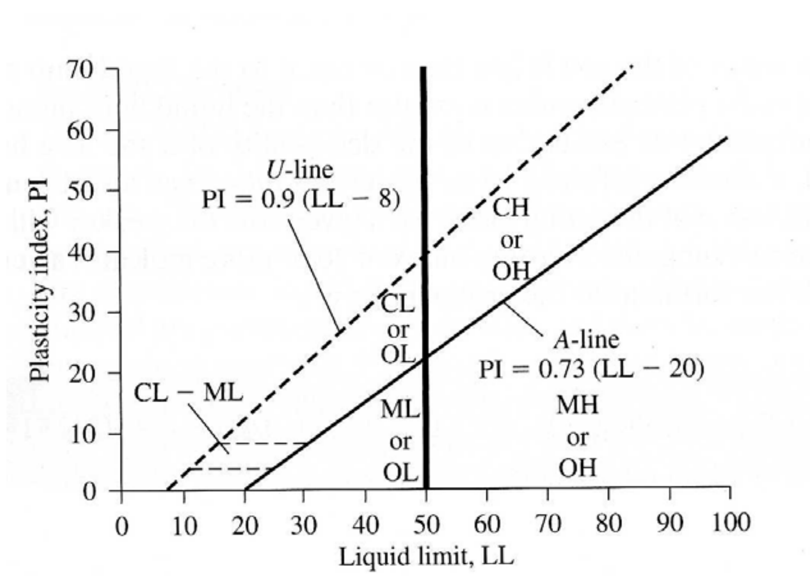


Figure 1.6 The plasticity chart

1.8 Hydraulic Conductivity of Soils

Water flow through the voids of the soil matrix may be described using Darcy's law. The study of water flow through soils is essential in designing earth dams, estimating seepage losses below concrete dams or sheet piles, and foundation dewatering. Darcy's law, which is usually used to describe flow of water through soils, may be expressed as follows:

$$v = k\, i$$

$$i = \frac{\Delta h}{L}$$

where; v is the superficial velocity, k is the hydraulic conductivity, and i is the hydraulic gradient causing the water flow, which can be defined as the head difference divided by the length of water flow through the soil. Figure 1.7 summarizes these definitions.

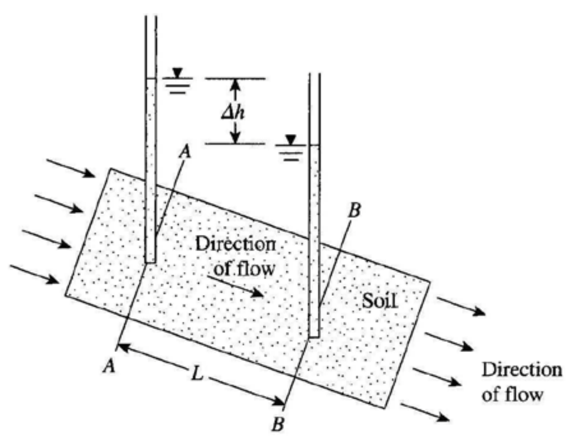


Figure 1.7 Basic definition of Darcy's law

The hydraulic conductivity may be determined in the laboratory using either the constant head permeability test (for granular soils) or falling head permeability test (for cohesive soils). In-field, either borehole tests or pumping out tests are used to estimate the in-field bulk hydraulic conductivity. Table 1.3 provides typical values of the hydraulic conductivities of various soils.

Table 1.3 Typical values of hydraulic conductivity

Type of soil	**Hydraulic conductivity, *k* (cm/sec)**
Medium to coarse gravel	Greater than 10^{-1}
Coarse to fine sand	10^{-1} to 10^{-3}
Fine sand, silty sand	10^{-3} to 10^{-5}
Silt, clayey silt, silty clay	10^{-4} to 10^{-6}
Clays	10^{-7} or less

Laplace's theory of continuity governs the steady state seepage. Flow net construction, which involves flow lines and equipotentional lines, is a graphical solution of the Laplace equation that may be used to estimate the amount of seepage through an earth dam or below a concrete dam or along a sheet pile wall. The flow net solution enables the engineer to estimate the amount of seepage through or below dams, estimate the uplift pressure on partially submerged structures, and check the stability of the downstream (exit face) against piping and heave.

1.9 Effective Stress Concept

The total stress (σ) at any point in a soil mass is the summation of the effective stress ($\sigma^{/}$) and the pore water pressure (u). The effective stress is the vertical component of forces at particle –to-particle contact points over a unit cross-sectional area. All the strength and compressibility problems are usually solved using the effective stresses rather than total stresses.

In order to calculate the effective stress at any depth in a soil mass, the total stress should be estimated using the bulk/saturated unit weight then the pore water pressure should be subtracted. However, the effective stress may be directly calculated using the bulk unit weight above the water table and the submerged unit weight below the water table. Figure 1.8 provides an example of calculating effective stresses in case there is no seepage.

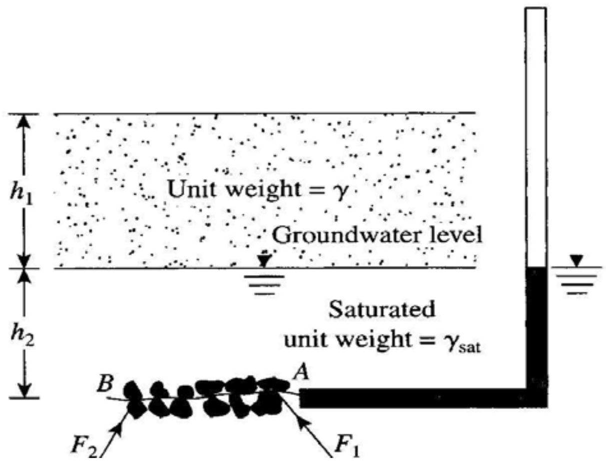


Figure 1.8 Effective stress calculation

$$\sigma = \sigma' + u = \gamma h_1 + \gamma_{sat} h_2$$
$$u = h_2 \gamma_w$$
$$\sigma' = (\gamma h_1 + \gamma_{sat} h_2) - (h_2 \gamma_w) = \gamma h_1 + h_2(\gamma_{sat} - \gamma_w) = \gamma h_1 + \gamma' h_2$$

1.10 Consolidation

Consolidation is the gradual decrease in volume of a saturated clay layer subjected to constant stress increase. The decrease in volume is very slow since the clay layer has very low hydraulic conductivity (K). The stress increase is initially transferred totally to the pore water then the water starts to gradually squeeze out of the voids transferring the excess stresses to the soil skeleton causing time dependent settlement “consolidation settlement”.

The consolidation properties of a clay layer may be estimated in the laboratory using the consolidation (oedometer) test. Consolidation test (ASTM D-2435) has to be performed on undisturbed clay samples. The test specimens are usually 2.5 inches in diameter and 1 inch in height. Specimens are placed inside a ring with top and bottom porous stones to facilitate water flow in or out of the sample. The sample is loaded vertically and settlement readings are taken for a period of 24 hours after which the load is doubled. The procedure is continued until the desired stress level is achieved. The sample may be unloaded to study the swelling behavior of the clay. Figure 1.9 shows a sketch for the Oedometer along with the resulting graph, which shows the variation of the voids ratio at the end of consolidation against the corresponding vertical effective stress on a log scale.

Three parameters may be defined using the e-log$\sigma^{/}$ graph:

The Preconsolidation Pressure ($\sigma^{/}_{c}$):

It is the maximum past effective overburden pressure that the sample has been subjected to. It may be estimated using Terzaghi's graphical procedure shown on Figure 1.9. The sample is pre-consolidated if the preconsolidation pressure ($\sigma^{/}_{c}$) is more than the current overburden pressure ($\sigma^{/}_{0}$), however, the sample is normally consolidated if $\sigma^{/}_{c} = \sigma^{/}_{0}$.

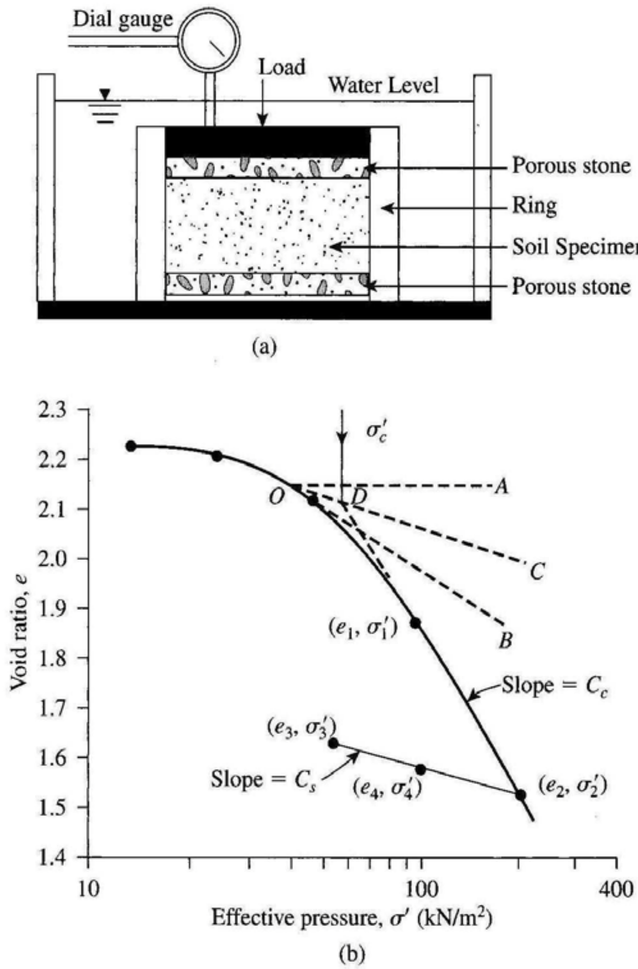


Figure 1.9 a) Schematic diagram of the Oedometer; b) typical e-logσ' curve

<u>The Compression Index (C_c):</u>

It is the slope of the virgin compression portion of the e-logσ' curve as follows:

$$C_c = \frac{\Delta e}{\log\left(\frac{\sigma'_2}{\sigma'_1}\right)}$$

An approximate value of the compression index may also be estimated using the Liquid Limit as follows:

$$C_c = 0.009(LL - 10)$$

<u>The Swelling Index (C_s):</u>

It is the slope of the unloading portion of the e-log$\sigma^{/}$ curve as follows:

$$C_s = \frac{\Delta e_{unloading}}{\log\left(\frac{\sigma'_2}{\sigma'_1}\right)_{unloading}}$$

In most cases, the swelling index is about 0.2 to 0.25 of the compression index.

1.10.1 End of Primary Consolidation Settlement

Figure 1.10 shows a clay layer of thickness (H), initial voids ratio (e_0), and compression index (C_c). The average overburden pressure is $\sigma^{/}_0$ and the average stress increase within the clay layer is $\Delta\sigma'$. Three cases may be encountered when estimating the end of primary consolidation settlement depending on the pressure history as follows:

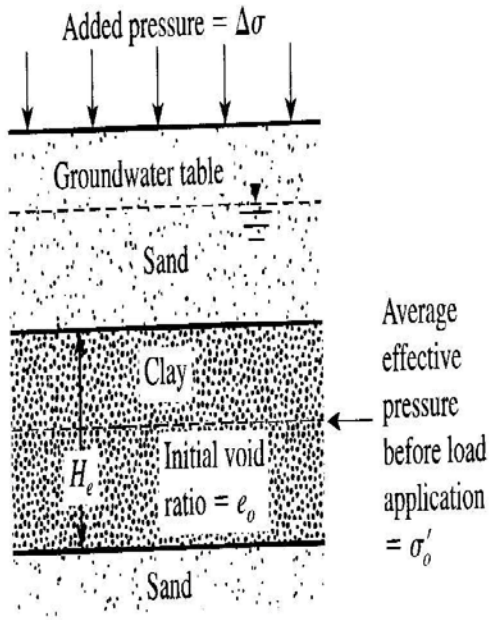


Figure 1.10 Calculation of end of primary consolidation settlement

For normally consolidated clay :

$$S_c = \frac{C_c H_c}{1+e_o}\log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_o}\right)$$

For pre - consolidated clay with $(\sigma'_o + \Delta\sigma') < \sigma'_c$:

$$S_c = \frac{C_s H_c}{1+e_o}\log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_o}\right)$$

For pre - consolidated clay with $(\sigma'_o + \Delta\sigma') > \sigma'_c$:

$$S_c = \frac{C_s H_c}{1+e_o}\log\left(\frac{\sigma'_c}{\sigma'_o}\right) + \frac{C_c H_c}{1+e_o}\log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_c}\right)$$

1.10.2 Time Rate of Consolidation

How long it will take for the end of primary settlement to occur? In order to answer this question, the engineer must study the consolidation with time. The consolidation with time may be studied utilizing the settlement-time data obtained from the consolidation test for the load increment that represents the actual loading conditions in field. The following definitions are essential in this type of study:

The Degree of Consolidation:

It is the ratio of the settlement at the desired time to the end of primary consolidation settlement.

$$U(\%) = \frac{S_{\text{at time t}}}{S_{\text{end of primary (EOP)}}} \times 100$$

The Time Factor:

This is a part of the constant of the solution of the basic differential equation of Terzaghi's consolidation theory. It is solely correlated to the degree of consolidation as follows:

$$T_v = \frac{\pi}{4}\left(\frac{U}{100}\right)^2 \quad ......U < 60\%$$

$$T_v = 1.781 - 0.933\log(100 - U) \quad .....U > 60\%$$

The Coefficient of Consolidation:

This is the main soil property that governs consolidation with time as described in Terzaghi's theory:

$$\frac{\partial(\Delta u)}{\partial t} = C_v \frac{\partial^2(\Delta u)}{\partial z^2}$$

The coefficient of consolidation is correlated to the hydraulic conductivity (k), the coefficient of volume decrease (m_v), and the unit weight of water as follows:

$$C_v = \frac{k}{m_v \gamma_w}$$

The coefficient of consolidation may be estimated from the consolidation test using either the root-time method (Taylor Method) or the log-time method (Casagrande Method). The following equation may be used after determining the time corresponding to a certain degree of consolidation (50% in Taylor Method and 90% in Casagrande Method) to estimate the coefficient of consolidation:

$$T_v = \frac{C_v t}{H_{dr}^2}$$

1.11 Shear Strength

Soils fail in shear rather than in compression, tension, or bending. Soil is a semi-infinite medium, hence, failure will occur by separating a part of the semi-infinite medium by shear. Soils resist shear stresses by friction (granular soils) and/or cohesion (fine grained soils).

1.11.1 Mohr-Coulomb Failure Criterion

This is a widely used failure criterion for soils. It has been utilized in most of the Finite Element Analysis packages for geotechnical applications. The shear stress at failure may be describes as follows:

$$\tau' = c' + \sigma' \tan\phi'$$

where, τ' is the shear stress at failure, c' is the cohesion, σ' is the effective normal stress at failure, and φ' is the effective angle of shearing resistance. Mohr-Coulomb criterion can be graphically represented as shown on Figure 1.11.

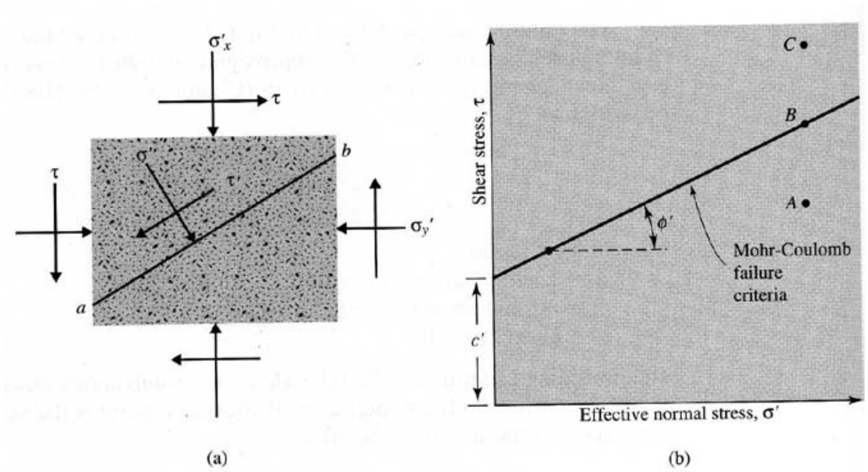


Figure 1.11 Graphical representation of Mohr-Coulomb failure criterion

The shear strength parameters, c' and φ' can be estimated in the laboratory using the direct shear test and/or the triaxial test.

1.11.2 Direct Shear Test

Soils, especially sandy soils, can be conventionally tested using direct shear test. The sand is placed in a shear box that is laterally split in two halves. The vertical load (N) is then applied to provide the desired normal stress. The top half is then moved at a constant rate to shear the sample. The lateral load is measured (R) using a load cell or a proving ring. Both the shear and volumetric displacements are measured during the test. As shown on Figure 1.12, the tests continues to failure, which may be characterized by reaching a peak and then excessive shear displacement (dense sands or stiff clays) or by reaching the ultimate shear stress that is associated with excessive displacement without increase in loads (loose sands and soft clays).

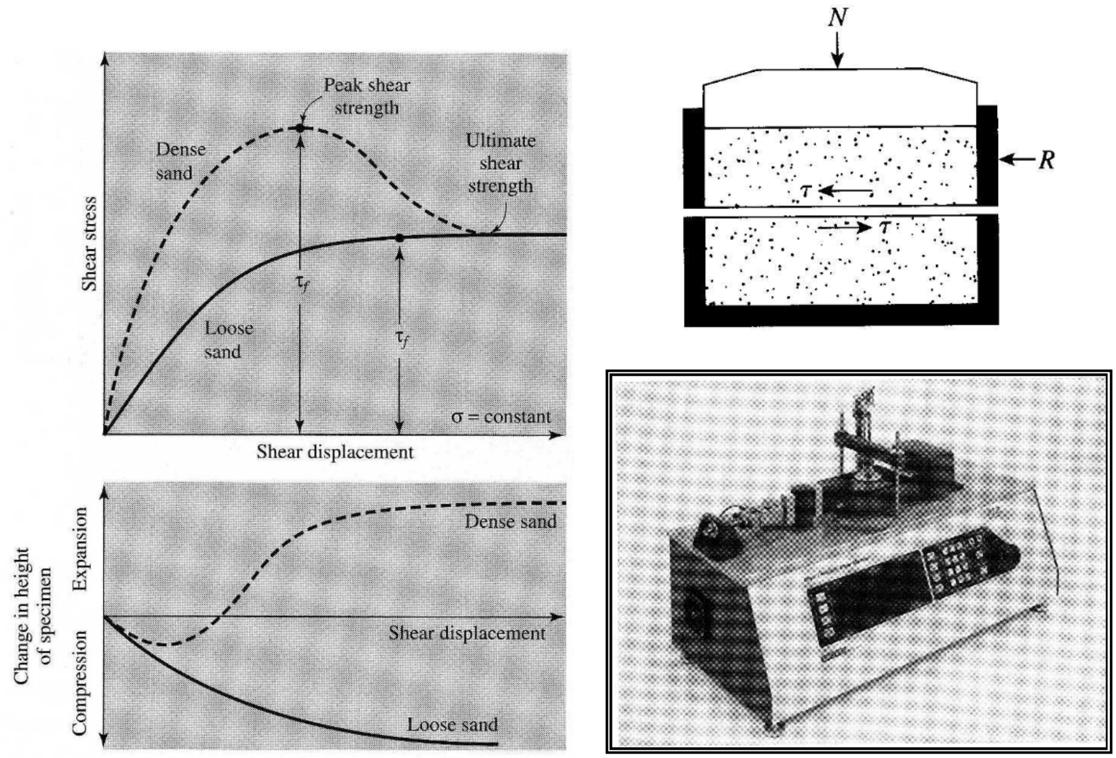


Figure 1.12 Direct shear test

The shear stress at failure is then plotted against the corresponding normal stress as shown on Figure 1.13, which will result in a single point in the τ'-σ' space. The test may be repeated as needed to better define the Mohr-Coulomb Criterion for the soil.

Figure 1.13 shows Mohr-Coulomb Criterion envelopes associated with sandy, clayey, and mixed soils. The direct shear test is popular, quick, and in-expensive, however, the main dispute about its results is that the plane of failure is pre-determined, which is basically between the two halves of the box.

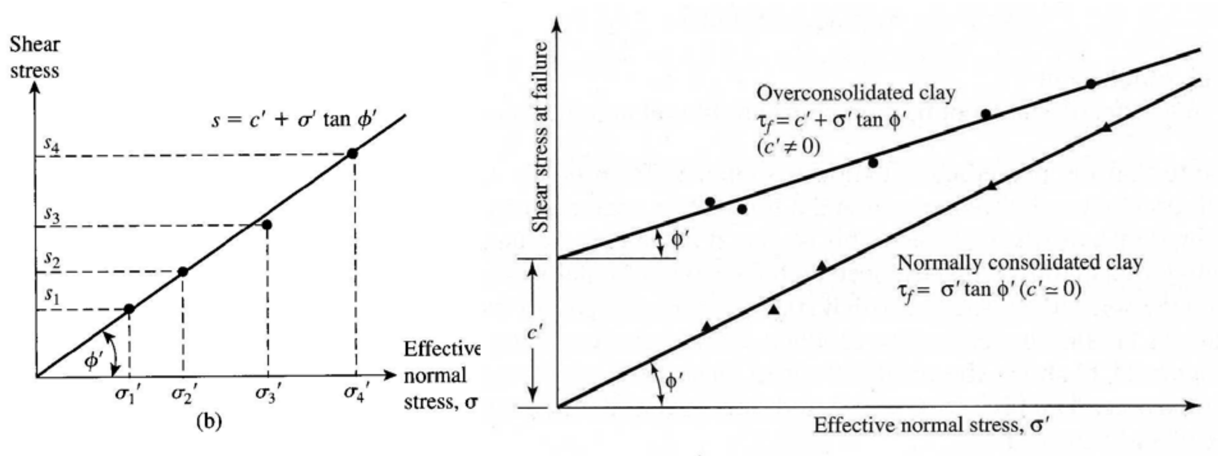


Figure 1.13 Mohr-Coulomb Criterion envelopes for different soil types.

1.11.3 Triaxial Compression Test

Triaxial compression tests may be conducted on all types of soils. The sample is subjected to three major compression stresses, two are in plane (σ_2 and σ_3), which are equal and the third is perpendicular to their plane (σ_1). The sample is confined by a rubber membrane and put in a cell that is usually filled with water under pressure to apply the all around pressure/cell pressure (σ_2 and σ_3). The sample is then compressed vertically by the deviatoric stress (σ_1 - σ_3) to failure. The sample will fail in the preferred plane of failure not at a pre-determined plane as in the direct shear test. Figure 1.14 shows a schematic diagram for the triaxial compression test apparatus.

The triaxial test consists of two stages; the consolidation stage and the compression/shear stage. Drainage could be either allowed or prevented during any of the two stages. If drainage is allowed during the consolidation stage, the test is called “Consolidated (C)” otherwise it is called “Unconsolidated (U)”. Also if drainage is allowed during compression, the test is called “Drained (D)”; otherwise it is called “Undrained (U)”. Three types of triaxial tests are usually performed in the laboratory; Consolidated Drained (CD) test also known as slow test, Consolidated Undrained (CU) test, and Unconsolidated Undrained (UU) test also know as the quick test.

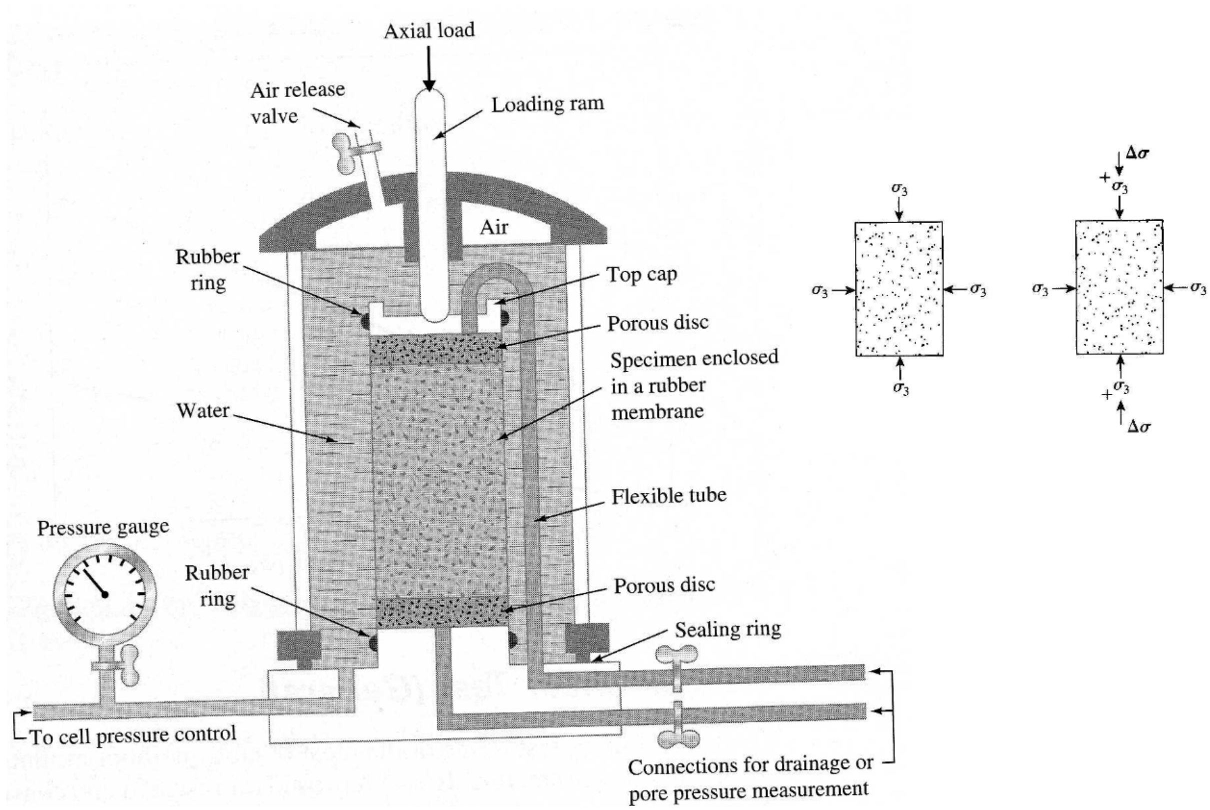


Figure 1.14 Schematic diagram for the triaxial compression test

Performing a triaxial test under a specific cell pressure, also known as minor principal stress, will result in a corresponding failure stress, also known as major principal stress, which in turn can be represented graphically by a Mohr circle. Repeating the test using a different cell pressure will result in another Mohr circle.

The common tangent to Mohr circles defines the shear strength parameters (total or effective) of soils. Figures 1.15 shows typical results of CD, CU, and UU triaxial compressions tests.

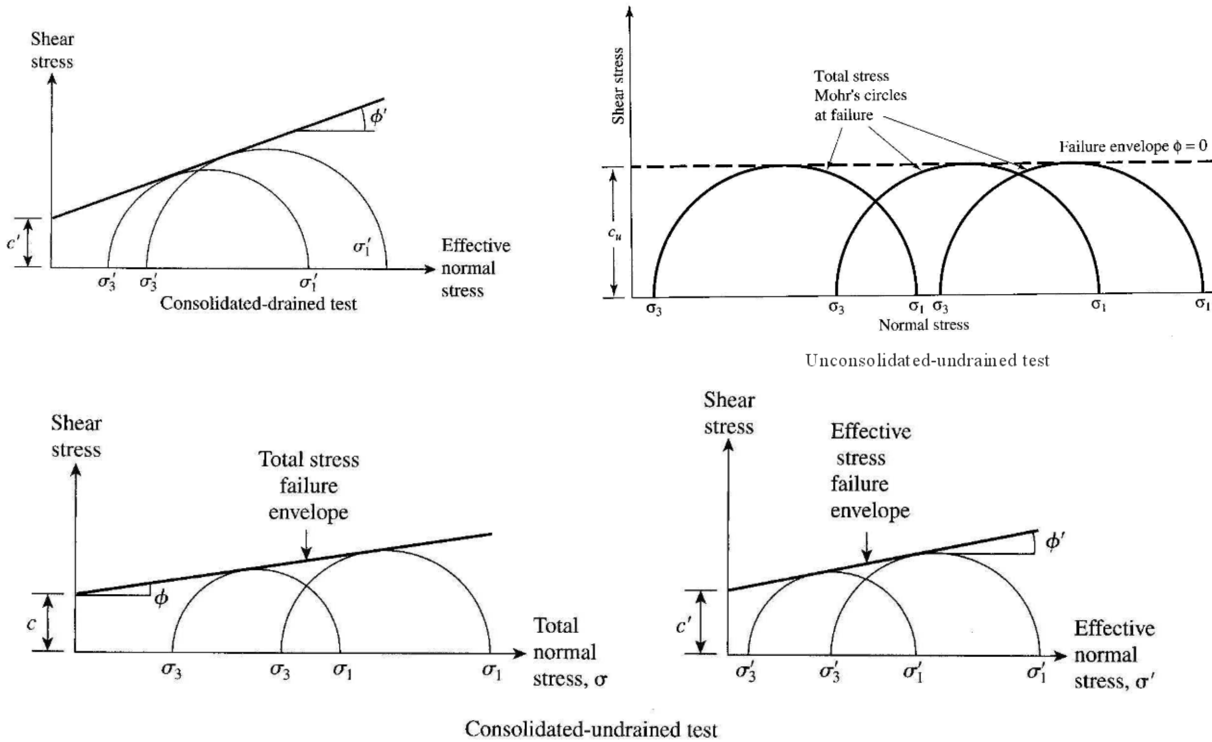


Figure 1.15 Typical results of CD, CU, and UU triaxial compression tests.

1.11.4 Unconfined Compression Test

The unconfined compression test is a special case of the UU test with a zero cell pressure. The sample should be able to support itself without the need for cell pressure. The test is usually performed on clayey soils. The stress at failure is called “The Unconfined Compression Strength (q_u)”. Figure 1.16 shows the basic setup as well as typical results of an unconfined compression test. The undrained shear strength C_u is defined as:

$$C_u = \frac{q_u}{2}$$

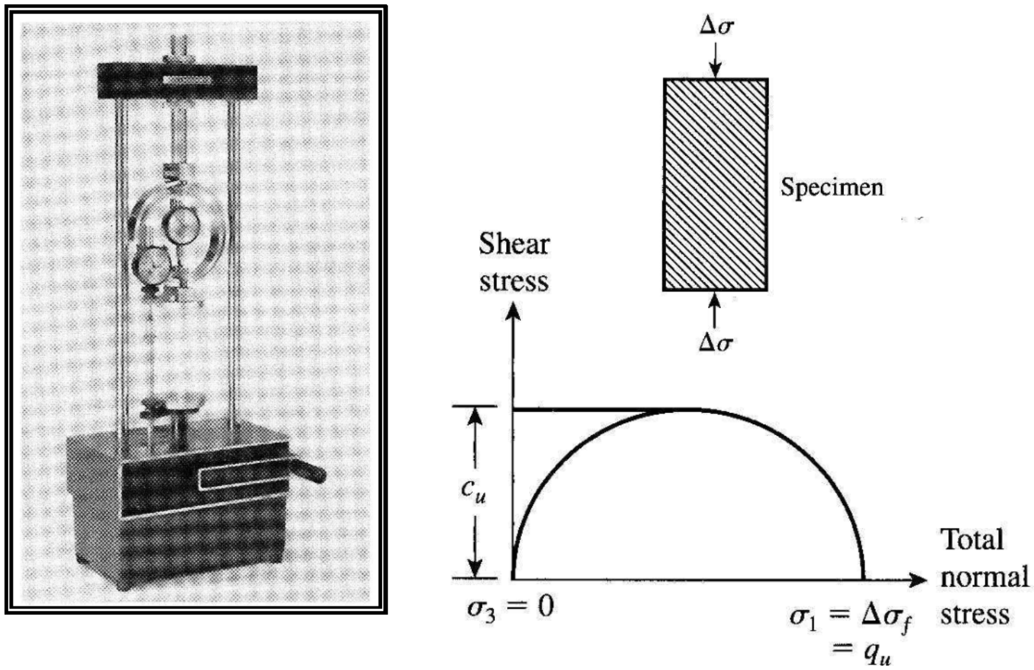


Figure 1.16 The unconfined compression test

2 SUBSURFACE EXPLORATIONS

2.1 Purpose

Subsurface explorations should provide:

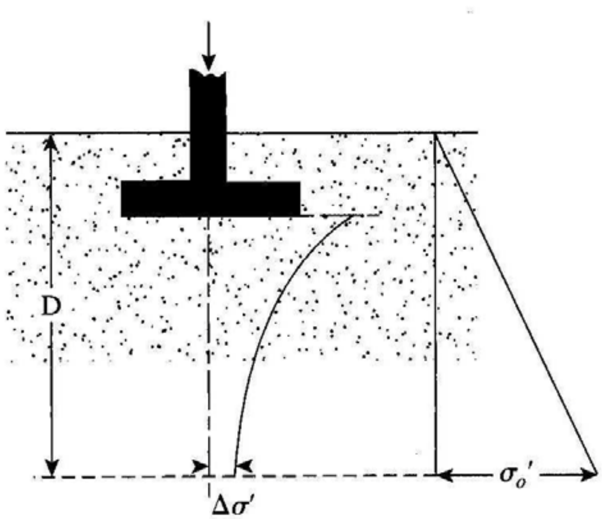
- Information to determine the type of foundation required (shallow or deep).
- Information to evaluate the allowable bearing capacity.
- Sufficient data and samples for laboratory tests to estimate initial and time dependent settlements.
- Depth to the groundwater table (GWT) and Seasonal High Water Level (SHWL).
- Information of any construction problems that may exist.
- Identification of potential problems concerning adjacent buildings.

2.2 Subsurface Explorations Program

The first step in performing a good subsoil exploration program is to review published data such as Geology publications for the county, state, etc., the USGS quadrangle map for the general site area, the USDA SCS Soil Survey maps for the specific site, and the hydrological data published by the water management districts. Those published data will provide the engineer the history, the overall picture, and information about surface soils, which is turn, guide the engineer during designing the specific subsurface exploration program.

The next step is to visit the site and perform "site reconnaissance" in order to identify site accessibility, general topo, type of vegetations, groundwater marks, etc.

The following step is to perform borings at specific locations to explore the subsoil and groundwater conditions at the site. Type of borings, number of borings, and depth of explorations should be selected depending on many factors such as the type of development, column loads, previous experience on site, expected foundation types, etc. Experience and engineering judgment play an important rule in designing a good exploration program. Figure 2.1 provides some recommendations for spacing and depth of borings (Bowles 1996). The figure also shows the “10% rule”, which basically states that the boring should extend to a minimum depth (D) at which the stress increase resulted from the foundation is equal or less than 10% of the overburden pressure.



No. of stories	Boring depth	
1	3.5 m	(11 ft)
2	6 m	(20 ft)
3	10 m	(33 ft)
4	16 m	(53 ft)
5	24 m	(79 ft)

	Spacing	
Type of project	(m)	(ft)
Multistory building	10–30	30–100
One-story industrial plants	20–60	60–200
Highways	250–500	800–1600
Residential subdivision	250–500	800–1600
Dams and dikes	40–80	130–260

Figure 2.1 Spacing and depth of borings

2.3 Exploration Methods and Soil Sampling

Table 2.1 summarizes different exploration methods and sample recovery methods (Bowles 1996). Soil samples may be disturbed or undisturbed. Disturbed samples are used for visual and manual soil classification, index property tests such as moisture content, specific gravity, sieve analysis, organic content, and Atterberg limits. Undisturbed samples are used for strength and compressibility tests such as consolidation, direct shear, triaxial compression, unconfined compression test, and permeability tests.

Table 2.1 Soil exploration and sample recovery methods

Method	Depths	Applicability
Disturbed samples taken		
Auger boring†	Depends on equipment and time available, practical depths being up to about 35 m	All soils. Some difficulty may be encountered in gravelly soils. Rock requires special bits, and wash boring is not applicable. *Penetration testing* is used in conjunction with these methods, and disturbed samples are recovered in the split spoon. Penetration counts are usually taken at 1- to 1.5 m increments of depth
Rotary drilling Wash boring Percussion drilling	Depends on equipment, most equipment can drill to depths of 70 m or more	
Test pits and open cuts	As required, usually less than 6 m; use power equipment	All soils
Undisturbed samples taken		
Auger drilling, rotary drilling, percussion drilling, wash boring	Depends on equipment, as for disturbed sample recovery	Thin-walled tube samplers and various piston samplers are used to recover samples from holes advanced by these methods. Commonly, samples of 50- to 100-mm diameter can be recovered
Test pits	Same as for disturbed samples	Hand-trimmed samples. Careful trimming of sample should yield the least sample disturbance of any method

* Marine sampling methods not shown.
† Most common method currently used.

2.4 Split Spoon Sampler (Standard Penetration Test SPT)

The standard penetration test (SPT) borings is the most popular sampling and in-situ penetration resistance testing method. The number of blows required to drive the split spoon sampler for 12 inches is recorded and called “The SPT N-value”. The N-value is correlated to most of the soil strength and compressibility characteristics. The hummer is 140 lb and drops from a distance of

30 inches. Figure 2.2 shows a typical SPT split spoon sampler. To perform SPT at a certain depth, the boring should be advanced to that depth, the drilling tool is then withdrawn, the hole is cleaned, the split spoon attached to the drilling rod should be inserted to that depth, the number of blows for three consecutive 6-inch of penetration should then be counted. The SPT "N-Value" is the summation of the number of blows for the last two 6-inch penetration increments.

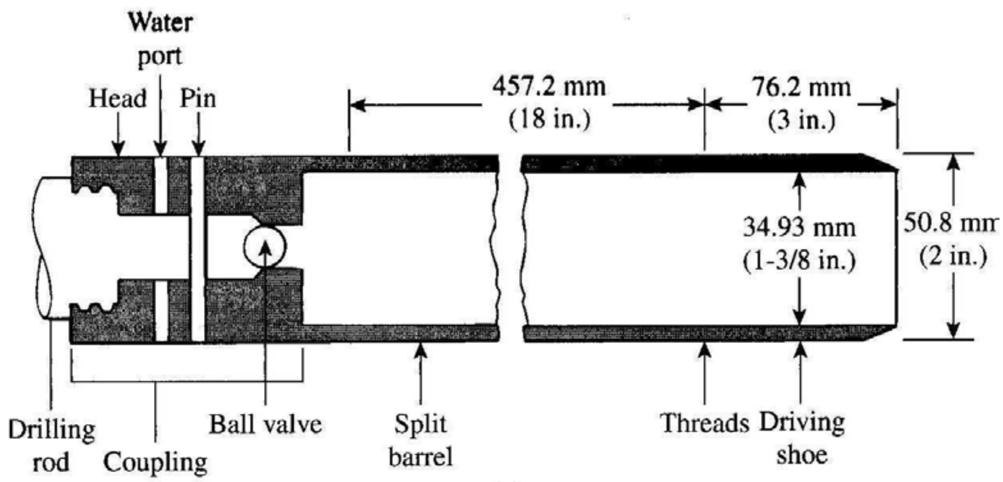


Figure 2.2 The split spoon sampler

2.4.1 SPT Corrections

The N-value measured in the field should be corrected for hammer type, sampler type (with or without liner), borehole diameter, and rod length. In addition to these corrections, the N-value measured for granular soils must be corrected for the effect of overburden pressure as follows:

$$(N_1)_{60} = C_N N_{60}$$

$$C_N = \sqrt{\frac{P_a}{\sigma'_o}} \qquad \text{or} \qquad C_N = \frac{2}{1 + \left(\frac{\sigma'_o}{P_a}\right)}$$

where N_{60} is the measured N-value corrected for hammer type, sampler type, borehole diameter, and rod length, $(N_1)_{60}$ is the N-value corrected for overburden, P_a is the atmospheric pressure, and σ_0 is the effective overburden pressure at the depth where N_{60} was measured.

2.4.2 SPT Correlations

The N-value was correlated to most of the strength and compressibility properties of both granular and cohesive soils. Tables 2.2 (Bowles 1996) is a very popular quick source for estimating angle of shearing resistance and density of sands. Table 2.3 describes the denseness of granular soils in terms of the N-value. For clayey soil, Table 2.4 provides consistency description as well as typical values for the unconfined compression strength as they correlate to the N-values. In addition, the following correlations may be used to estimate soil properties needed for foundation design:

$$D_r(\%) = \left(\frac{(N_1)_{60}}{(60 + 25\log(D_{50}))\left(1.2 + 0.05\log\left(\frac{t}{100}\right)\right)OCR^{0.18}} \right)^{0.5} \quad .....\ t = \text{time}, D_{50} \text{ from GSD.}$$

$$\phi'(\text{degree}) = 27.1 + 0.3N_{60} - 0.00054(N_{60})^2$$

$$\phi' = \tan^{-1}\left(\frac{N_{60}}{12.2 + 20.3\left(\frac{\sigma'_0}{P_a}\right)} \right)^{0.34}$$

$$\phi' = \sqrt{20(N_1)_{60}} + 20$$

$$\frac{E_s}{P_a} = \alpha N_{60} \quad .....\ \alpha = 5\ (\text{sand with fines}), \alpha = 10 (\text{clean NC sand}), \alpha = 15 (\text{clean OC sand})$$

$$C_u = (0.507 - 0.942)N_{60} \quad ..... \text{psi}$$

$$C_u = 29N_{60}^{0.72} \quad ..... \text{kPa}$$

$$OCR = 0.193\left(\frac{N_{60}}{\sigma'_o}\right)^{0.689} \quad ..... \sigma'_o \text{ in MPa} ... OCR = \frac{\sigma'_c}{\sigma'_o}$$

Table 2.2 Angle of shearing resistance of granular soils using N-values (Bowles 1996)

Description		Very loose	Loose	Medium	Dense	Very dense
Relative density D_r		0	0.15	0.35	0.65	0.85
SPT N'_{70}:	fine	1–2	3–6	7–15	16–30	?
	medium	2–3	4–7	8–20	21–40	> 40
	coarse	3–6	5–9	10–25	26–45	> 45
ϕ:	fine	26–28	28–30	30–34	33–38	
	medium	27–28	30–32	32–36	36–42	< 50
	coarse	28–30	30–34	33–40	40–50	
γ_{wet}, kN/m^3		11–16*	14–18	17–20	17–22	20–23

* Excavated soil or material dumped from a truck has a unit weight of 11 to 14 kN/m^3 and must be quite dense to weigh much over 21 kN/m^3. No existing soil has a D_r = 0.00 nor a value of 1.00. Common ranges are from 0.3 to 0.7.

Table 2.3 Denseness of granular soils as correlated to N-values

Standard penetration number, $(N_1)_{60}$	Approximate relative density, D_r, (%)
0–5	0–5
5–10	5–30
10–30	30–60
30–50	60–95

Table 2.4 Consistency and unconfined compression strength of cohesive soils using N-value

Standard penetration number, N_{60}	Consistency	CI	Unconfined compression strength, q_u	
			(kN/m^2)	(lb/ft^2)
<2	Very soft	<0.5	<25	500
2–8	Soft to medium	0.5–0.75	25–80	500–1700
8–15	Stiff	0.75–1.0	80–150	1700–3100
15–30	Very stiff	1.0–1.5	150–400	3100–8400
>30	Hard	>1.5	>400	8400

2.5 Cone Penetration Test (CPT)

The CPT is performed by statically pushing either a mechanical or electrical cone into the ground and measuring both the tip and sleeve resistances. The test does not need a borehole and there is no sample recovery. The penetration resistance is continuously recorded and saved to a computer and, later, printed. Figure 2.3 shows a schematic diagram for the main components of a piezocone as well as a sample of the recorded data for clayey soil. Figure 2.4 presents the classification chart used by most CPT software packages to classify soils using the tip/cone and sleeve/skin friction resistances.

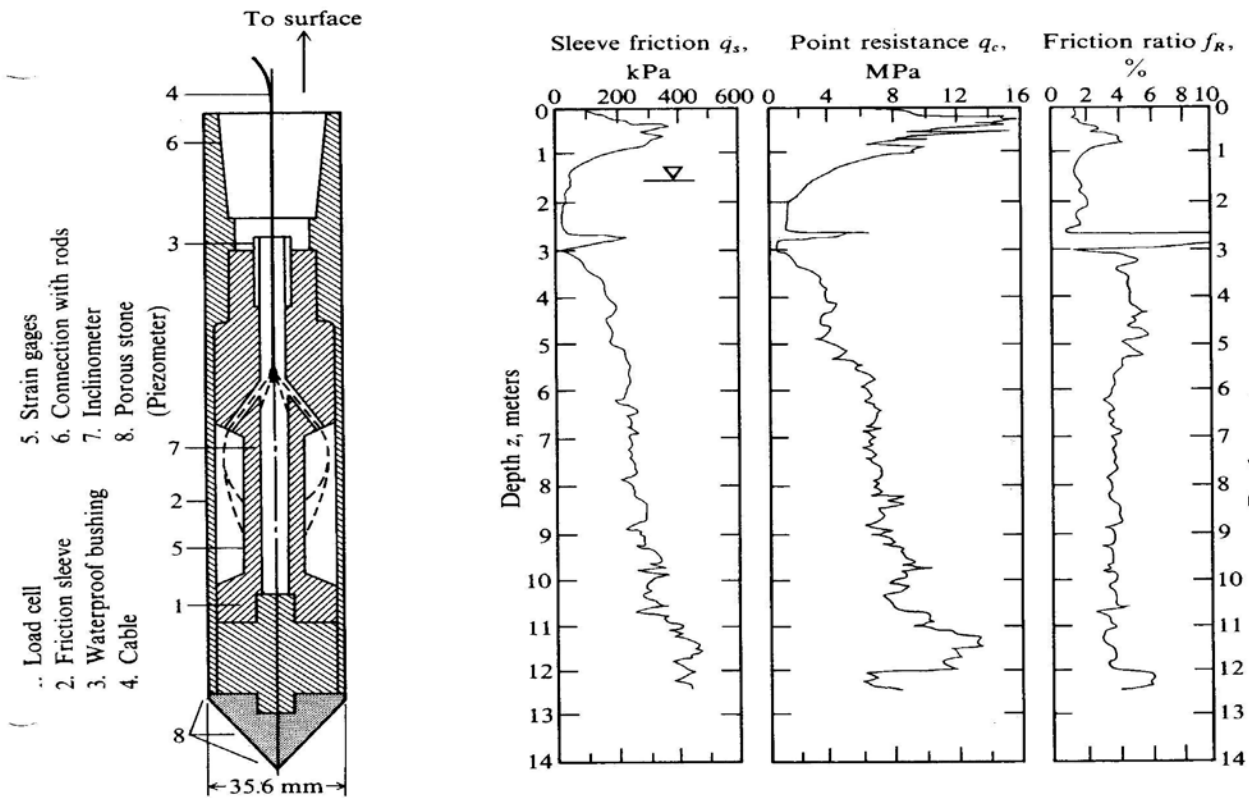


Figure 2.3 Schematic of a piezocone and sample of CPT output for clayey soil

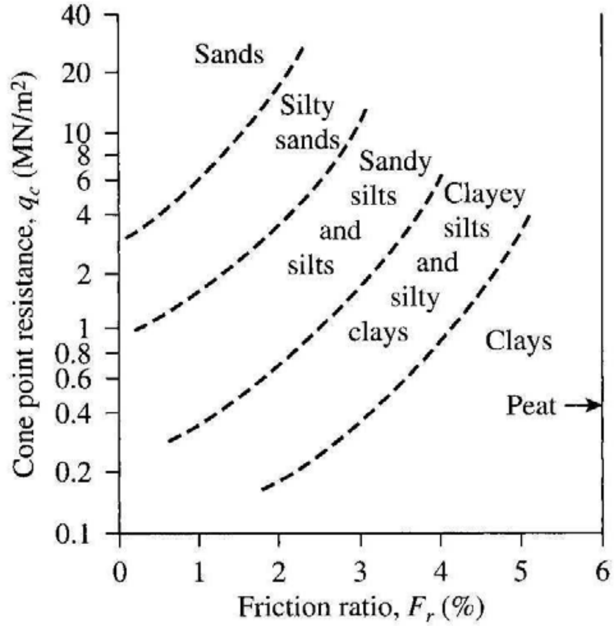


Figure 2.4 CPT soil classification chart

2.5.1 CPT Correlations

The cone penetration resistance (q_c) is correlated to most soil strength and compressibility properties. The following are some of the popular emperical correlations:

$$D_r = \sqrt{\left(\frac{1}{305Q_c OCR^{1.8}}\right)\left(\frac{\frac{q_c}{P_a}}{\left(\frac{\sigma'_o}{P_a}\right)^{0.5}}\right)} \quad \text{.....} \; Q_c = 0.91 - 1.09 \approx 1.0$$

$$\phi' = \tan^{-1}\left(0.1 + 0.38\log\left(\frac{q_c}{\sigma'_o}\right)\right) \quad \text{..... for Quatrtz Sand}$$

$$\phi' = \tan^{-1}\left(0.38 + 0.27\log\left(\frac{q_c}{\sigma'_o}\right)\right) \quad \text{..... for ML and SP - SM soils}$$

$$\phi' = 15.575\left(\frac{q_c}{\sigma'_h}\right)^{0.1714} \quad \text{.....} \; \sigma'_h \text{ is the horizontal effective pressure}$$

$$C_u = \frac{q_c - \sigma'_o}{N_k} \quad \text{.....} \; N_k = 15 - 17.2 \quad \text{(electric cone)}$$

$$\text{.....} \; N_k = 18.9 - 20 \quad \text{(mechanical cone)}$$

$$\sigma'_c = 0.243(q_c)^{0.96} \quad \text{..... in MN/m}^2$$

$$OCR = 0.37\left(\frac{q_c - \sigma_o}{\sigma'_o}\right)^{1.01}$$

2.5.2 CPT & SPT

The SPT N-value may be estimated from the CPT q_c-value and visa versa using the following correlation, which is graphically represented on Figure 2.5.

$$\frac{\left(\frac{q_c}{P_a}\right)}{N_{60}} = 7.6429\, D_{50}^{0.26}$$

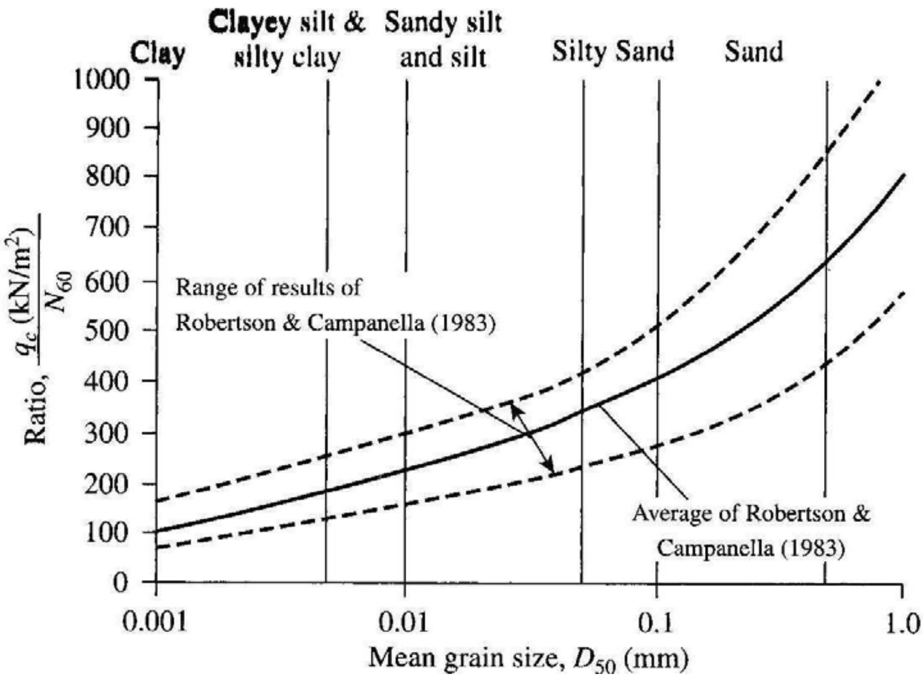


Figure 2.5 SPT & CPT correlation

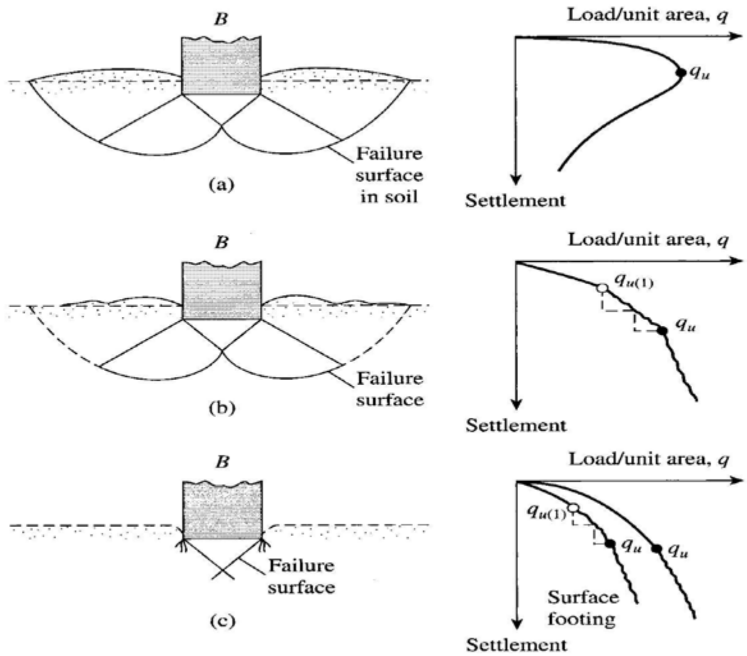
3 ULTIMATE BEARING CAPACITY OF SHALLOW FOUNDATIONS

3.1 Concept

An adequately designed shallow foundation system must:

- Have sufficient safety factor against shear failure (Bearing Capacity Failure)
- Control settlement,, both immediate and long term, within the allowable/tolerable limits as specified by the structural engineer.
- Provide an economical advantage over other type of foundations.
- Be constructible.

The ultimate bearing capacity of the soil is the minimum vertical stress that can result in failure shear. Depending on the soil type and density/consistency, as shown on Figure 3.1, the failure could be general shear failure such as in dense sands and stiff clays, local shear failure such as in loose to medium sands and medium clays, or punching shear such as in very loose sands and soft clays.



Nature of bearing capacity failure in soil: (a) general shear failure: (b) local shear failure; (c) punching shear failure (Redrawn after Vesic, 1973)

Figure 3.1 Types of shear failure

3.2 Terzaghi's Bearing Capacity Theory

Terzaghi (1943) presented the first comprehensive bearing capacity theory to evaluate the ultimate bearing capacity of shallow foundations. The foundation is shallow, according to Terzaghi, if its depth (D_f) is less than its width (B). Terzaghi assumed the failure surfaces shown on Figure 3.2 to occur for strip/continuous footing. The soil resistance along JH and GI was neglected, however, the effect of the overburden above the foundation level was considered. The failure zone generally consists of three parts:

- The triangle ACD immediately under the footing
- The radial shear zones ADF and CDE with the curve being log-spiral
- Two triangular Rankine passive zones AFH and CEG

Using equilibrium analysis, Terzaghi presented the following equations:

$q_{ult} = c'N_c + qN_q + 0.5\gamma BN_\gamma$ ..... for strip footing

$q_{ult} = 1.3c'N_c + qN_q + 0.4\gamma BN_\gamma$ ..... for square footing

$q_{ult} = 1.3c'N_c + qN_q + 0.3\gamma BN_\gamma$ ..... for circular footing

where; q_{ult} is the ultimate bearing capacity of the soil, c is the undrained cohesion, γ is the unit weight of the soil within the plastic zones, q is the effective overburden pressure at the foundation level, N_c, N_q, and N_γ are the bearing capacity factors defined by Terzaghi as follows:

$$N_c = \cot\phi'\left[\frac{e^{2\left(\frac{3\pi}{4}-\frac{\phi'}{2}\right)\tan\phi'}}{2\cos^2\left(\frac{\pi}{4}+\frac{\phi'}{2}\right)}-1\right] = \cot\phi'(N_q - 1)$$

$$N_q = \frac{e^{2\left(\frac{3\pi}{4}-\frac{\phi'}{2}\right)\tan\phi'}}{2\cos^2\left(\frac{\pi}{4}+\frac{\phi'}{2}\right)}$$

$$N_\gamma = \frac{1}{2}\left(\frac{K_{p\gamma}}{\cot^2\phi'}-1\right)\tan\phi'$$

where, $K_{p\gamma}$ is the passive earth pressure coefficient

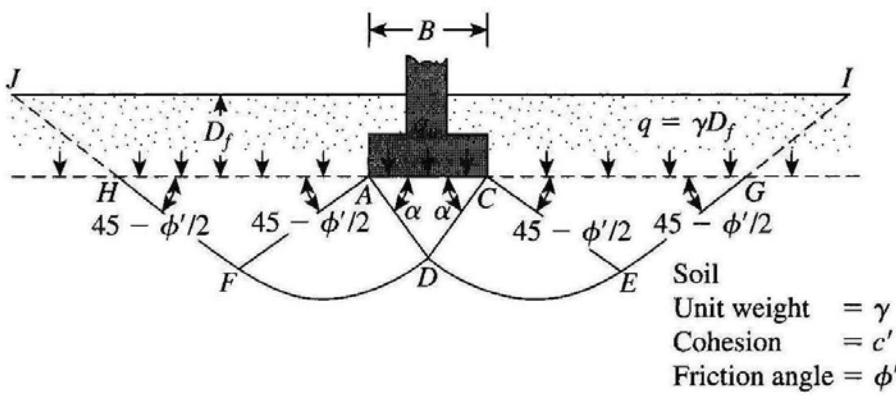


Figure 3.2 Terzaghi’s Bearing capacity Theory

Terzaghi’s bearing capacity factors may also be determined from Table 3.1.

Table 3.1 Terzaghi’s bearing capacity factors

ϕ'	N_c	N_q	N_γ[a]	ϕ'	N_c	N_q	N_γ[a]
0	5.70	1.00	0.00	26	27.09	14.21	9.84
1	6.00	1.10	0.01	27	29.24	15.90	11.60
2	6.30	1.22	0.04	28	31.61	17.81	13.70
3	6.62	1.35	0.06	29	34.24	19.98	16.18
4	6.97	1.49	0.10	30	37.16	22.46	19.13
5	7.34	1.64	0.14	31	40.41	25.28	22.65
6	7.73	1.81	0.20	32	44.04	28.52	26.87
7	8.15	2.00	0.27	33	48.09	32.23	31.94
8	8.60	2.21	0.35	34	52.64	36.50	38.04
9	9.09	2.44	0.44	35	57.75	41.44	45.41
10	9.61	2.69	0.56	36	63.53	47.16	54.36
11	10.16	2.98	0.69	37	70.01	53.80	65.27
12	10.76	3.29	0.85	38	77.50	61.55	78.61
13	11.41	3.63	1.04	39	85.97	70.61	95.03
14	12.11	4.02	1.26	40	95.66	81.27	115.31
15	12.86	4.45	1.52	41	106.81	93.85	140.51
16	13.68	4.92	1.82	42	119.67	108.75	171.99
17	14.60	5.45	2.18	43	134.58	126.50	211.56
18	15.12	6.04	2.59	44	151.95	147.74	261.60
19	16.56	6.70	3.07	45	172.28	173.28	325.34
20	17.69	7.44	3.64	46	196.22	204.19	407.11
21	18.92	8.26	4.31	47	224.55	241.80	512.84
22	20.27	9.19	5.09	48	258.28	287.85	650.67
23	21.75	10.23	6.00	49	298.71	344.63	831.99
24	23.36	11.40	7.08	50	347.50	415.14	1072.80
25	25.13	12.72	8.34				

[a]From Kumbhojkar (1993)

3.3 Allowable Bearing Capacity

It should be noted that more than one method should be used to estimate the ultimate bearing capacity. Famous methods include Terzaghi, Meyerhof, Hansen, Vesic, and the general bearing capacity equation. After the ultimate bearing capacity is estimated, the engineer should apply an appropriate safety factor against shear in order to provide the structural engineer with the allowable bearing pressure for design. The gross allowable bearing capacity can be estimated as follows:

$$q_{all} = \frac{q_{ult}}{FS} \quad \ldots\ldots \quad FS = 2.5 \text{ to } 3$$

Some geotechnical engineers prefer to use the net allowable bearing pressure since the safety factor should not be applied for the overburden pressure. The net allowable bearing pressure may be estimated as follows:

$$q_{ult(net)} = q_{ult} - q$$

$$q_{all(net)} = \frac{q_{ult(net)}}{FS} \;\; ..... \;\; FS = 2.5 \text{ to } 3$$

The safety factor may also be applied to the shear strength parameters of the foundation soil (FS_{shear}). In most cases, FS_{shear} = 1.4 to 1.6. The shear strength parameters will be reduced and the reduced values should be used to estimate the bearing capacity as follows:

$$C'_d = \frac{C'}{FS_{shear}}$$

$$\varphi'_d = \tan^{-1}\left(\frac{\tan\varphi'}{FS_{shear}}\right)$$

3.4 The Design Equation

The contact pressure transferred to the soil by the column/footing system should not exceed the net allowable bearing pressure of the soil. The optimum design is when the contact pressure is equal to the net allowable bearing pressure as follows:

$$\frac{P_{col}}{A_f} = q_{all(net)} \quad \rightarrow \quad \frac{P_{col}}{A_f} = \frac{q_{ult} - q}{FS}$$

where, P_{col} is the column load and A_f is the footing contact area

3.5 Effect of the Groundwater Table (GWT)

The effect of the groundwater table depends on the relative depth of the GWT as compared to the depth to the foundation level (D_f) and the depth of failure zones (D). Three cases, as shown on Figure 3.3, may be encountered:

<u>GWT is located between the ground surface and the foundation level:</u>

In this case, the overburden pressure at the foundation level should be calculated using the submerged unit weight of the soil. Also, the submerged unit weight of the soil within the failure zones should be used as follows:

$$q = \gamma D_1 + \gamma_{sub} D_2$$

$$\gamma \rightarrow\rightarrow \gamma_{sub}$$

<u>GWT is located below the foundation level within the failure zones:</u>

In this case, there is no change to the overburden pressure; however, for the third term of the bearing capacity equation, an equivalent unit weight should be calculated as follows:

$$\gamma^* = \gamma_{sub} + \frac{d}{B}\left(\gamma - \gamma_{sub}\right)$$

<u>GWT is below the failure zones:</u>

In this case, there is no effect for the GWT on the Bearing Capacity of the soil.

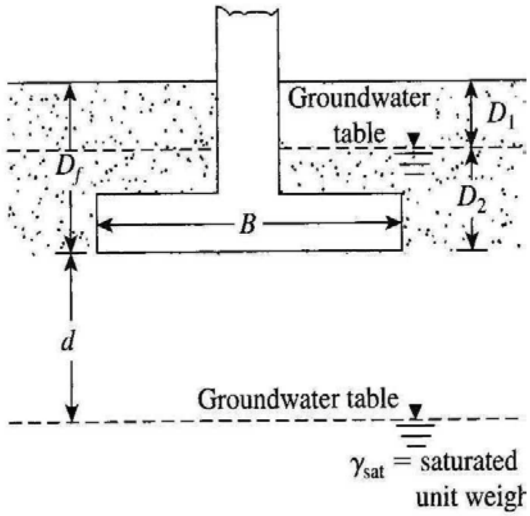


Figure 3.3 Effect of GWT on the soil bearing capacity

3.6 The General Bearing Capacity Equation

In order to take into consideration footings with rectangular shapes, footings with inclined loads, the resistance provided from the portion of the failure surfaces above the foundation level, Meyerhof (1963) presented the general bearing capacity equation as follows:

$$q_{ult} = c' N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qi} + 0.5 \gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}$$

where, N_c, N_q, and N_γ are the bearing capacity factors , F_{cs}, F_{qs}, $F_{\gamma s}$ are the shape factors, F_{cd}, F_{qd}, $F_{\gamma d}$ are the depth factors, and F_{ci}, F_{qi}, $F_{\gamma i}$ are the inclination factors defined as follows:

<u>Bearing Capacity Factors:</u>

$$N_q = \tan^2\left(45 + \frac{\phi'}{2}\right)e^{\pi \tan\phi'}$$

$$N_c = \cot\phi'(N_q - 1)$$

$$N_\gamma = 2(N_q + 1)\tan\phi'$$

Shape Factors:

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

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

$$F_{\gamma s} = 1 - 0.4\left(\frac{B}{L}\right)$$

Depth Factors:

For $D_f/B < 1$:

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

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

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

For $D_f/B \geq 1$:

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

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

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

<u>Inclination Factors:</u>

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

$$F_{\gamma i} = \left(1 - \frac{\beta^\circ}{\phi^\circ}\right)^2$$

4 SETTLEMENT OF SHALLOW FOUNDATIONS

In most cases, especially when the foundation soil has considerably high shear strength, settlement controls the recommended net allowable bearing pressure for shallow foundations. This may be explained by understanding that, even though there is still a good margin to the allowable bearing pressure, the associated settlement with high contact pressures may results in intolerable settlements that may be harmful for the structural framing system.

4.1 Types of Settlement

Foundation settlement may be divided into immediate settlement, also called initial or elastic settlement, and consolidation settlement, which include both primary and secondary consolidation settlements. Initial settlement comprises most of the settlement that occurs in sandy soils, whereas primary consolidation settlement comprises most of the settlement in clayey soils. Secondary consolidation settlement comprises most of the settlement in organic soils.

4.2 Stress Distribution

Settlement results from the increase in effective stresses in compressible soils. The foundation loads/pressures are transmitted to the foundation soil at the foundation level, which is usually a few feet below grade in case of shallow foundations. In order to estimate the settlement of a specific layer due to a foundation load, the stress increase within this specific layer should be calculated. There are a few methods to estimate the stress increase at different locations and different depths in a soil mass. Some of the simple and commonly used methods are explained below.

4.2.1 Boussinesq Solution

This solution is for a concentrated load at ground surface. The stress increase due to a footing of any shape may be solved using Boussinesq's solution by dividing the footing into small areas and dealing with each area as a concentrated load and then add the effect of all the small areas to get the stress increase as a result of the footing. The following equation provides the vertical stress increase at Point “A” at a radial distance of “r” and at a depth “z” from the concentrated load “P” (Figure 4.1) according to Boussinesq solution:

$$\Delta\sigma = \frac{3P}{2\pi z^2\left(1+\left(\frac{r}{z}\right)^2\right)^{\frac{5}{2}}}$$

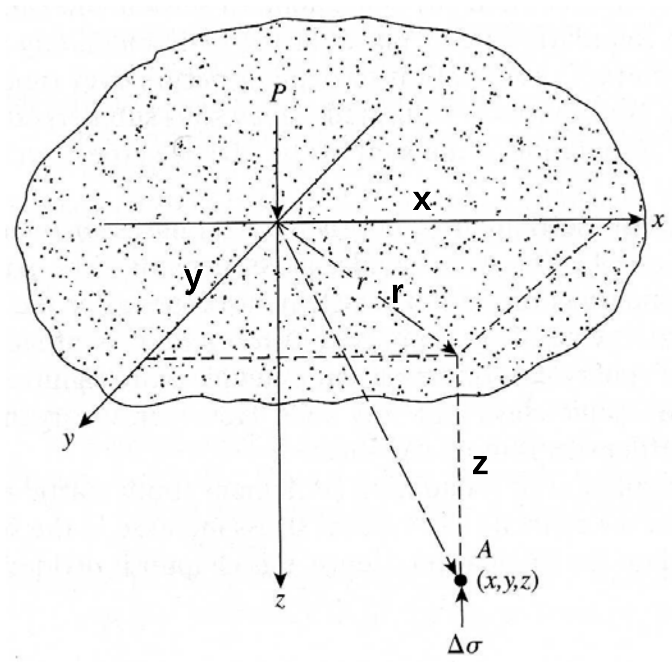


Figure 4.1 Stresses due to point load (Boussinesq solution)

4.2.2 Average Stresses below the Corner of Rectangular Footings in a Surface Layer

This method provides an average value for the vertical stress increase within a surface layer, however, the solution is only available for points below the corner of the footing. For other locations, the footing may be divided into a few small footings such that the target point is at each small footing’s corner. The stress increase from each footing should be added to get the final stress increase at the target point. The stress increase may be calculated as follows:

$$\Delta\sigma = q_0 I_a$$

To estimate the value of the influence factor I_a, calculate B/H and L/H and use Figure 4.2. Figure 4.3 defines L, B, and H.

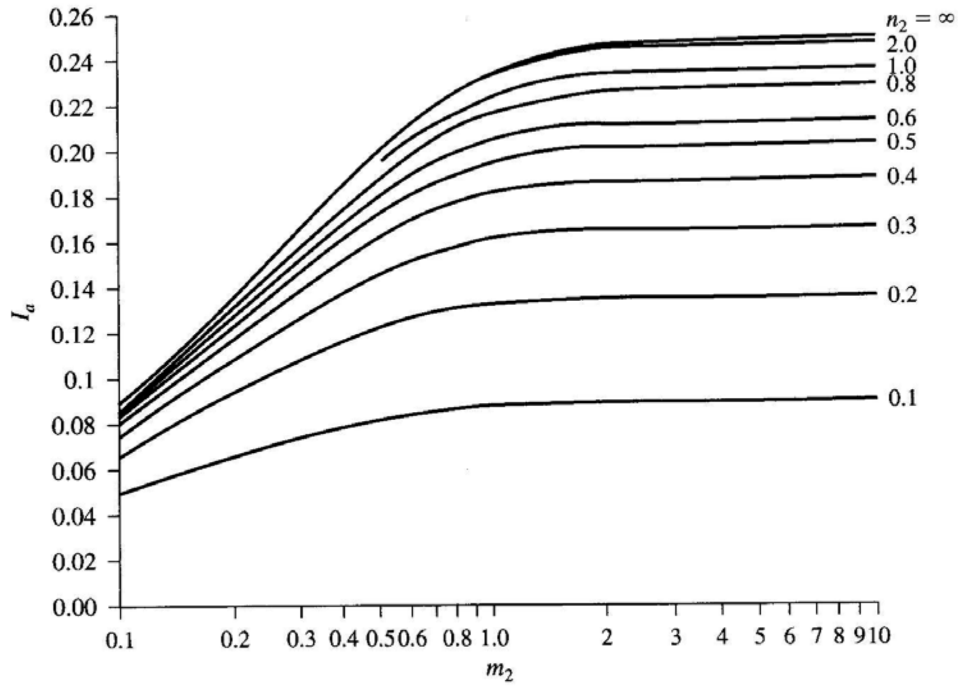


Figure 4.2 Influence factor for estimating average stress increase in surface layer

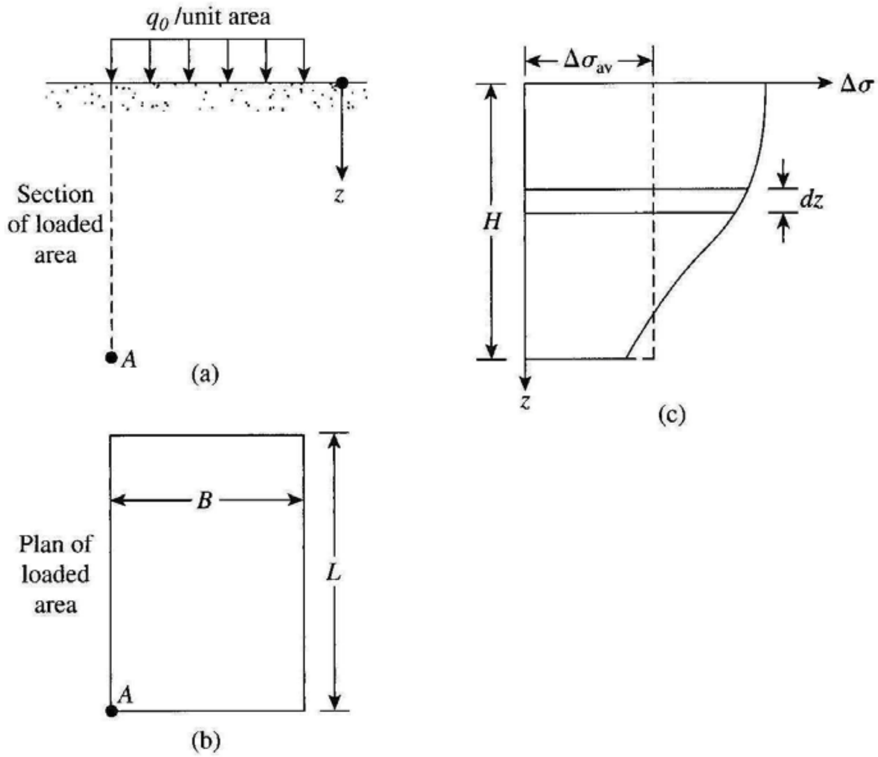


Figure 4.3 Definition of B, L, and H for estimating average stress increase in surface layer

4.2.3 Average Stresses below the Corner of Rectangular Footings in a Deep Layer

Using the same procedure for surface layer, solve two times for H_1 and H_2 (Figure 4.4), then substitute in the following equation to calculate the average stress increase within the deep layer:

$$\Delta\sigma = q_0\left[\frac{H_2 I_{a(H_2)} - H_1 I_{a(H_1)}}{H_2 - H_1}\right]$$

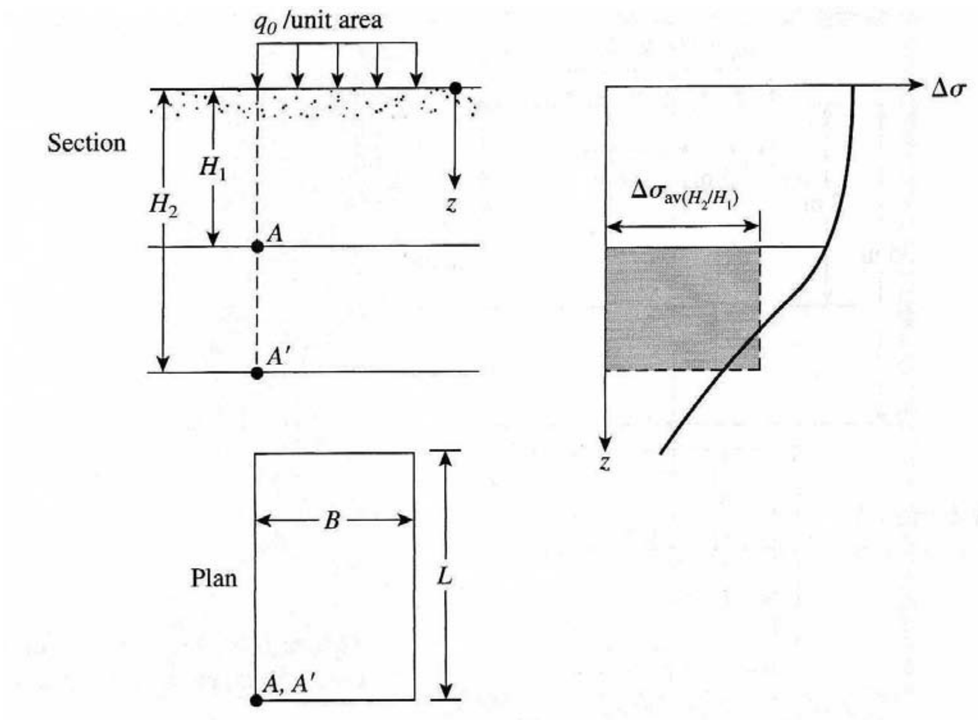


Figure 4.4 Definition of B, L, H_1, and H_2 for estimating average stress increase in deep layer

4.2.4 2:1 Line Method

This is an approximate, quick, and very popular method to estimate the stress increase at a specific depth. The stress increase estimated using this method will be equal at all points at the same depth, which is a somewhat unacceptable approximation. The method assumes that the vertical stress spreads out along lines with a vertical-to-horizontal slope of 2:1 (Figure 4.5). At a depth “z” below a rectangular footing of length “L” and width “B” may be calculated as follows:

$$\Delta\sigma = \frac{q_0 \times A_{F_{\text{foundation level}}}}{A_{F_{\text{at depth z from foundation level}}}} = \frac{q_0 \times B \times L}{(B+z)(L+z)}$$

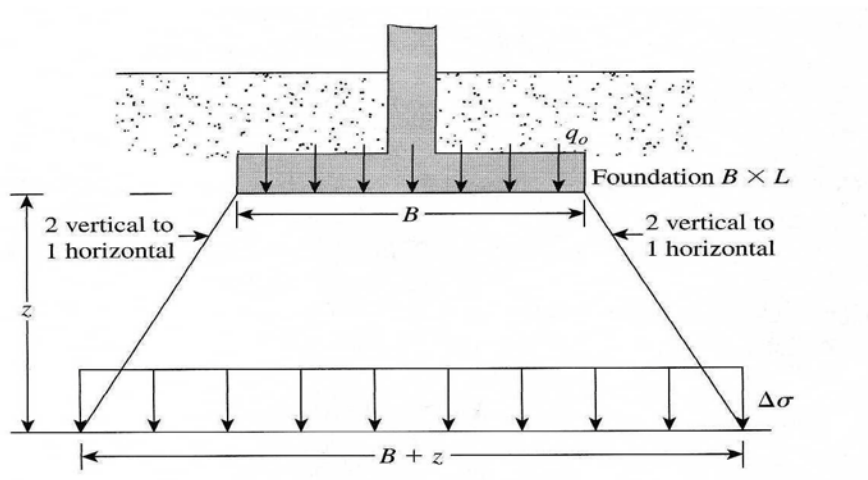


Figure 4.5 2:1 Line method

4.3 Elastic Settlement

4.3.1 Solution Based on Theory of Elasticity

The elastic settlement may be estimated using the theory of elasticity. From Hooke's law and assuming that the foundation is perfectly flexible (Figure 4.6), the elastic settlement may be estimated as follows:

$$S_e = q_0 \alpha B' \frac{1-\mu_s^2}{E_s} I_s I_f$$

where;

q_0 is the net contact pressure at the foundation level

μ_s is Poisson's ratio of the soil

E_s is the average modulus of elasticity (0 to 4B below FL)

$B^{/}$ is 0.5B for center & B for corner of footing

$I_f = f(D_f, B, L)$ and can be estimated from Figure 4.7

I_s is the shape factor = $f(L,B,H,\mu_s)$ and can be determined using the following equation:

$$I_s = F_1 + \frac{1-2\mu_s}{1-\mu_s} F_2$$

To get F_1 and F_2, use Table 4.1 and the following definitions for $m^{/}$, $n^{/}$ and α:

At center of footing:

$$\alpha = 4 \;,\; m' = \frac{L}{B} \;,\; n' = \frac{2H}{B}$$

At corner of footing:

$$\alpha = 1 \;,\; m' = \frac{L}{B} \;,\; n' = \frac{H}{B}$$

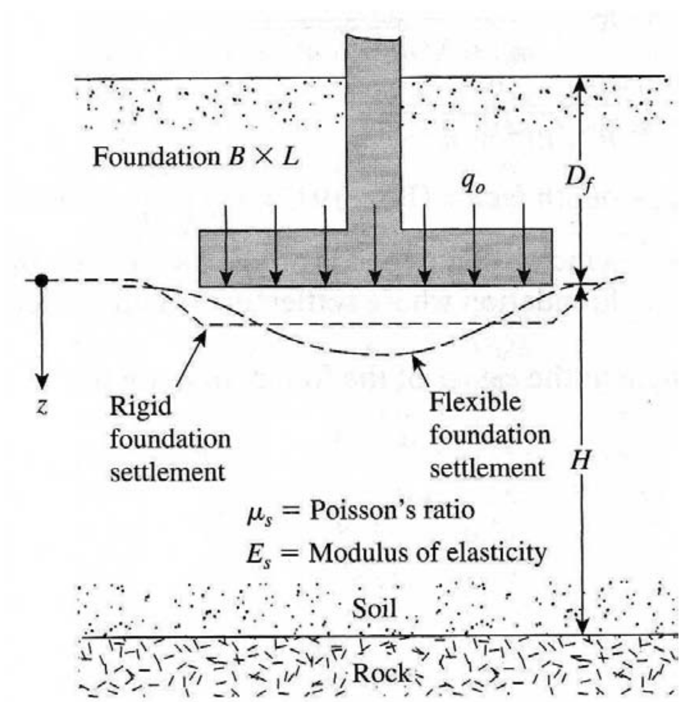


Figure 4.6 Elastic settlement using the theory of elasticity

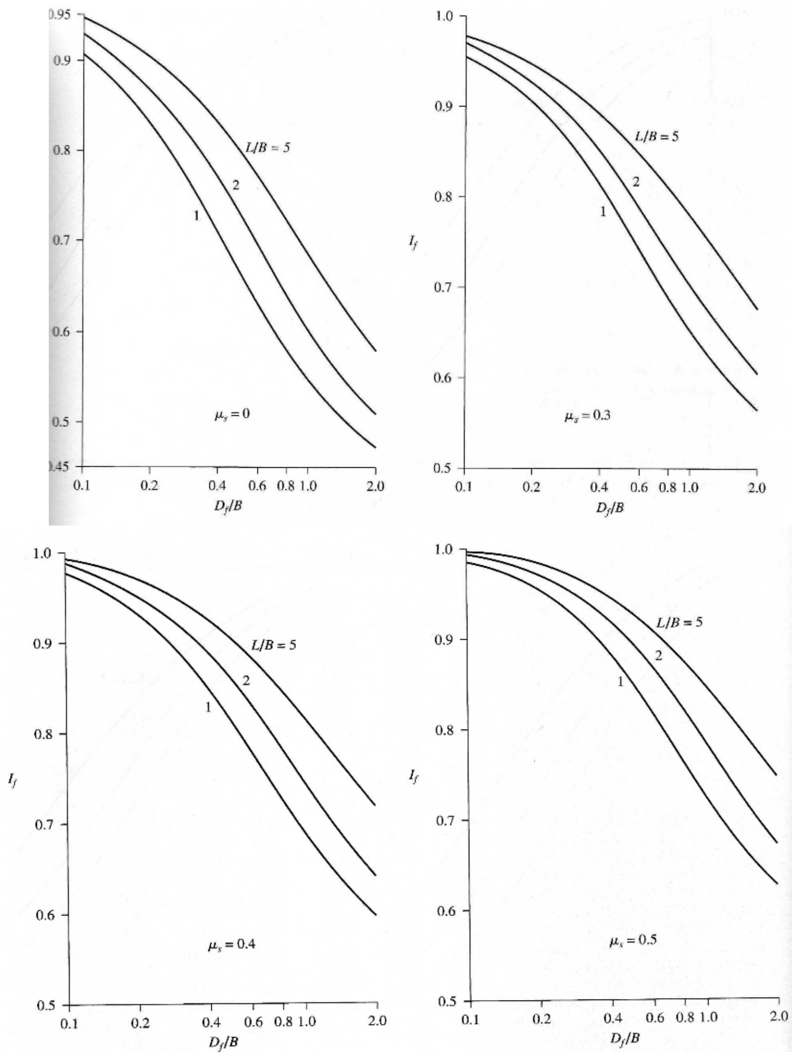


Figure 4.7 Factor If for different Poisson's ratios

In case of rigid footing, the previous method may be used to estimate the settlement at the center of a flexible footing similar to the rigid footing geometry then, this formula may be applied:

$$S_{e(rigid)} \approx 0.93 S_{e(\text{flexible at the center of the footing})}$$

In case of non-homogeneous soil, the average weighted modulus of elasticity may be used:

$E_s = \dfrac{\Sigma E_{s(i)} \Delta z_i}{\Sigma \Delta z_i}$, where Δz_i is the thickness of each layer.

Table 4.1 Values of F_1 and F_2

Table 5.4 Variation of F_1 with m' and n'

n'	m' = 1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	3.5	4.0
0.25	0.014	0.013	0.012	0.011	0.011	0.011	0.010	0.010	0.010	0.010
0.50	0.049	0.046	0.044	0.042	0.041	0.040	0.038	0.038	0.037	0.037
0.75	0.095	0.090	0.087	0.084	0.082	0.080	0.077	0.076	0.074	0.074
1.00	0.142	0.138	0.134	0.130	0.127	0.125	0.121	0.118	0.116	0.115
1.25	0.186	0.183	0.179	0.176	0.173	0.170	0.165	0.161	0.158	0.157
1.50	0.224	0.224	0.222	0.219	0.216	0.213	0.207	0.203	0.199	0.197
1.75	0.257	0.259	0.259	0.258	0.255	0.253	0.247	0.242	0.238	0.235
2.00	0.285	0.290	0.292	0.292	0.291	0.289	0.284	0.279	0.275	0.271
2.25	0.309	0.317	0.321	0.323	0.323	0.322	0.317	0.313	0.308	0.305
2.50	0.330	0.341	0.347	0.350	0.351	0.351	0.348	0.344	0.340	0.336
2.75	0.348	0.361	0.369	0.374	0.377	0.378	0.377	0.373	0.369	0.365
3.00	0.363	0.379	0.389	0.396	0.400	0.402	0.402	0.400	0.396	0.392
3.25	0.376	0.394	0.406	0.415	0.420	0.423	0.426	0.424	0.421	0.418
3.50	0.388	0.408	0.422	0.431	0.438	0.442	0.447	0.447	0.444	0.441
3.75	0.399	0.420	0.436	0.447	0.454	0.460	0.467	0.458	0.466	0.464
4.00	0.408	0.431	0.448	0.460	0.469	0.476	0.484	0.487	0.486	0.484
4.25	0.417	0.440	0.458	0.472	0.481	0.484	0.495	0.514	0.515	0.515
4.50	0.424	0.450	0.469	0.484	0.495	0.503	0.516	0.521	0.522	0.522
4.75	0.431	0.458	0.478	0.494	0.506	0.515	0.530	0.536	0.539	0.539
5.00	0.437	0.465	0.487	0.503	0.516	0.526	0.543	0.551	0.554	0.554
5.25	0.443	0.472	0.494	0.512	0.526	0.537	0.555	0.564	0.568	0.569
5.50	0.448	0.478	0.501	0.520	0.534	0.546	0.566	0.576	0.581	0.584
5.75	0.453	0.483	0.508	0.527	0.542	0.555	0.576	0.588	0.594	0.597
6.00	0.457	0.489	0.514	0.534	0.550	0.563	0.585	0.598	0.606	0.609
6.25	0.461	0.493	0.519	0.540	0.557	0.570	0.594	0.609	0.617	0.621
6.50	0.465	0.498	0.524	0.546	0.563	0.577	0.603	0.618	0.627	0.632
6.75	0.468	0.502	0.529	0.551	0.569	0.584	0.610	0.627	0.637	0.643
7.00	0.471	0.506	0.533	0.556	0.575	0.590	0.618	0.635	0.646	0.653
7.25	0.474	0.509	0.538	0.561	0.580	0.596	0.625	0.643	0.655	0.662
7.50	0.477	0.513	0.541	0.565	0.585	0.601	0.631	0.650	0.663	0.671
7.75	0.480	0.516	0.545	0.569	0.589	0.606	0.637	0.658	0.671	0.680
8.00	0.482	0.519	0.549	0.573	0.594	0.611	0.643	0.664	0.678	0.688
8.25	0.485	0.522	0.552	0.577	0.598	0.615	0.648	0.670	0.685	0.695
8.50	0.487	0.524	0.555	0.580	0.601	0.619	0.653	0.676	0.692	0.703
8.75	0.489	0.527	0.558	0.583	0.605	0.623	0.658	0.682	0.698	0.710
9.00	0.491	0.529	0.560	0.587	0.609	0.627	0.663	0.687	0.705	0.716
9.25	0.493	0.531	0.563	0.589	0.612	0.631	0.667	0.693	0.710	0.723
9.50	0.495	0.533	0.565	0.592	0.615	0.634	0.671	0.697	0.716	0.719
9.75	0.496	0.536	0.568	0.595	0.618	0.638	0.675	0.702	0.721	0.735
10.00	0.498	0.537	0.570	0.597	0.621	0.641	0.679	0.707	0.726	0.740
20.00	0.529	0.575	0.614	0.647	0.677	0.702	0.756	0.797	0.830	0.858
50.00	0.548	0.598	0.640	0.678	0.711	0.740	0.803	0.853	0.895	0.931
100.00	0.555	0.605	0.649	0.688	0.722	0.753	0.819	0.872	0.918	0.956

n'	m' = 4.5	5.0	6.0	7.0	8.0	9.0	10.0	25.0	50.0	100.0
0.25	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010
0.50	0.036	0.036	0.036	0.036	0.036	0.036	0.036	0.036	0.036	0.036
0.75	0.073	0.073	0.072	0.072	0.072	0.072	0.071	0.071	0.071	0.071
1.00	0.114	0.113	0.112	0.112	0.112	0.111	0.111	0.110	0.110	0.110
1.25	0.155	0.154	0.153	0.152	0.152	0.151	0.151	0.150	0.150	0.150
1.50	0.195	0.194	0.192	0.191	0.190	0.190	0.189	0.188	0.188	0.188
1.75	0.233	0.232	0.229	0.228	0.227	0.226	0.225	0.223	0.223	0.223
2.00	0.269	0.267	0.264	0.262	0.261	0.260	0.259	0.257	0.256	0.256
2.25	0.302	0.300	0.296	0.294	0.293	0.291	0.291	0.287	0.287	0.287
2.50	0.333	0.331	0.327	0.324	0.322	0.321	0.320	0.316	0.315	0.315
2.75	0.362	0.359	0.355	0.352	0.350	0.348	0.347	0.343	0.342	0.342
3.00	0.389	0.386	0.382	0.378	0.376	0.374	0.373	0.368	0.367	0.367
3.25	0.415	0.412	0.407	0.403	0.401	0.399	0.397	0.391	0.390	0.390
3.50	0.438	0.435	0.430	0.427	0.424	0.421	0.420	0.413	0.412	0.411
3.75	0.461	0.458	0.453	0.449	0.446	0.443	0.441	0.433	0.432	0.432
4.00	0.482	0.479	0.474	0.470	0.466	0.464	0.462	0.453	0.451	0.451
4.25	0.516	0.496	0.484	0.473	0.471	0.471	0.470	0.468	0.462	0.460
4.50	0.520	0.517	0.513	0.508	0.505	0.502	0.499	0.489	0.487	0.487
4.75	0.537	0.535	0.530	0.526	0.523	0.519	0.517	0.506	0.504	0.503
5.00	0.554	0.552	0.548	0.543	0.540	0.536	0.534	0.522	0.519	0.519
5.25	0.569	0.568	0.564	0.560	0.556	0.553	0.550	0.537	0.534	0.534
5.50	0.584	0.583	0.579	0.575	0.571	0.568	0.585	0.551	0.549	0.548
5.75	0.597	0.597	0.594	0.590	0.586	0.583	0.580	0.565	0.583	0.562
6.00	0.611	0.610	0.608	0.604	0.601	0.598	0.595	0.579	0.576	0.575
6.25	0.623	0.623	0.621	0.618	0.615	0.611	0.608	0.592	0.589	0.588
6.50	0.635	0.635	0.634	0.631	0.628	0.625	0.622	0.605	0.601	0.600
6.75	0.646	0.647	0.646	0.644	0.641	0.637	0.634	0.617	0.613	0.612
7.00	0.656	0.658	0.658	0.656	0.653	0.650	0.647	0.628	0.624	0.623
7.25	0.666	0.669	0.669	0.668	0.665	0.662	0.659	0.640	0.635	0.634
7.50	0.676	0.679	0.680	0.679	0.676	0.673	0.670	0.651	0.646	0.645
7.75	0.685	0.688	0.690	0.689	0.687	0.684	0.681	0.661	0.656	0.655
8.00	0.694	0.697	0.700	0.700	0.698	0.695	0.692	0.672	0.666	0.665
8.25	0.702	0.706	0.710	0.710	0.708	0.705	0.703	0.682	0.676	0.675
8.50	0.710	0.714	0.719	0.719	0.718	0.715	0.713	0.692	0.686	0.684
8.75	0.717	0.722	0.727	0.728	0.727	0.725	0.723	0.701	0.695	0.693
9.00	0.725	0.730	0.736	0.737	0.736	0.735	0.732	0.710	0.704	0.702
9.25	0.731	0.737	0.744	0.746	0.745	0.744	0.742	0.719	0.713	0.711
9.50	0.738	0.744	0.752	0.754	0.754	0.753	0.751	0.728	0.721	0.719
9.75	0.744	0.751	0.759	0.762	0.762	0.761	0.759	0.737	0.729	0.727
10.00	0.750	0.758	0.766	0.770	0.770	0.770	0.768	0.745	0.738	0.735
20.00	0.878	0.896	0.925	0.945	0.959	0.969	0.977	0.982	0.965	0.957
50.00	0.962	0.989	1.034	1.070	1.100	1.125	1.146	1.265	1.279	1.261
100.00	0.990	1.020	1.072	1.114	1.150	1.182	1.209	1.408	1.489	1.499

Table 4.1 Continue

Table 5.5 Variation of F_2 with m' and n'

n'	m' = 1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	3.5	4.0
0.25	0.049	0.050	0.051	0.051	0.051	0.052	0.052	0.052	0.052	0.052
0.50	0.074	0.077	0.080	0.081	0.083	0.084	0.086	0.086	0.0878	0.087
0.75	0.083	0.089	0.093	0.097	0.099	0.101	0.104	0.106	0.107	0.108
1.00	0.083	0.091	0.098	0.102	0.106	0.109	0.114	0.117	0.119	0.120
1.25	0.080	0.089	0.096	0.102	0.107	0.111	0.118	0.122	0.125	0.127
1.50	0.075	0.084	0.093	0.099	0.105	0.110	0.118	0.124	0.128	0.130
1.75	0.069	0.079	0.088	0.095	0.101	0.107	0.117	0.123	0.128	0.131
2.00	0.064	0.074	0.083	0.090	0.097	0.102	0.114	0.121	0.127	0.131
2.25	0.059	0.069	0.077	0.085	0.092	0.098	0.110	0.119	0.125	0.130
2.50	0.055	0.064	0.073	0.080	0.087	0.093	0.106	0.115	0.122	0.127
2.75	0.051	0.060	0.068	0.076	0.082	0.089	0.102	0.111	0.119	0.125
3.00	0.048	0.056	0.064	0.071	0.078	0.084	0.097	0.108	0.116	0.122
3.25	0.045	0.053	0.060	0.067	0.074	0.080	0.093	0.104	0.112	0.119
3.50	0.042	0.050	0.057	0.064	0.070	0.076	0.089	0.100	0.109	0.116
3.75	0.040	0.047	0.054	0.060	0.067	0.073	0.086	0.096	0.105	0.113
4.00	0.037	0.044	0.051	0.057	0.063	0.069	0.082	0.093	0.102	0.110
4.25	0.036	0.042	0.049	0.055	0.061	0.066	0.079	0.090	0.099	0.107
4.50	0.034	0.040	0.046	0.052	0.058	0.063	0.076	0.086	0.096	0.104
4.75	0.032	0.038	0.044	0.050	0.055	0.061	0.073	0.083	0.093	0.101
5.00	0.031	0.036	0.042	0.048	0.053	0.058	0.070	0.080	0.090	0.098
5.25	0.029	0.035	0.040	0.046	0.051	0.056	0.067	0.078	0.087	0.095
5.50	0.028	0.033	0.039	0.044	0.049	0.054	0.065	0.075	0.084	0.092
5.75	0.027	0.032	0.037	0.042	0.047	0.052	0.063	0.073	0.082	0.090
6.00	0.026	0.031	0.036	0.040	0.045	0.050	0.060	0.070	0.079	0.087
6.25	0.025	0.030	0.034	0.039	0.044	0.048	0.058	0.068	0.077	0.085
6.50	0.024	0.029	0.033	0.038	0.042	0.046	0.056	0.066	0.075	0.083
6.75	0.023	0.028	0.032	0.036	0.041	0.045	0.055	0.064	0.073	0.080
7.00	0.022	0.027	0.031	0.035	0.039	0.043	0.053	0.062	0.071	0.078
7.25	0.022	0.026	0.030	0.034	0.038	0.042	0.051	0.060	0.069	0.076
7.50	0.021	0.025	0.029	0.033	0.037	0.041	0.050	0.059	0.067	0.074
7.75	0.020	0.024	0.028	0.032	0.036	0.039	0.048	0.057	0.065	0.072
8.00	0.020	0.023	0.027	0.031	0.035	0.038	0.047	0.055	0.063	0.071
8.25	0.019	0.023	0.026	0.030	0.034	0.037	0.046	0.054	0.062	0.069
8.50	0.018	0.022	0.026	0.029	0.033	0.036	0.045	0.053	0.060	0.067
8.75	0.018	0.021	0.025	0.028	0.032	0.035	0.043	0.051	0.059	0.066
9.00	0.017	0.021	0.024	0.028	0.031	0.034	0.042	0.050	0.057	0.064
9.25	0.017	0.020	0.024	0.027	0.030	0.033	0.041	0.049	0.056	0.063
9.50	0.017	0.020	0.023	0.026	0.029	0.033	0.040	0.048	0.055	0.061
9.75	0.016	0.019	0.023	0.026	0.029	0.032	0.039	0.047	0.054	0.060
10.00	0.016	0.019	0.022	0.025	0.028	0.031	0.038	0.046	0.052	0.059
20.00	0.008	0.010	0.011	0.013	0.014	0.016	0.020	0.024	0.027	0.031
50.00	0.003	0.004	0.004	0.005	0.006	0.006	0.008	0.010	0.011	0.013
100.00	0.002	0.002	0.002	0.003	0.003	0.003	0.004	0.005	0.006	0.006

n'	m' = 4.5	5.0	6.0	7.0	8.0	9.0	10.0	25.0	50.0	100.0
0.25	0.053	0.053	0.053	0.053	0.053	0.053	0.053	0.053	0.053	0.053
0.50	0.087	0.087	0.088	0.088	0.088	0.088	0.088	0.088	0.088	0.088
0.75	0.109	0.109	0.109	0.110	0.110	0.110	0.110	0.111	0.111	0.111
1.00	0.121	0.122	0.123	0.123	0.124	0.124	0.124	0.125	0.125	0.125
1.25	0.128	0.130	0.131	0.132	0.132	0.133	0.133	0.134	0.134	0.134
1.50	0.132	0.134	0.136	0.137	0.138	0.138	0.139	0.140	0.140	0.140
1.75	0.134	0.136	0.138	0.140	0.141	0.142	0.142	0.144	0.144	0.145
2.00	0.134	0.136	0.139	0.141	0.143	0.144	0.145	0.147	0.147	0.148
2.25	0.133	0.136	0.140	0.142	0.144	0.145	0.146	0.149	0.150	0.150
2.50	0.132	0.135	0.139	0.142	0.144	0.146	0.147	0.151	0.151	0.151
2.75	0.130	0.133	0.138	0.142	0.144	0.146	0.147	0.152	0.152	0.153
3.00	0.127	0.131	0.137	0.141	0.144	0.145	0.147	0.152	0.153	0.154
3.25	0.125	0.129	0.135	0.140	0.143	0.145	0.147	0.153	0.154	0.154
3.50	0.122	0.126	0.133	0.138	0.142	0.144	0.146	0.153	0.155	0.155
3.75	0.119	0.124	0.131	0.137	0.141	0.143	0.145	0.154	0.155	0.155
4.00	0.116	0.121	0.129	0.135	0.139	0.142	0.145	0.154	0.155	0.156
4.25	0.113	0.119	0.127	0.133	0.138	0.141	0.144	0.154	0.156	0.156
4.50	0.110	0.116	0.125	0.131	0.136	0.140	0.143	0.154	0.156	0.156
4.75	0.107	0.113	0.123	0.130	0.135	0.139	0.142	0.154	0.156	0.157
5.00	0.105	0.111	0.120	0.128	0.133	0.137	0.140	0.154	0.156	0.157
5.25	0.102	0.108	0.118	0.126	0.131	0.136	0.139	0.154	0.156	0.157
5.50	0.099	0.106	0.116	0.124	0.130	0.134	0.138	0.154	0.156	0.157
5.75	0.097	0.103	0.113	0.122	0.128	0.133	0.136	0.154	0.157	0.157
6.00	0.094	0.101	0.111	0.120	0.126	0.131	0.135	0.153	0.157	0.157
6.25	0.092	0.098	0.109	0.118	0.124	0.129	0.134	0.153	0.157	0.158
6.50	0.090	0.096	0.107	0.116	0.122	0.128	0.132	0.153	0.157	0.158
6.75	0.087	0.094	0.105	0.114	0.121	0.126	0.131	0.153	0.157	0.158
7.00	0.085	0.092	0.103	0.112	0.119	0.125	0.129	0.152	0.157	0.158
7.25	0.083	0.090	0.101	0.110	0.117	0.123	0.128	0.152	0.157	0.158
7.50	0.081	0.088	0.099	0.108	0.115	0.121	0.126	0.152	0.156	0.158
7.75	0.079	0.086	0.097	0.106	0.114	0.120	0.125	0.151	0.156	0.158
8.00	0.077	0.084	0.095	0.104	0.112	0.118	0.124	0.151	0.156	0.158
8.25	0.076	0.082	0.093	0.102	0.110	0.117	0.122	0.150	0.156	0.158
8.50	0.074	0.080	0.091	0.101	0.108	0.115	0.121	0.150	0.156	0.158
8.75	0.072	0.078	0.089	0.099	0.107	0.114	0.119	0.150	0.156	0.158
9.00	0.071	0.077	0.088	0.097	0.105	0.112	0.118	0.149	0.156	0.158
9.25	0.069	0.075	0.086	0.096	0.104	0.110	0.116	0.149	0.156	0.158
9.50	0.068	0.074	0.085	0.094	0.102	0.109	0.115	0.148	0.156	0.158
9.75	0.066	0.072	0.083	0.092	0.100	0.107	0.113	0.148	0.156	0.158
10.00	0.065	0.071	0.082	0.091	0.099	0.106	0.112	0.147	0.156	0.158
20.00	0.035	0.039	0.046	0.053	0.059	0.065	0.071	0.124	0.148	0.156
50.00	0.014	0.016	0.019	0.022	0.025	0.028	0.031	0.071	0.113	0.142
100.00	0.007	0.008	0.010	0.011	0.013	0.014	0.016	0.039	0.071	0.113

4.3.2 Footings in Saturated Clays

Janbu et al. (1956) presented an equation to estimate the average settlement of flexible footings on saturated clay soils (Poisson's ratio is about 0.5). For the notation shown on Figure 4.8, the elastic settlement may be estimated as follows:

$$S_e = A_1 A_2 \frac{q_0 B}{E_s}$$

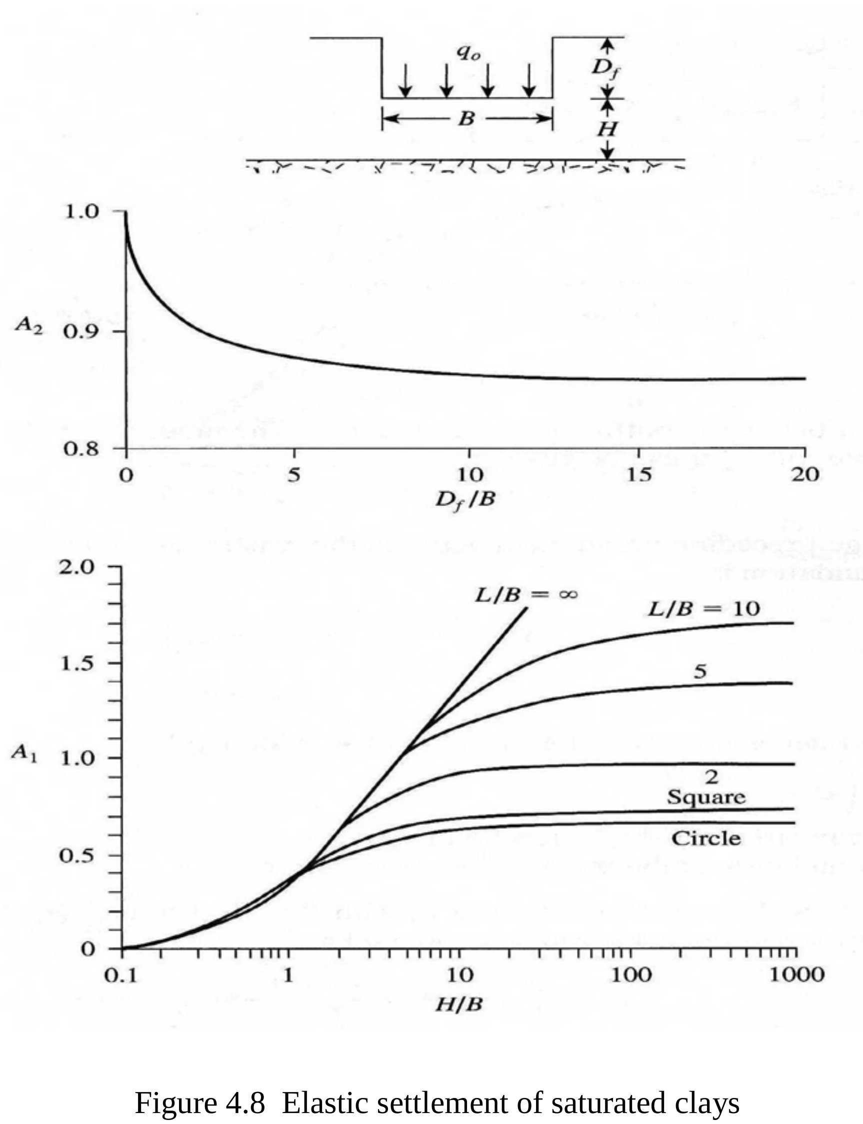


Figure 4.8 Elastic settlement of saturated clays

4.3.3 Settlement of Sandy Soils Using the Strain Influence Factor (Schmertmann 1978)

The settlement of sandy soils may be estimated using the semiempirical strain influence factor method introduced by Schmertmann et al. (1978). According to this method, the settlement may be calculated as follows:

$$S_e = C_1 C_2 q_0 \sum_0^{z_2} \frac{I_z}{E_s} \Delta z$$

where;

q_0 is the net contact pressure at foundation level

E_s is the modulus of elasticity of the soil

I_z is the strain influence factor as defined on Figure 4.9 for square and strip footing

C_1 is a correction factor for the foundation depth and may be calculated as follows:

$$C_1 = 1 - 0.5 \frac{q}{q_0}$$

C_2 is a correction factor for creep in soil and may be calculated as follows:

$$C_2 = 1 + 0.2 \log \frac{t_{\text{time in years}}}{0.1}$$

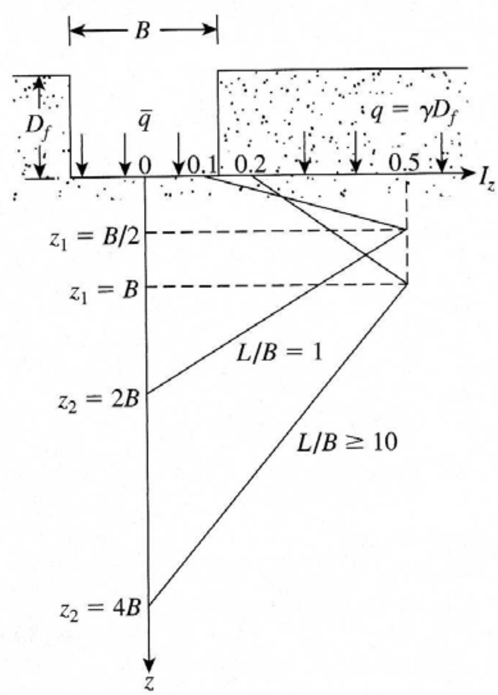
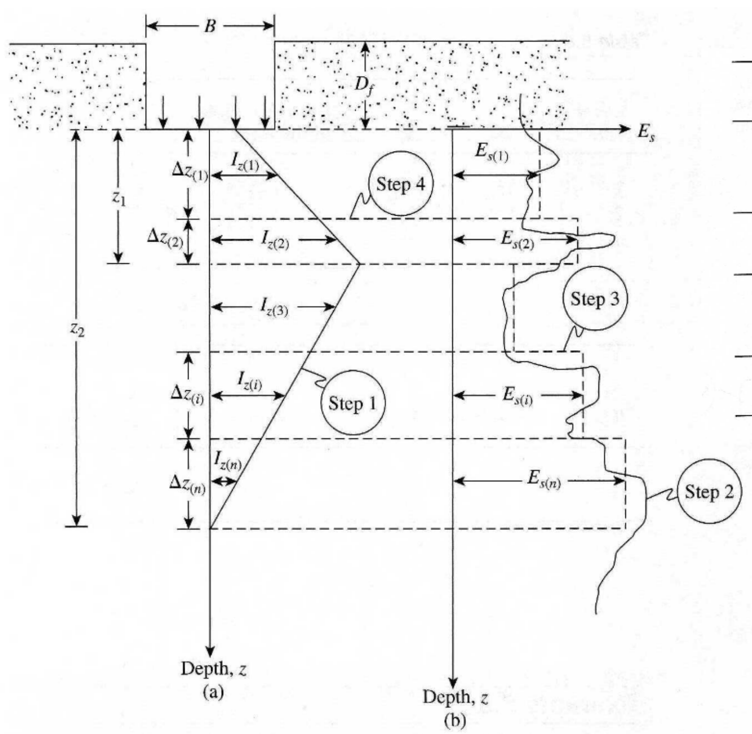


Figure 4.9 The strain influence factor

In order to estimate the settlement using the strain influence method the following steps may be followed:

- Plot the foundation and the variation of the I_z with depth
- Plot the actual distribution of E_s with depth next to the I_z plot and approximate it into a number of layers each with an average E_s (Figure 4.10)
- Divide the soil within the influence depth into sublayer depending on the break in continuity in the I_z and E_s diagrams
- Prepare a table such as shown on Figure 4.10 to obtain $\sum_{0}^{z_2} \frac{I_z}{E_s} \Delta z$
- Calculate C_1, C_2
- Calculate S_e



Layer No.	Δz	E_s	I_z at the middle of the layer	$\frac{I_z}{E_s}\Delta z$
1	$\Delta z_{(1)}$	$E_{s(1)}$	$I_{z(1)}$	$\frac{I_{z(1)}}{E_{s(1)}}\Delta z_1$
2	$\Delta z_{(2)}$	$E_{s(2)}$	$I_{z(2)}$	
⋮	⋮	⋮	⋮	
i	$\Delta z_{(i)}$	$E_{s(i)}$	$I_{z(i)}$	$\frac{I_{z(i)}}{E_{s(i)}}\Delta z_i$
⋮	⋮	⋮	⋮	⋮
n	$\Delta z_{(n)}$	$E_{s(n)}$	$I_{z(n)}$	$\frac{I_{z(n)}}{E_{s(n)}}\Delta z_n$
				$\Sigma\frac{I_z}{E_s}\Delta z$

Figure 4.10 The procedure to calculate elastic settlement using strain influence factor method

4.4 Elastic Properties of Soils

The elastic properties of soils (E_s and μ_s) may be estimated from the results of laboratory tests such as direct shear test and triaxial compression test, however, they may also be estimated from some empirical correlation to both the SPT N-value or the CPT q_c-value. Furhermore, typical values are also available for different types of soils. The geotechnical engineer should evaluate all the available field and laboratory information, in addition to typical values and previous experience, to estimate the value of the modulus of elasticity to be used in estimating settlement or soil modeling since the analysis is sensitive to this value.

For sandy soils, the following correlations may be used:

$$\frac{E_s}{P_a} = \alpha N_{60}$$

$$\alpha = 5 \text{ ...... for sands with fines}$$

$$\alpha = 10 \text{ ...... for clean NC sands}$$

$$\alpha = 15 \text{ ...... for clean OC sands}$$

$$P_a = \text{atmospheric pressure} = 100 \text{ KPa} = 2000 \text{ psf}$$

$$E_s = 2.5 q_c \text{ ...... for square and circular footings}$$

$$E_s = 3.5 q_c \text{ ...... for strip footings}$$

For clayey soils, the following correlation may be used:

$$E_s = \beta C_u$$

The value of β may be obtained from Table 4.2. Also, Table 4.3 provides typical values for the modulus of elasticity and Poisson’s ratio for different soil types.

Table 4.2 Values of factor β to estimate modulus of elasticity of clays

Plasticity index	β				
	OCR = 1	OCR = 2	OCR = 3	OCR = 4	OCR = 5
< 30	1500–600	1380–500	1200–580	950–380	730–300
30 to 50	600–300	550–270	580–220	380–180	300–150
> 50	300–150	270–120	220–100	180–90	150–75

[a]Interpolated from Duncan and Buchignani (1976)

Table 4.3 Typical values for the modulus of elasticity and Poisson's ratio for different soil types

Type of soil	Modulus of elasticity, E_s		Poisson's ratio, μ_s
	MN/m²	lb/in²	
Loose sand	10.5–24.0	1500–3500	0.20–0.40
Medium dense sand	17.25–27.60	2500–4000	0.25–0.40
Dense sand	34.50–55.20	5000–8000	0.30–0.45
Silty sand	10.35–17.25	1500–2500	0.20–0.40
Sand and gravel	69.00–172.50	10,000–25,000	0.15–0.35
Soft clay	4.1–20.7	600–3000	
Medium clay	20.7–41.4	3000–6000	0.20–0.50
Stiff clay	41.4–96.6	6000–14,000	

4.5 Elastic Settlement Using SPT N-Value

Since in most projects, the available data for the geotechnical engineer may only be the SPT N-Values and/or the CPT q_c-values, it will be helpful to have a direct correlation between N and/or q_c and the allowable bearing pressure and/or elastic settlement. Meyerhof (1956) introduced the following equations to estimate the bearing capacity from the SPT N-value for a limiting settlement of 1 inch:

$$q_{net} = \frac{N_{60}}{4} \quad \text{... in ksf ... for } B \le 4 \text{ feet}$$

$$q_{net} = \frac{N_{60}}{6}\left(\frac{B+1}{B}\right)^2 \quad \text{... in ksf ... for } B > 4 \text{ feet}$$

$$\text{where } q_{net} = q_0 = \frac{P_{col}}{A_f} \quad \text{or} \quad \frac{P_{total}}{A_f} - q$$

P_{col} is the column load

P_{total} is the column load in addition to the load of the footing and the fill on top of it

A_f if the footing area (BxL)

q is the overburden pressure at the foundation level

Bowles (1977) found that Meyerhof's equations were very conservative. Bowles modified the equation and added "Se" to enable the engineer to use the equation for different allowable elastic settlement values, as follows:

$$q_{net} = \frac{N_{60}}{2.5} F_d S_e \quad \text{... in ksf ... for } B \le 4 \text{ feet}$$

$$q_{net} = \frac{N_{60}}{4}\left(\frac{B+1}{B}\right)^2 F_d S_e \quad \text{... in ksf ... for } B > 4 \text{ feet}$$

$$F_d = 1 + 0.33\frac{D_f}{B} \quad \text{..... depth factor}$$

Re-writing the equation, the elastic settlement may be estimated as follows:

$$S_e(\text{inch}) = \frac{2.5 q_{net}(\text{ksf})}{N_{60} F_d} \quad \text{........... for } B \le 4 \text{ feet}$$

$$S_e(\text{inch}) = \frac{4 q_{net}(\text{ksf})}{N_{60} F_d}\left(\frac{B}{B+1}\right)^2 \quad \text{....for } B > 4 \text{ feet}$$

4.6 Consolidation Settlement

4.6.1 End of Primary Consolidation Settlement

The concept was previously explained in the consolidation section of the soil review chapter, however, briefly, Figure 4.11 explains the concept of the average effective stress increase within the clayey layer under consideration. The stress increase may be calculated using any method at the top, bottom, and center of the layer and then the average stress increase may be calculated as:

$$\Delta\sigma'_{\text{average}} = \frac{1}{6}\left(\Delta\sigma'_{\text{top}} + 4\Delta\sigma'_{\text{center}} + \Delta\sigma'_{\text{bottom}}\right)$$

The average stress increase may also be estimated using the method explained in section 4.2.3 for estimating the average stress increase within a deep layer.

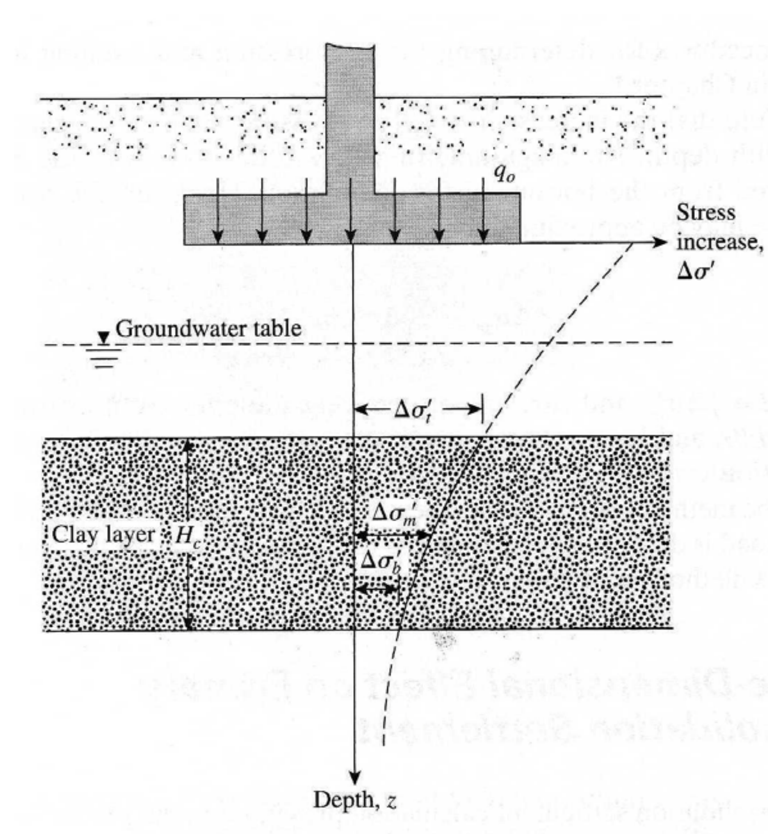


Figure 4.11 Consolidation settlement – average stress increase

The following equations may be used to estimate the end of primary consolidation settlement depending on the condition of the clay being normal or pre-consolidated and also depending on the relative values of the initial overburden pressure, stress increase, final pressure, and the pre-consolidation pressure.

If the clay layer is thick, it should be divided into a few sublayers in order to increase the calculation accuracy. However, it should be noted that, the engineer may need to use different equations for different sublayers.

For NC clay :

$$S_c = \frac{C_c H_c}{1+e_o} \log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_o}\right)$$

For OC clay with $(\sigma'_o + \Delta\sigma') < \sigma'_c$:

$$S_c = \frac{C_s H_c}{1+e_o} \log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_o}\right)$$

For OC clay with $(\sigma'_o + \Delta\sigma') > \sigma'_c$:

$$S_c = \frac{C_s H_c}{1+e_o} \log\left(\frac{\sigma'_c}{\sigma'_o}\right) + \frac{C_c H_c}{1+e_o} \log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_c}\right)$$

In case consolidation test results are not available, quick estimates for the compression index (C_c) and the swelling index (C_s) may be obtained using natural moisture content (w), plasticity index (I_p), Unit weight, void ratio, specific gravity, etc. as shown on Table 4.4.

Table 4.4 Correlations for compression and swelling indices

Compression index, C_c	Comments	Source/Reference
$C_c = 0.009(w_L - 10)$ ($\pm 30\%$ error)	Clays of moderate S_t	Terzaghi and Peck (1967)
$C_c = 0.37(e_o + 0.003w_L + 0.0004w_N - 0.34)$	678 data points	Azzouz et al. (1976)
$C_c = 0.141G_s\left(\frac{\gamma_{sat}}{\gamma_{dry}}\right)^{2.4}$	All clays	Rendon-Herrero (1983)
$C_c = 0.0093w_N$	109 data points	Koppula (1981)
$C_c = -0.0997 + 0.009w_L + 0.0014I_P + 0.0036w_N + 0.1165e_o + 0.0025C_P$	109 data points	Koppula (1981)
$C_c = 0.329[w_N G_s - 0.027w_P + 0.0133I_P(1.192 + C_P/I_P)$	All inorganic clays	Carrier (1985)
$C_c = 0.046 + 0.0104I_P$	Best for $I_P < 50\%$	Nakase et al. (1988)
$C_c = 0.00234w_L G_s$	All inorganic clays	Nagaraj and Srinivasa Murthy (1985, 1986)
$C_c = 1.15(e_o - 0.35)$	All clays	Nishida (1956)
$C_c = 0.009w_N + 0.005w_L$	All clays	Koppula (1986)
$C_c = -0.156 + 0.411e_o + 0.00058w_L$	72 data points	Al-Khafaji and Andersland (1992)
Recompression index, C_r		
$C_r = 0.000463w_L G_s$		Nagaraj and Srinivasa Murthy (1985)
$C_r = 0.00194(I_P - 4.6)$	Best for $I_P < 50\%$	Nakase et al. (1988)
$= 0.05$ to $0.1C_c$	In desperation	

4.6.2 Secondary Consolidation Settlement

After the end of primary consolidation settlement, which basically means the dissipation of all the excess pore water pressure, some additional settlement is observed due to plastic adjustment of the soil fabric. This settlement is called the secondary consolidation settlement and it is significant in organic soils and very soft cohesive soils. Figure 4.12 shows the end of primary consolidation, which is identified by the void ratio "e_p". Settlements that occurs after "e_p" is secondary settlement.

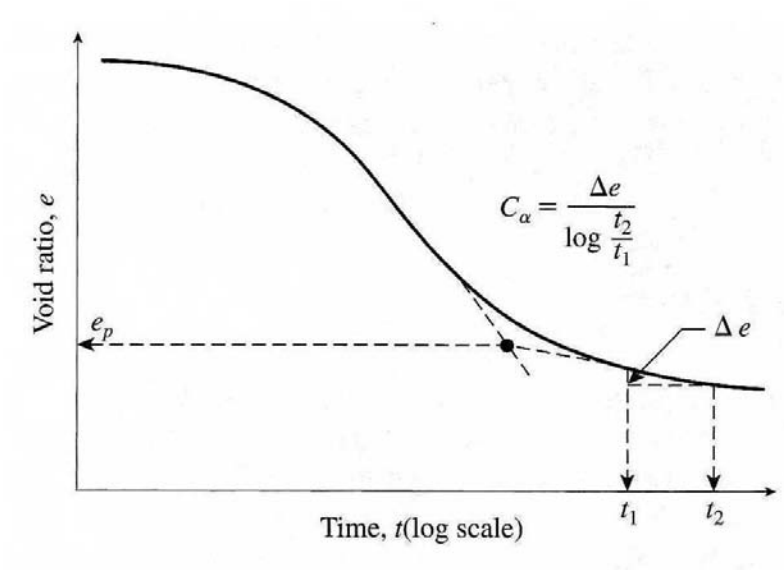


Figure 4.12 Secondary consolidation settlement

The slope of the curve between two time intervals is the secondary compression index:

$$C_\alpha = \frac{\Delta e}{\log\left(\frac{t_2}{t_1}\right)}$$

The secondary compression index may be used to calculate the secondary consolidation settlement as follows:

$$S_{e(s)} = \frac{C_\alpha}{1 + e_{eop}} H \log\left(\frac{t_2}{t_1}\right)$$

where; e_{eop} is the void ratio at the end of primary consolidation, H is the thickness of the compressible layer, t_1 and t_2 are the time limits for estimating the secondary consolidation settlement.

The following correlations along with Table 4.5 may be used to estimate the value of the secondary compression index using the natural moisture content (w) or the compression index (C_c):

$$C_\alpha = 0.0001\,w$$

$$\frac{C_\alpha}{C_c} \approx 0.04 \pm 0.01 \text{ ..... for inorganic clays and silts}$$

$$\frac{C_\alpha}{C_c} \approx 0.05 \pm 0.01 \text{ ..... for organic clays and silts}$$

$$\frac{C_\alpha}{C_c} \approx 0.075 \pm 0.01 \text{ ..... for peat}$$

Table 4.5 Correlation for secondary compression index

Secondary compression index, C_α		
$C_\alpha = 0.00168 + 0.00033 I_P$		Nakase et al. (1988)
$= 0.0001 w_N$		NAFAC DM7.1 p. 7.1–237
$C_\alpha = 0.032 C_c$	$0.025 < C_\alpha < 0.1$	Mesri and Godlewski (1977)
$= 0.06$ to $0.07 C_c$	Peats and organic soil	Mesri (1986)
$= 0.015$ to $0.03 C_c$	Sandy clays	Mesri et al. (1990)

Geotechnical Engineering Overview Examination

After you have completed answering all of the questions, go back and check your work. Make certain that you have marked only one answer for each question. There is only one correct answer to each question. Make certain that you have answered each question. Any question that is left blank will be counted as incorrect.

A score of 70% is required to complete the course. Failing to achieve a 70% score all your answers will be erased. You will have three opportunities to achieve a passing grade. Failing to score a passing grade on the third attempt will block you from further attempts and your course fee returned to you.

Once you have successfully completed exam you will be able to print out your completion certificate. We suggest you file it electronically or print it out should you be audited by your licensure board for compliance with continuing education requirements. At that time you will also be able to compare your answers to the school answers on questions you may have missed.

1- For a soil, given: void ratio = 0.81, moisture content = 21%, and specific gravity = 2.68. The degree of saturation is:

a- 25.6%

b- 92.6%

c- 69.5%

d- 34.7%

2- A soil with a specific gravity of 1.6 is mostly:

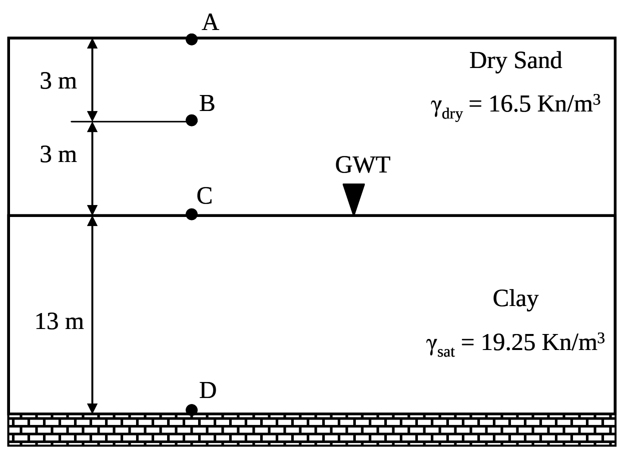
a- Clay

b- Sand

c- Silt

d- Peat

3- For the shown soil profile, the effective vertical stress at Point "D" is:



a- 222 kN/m^2

b- 120 kN/m^2

c- 340 kN/m^2

d- 92 kN/m^2

4- For a normally consolidated clay, the following is given:

Pressure (kN/m^2)	Void ratio
120	0.82
360	0.64

The compression index is:

a- 0.255

b- 0.377

c- 0.754

d- 0.124

5- A direct shear test was conducted on dry sand. The results were as follows:

Normal Force (lb)	Shear Force at Failure (lb)
50	43.5
110	95.5
150	132

The area of the specimen is 2 in. x 2 in. The angle of shearing resistance is approximately:

a- 45 degrees

b- 35 degrees

c- 31 degrees

d- 41 degrees

6- Which of the following can be considered an advantage of the triaxial compression test when compared to direct shear test?

a- No pre-determined plan of failure
b- Provide drainage control during consolidation stage
c- Provide drainage control during shear stage
d- All the above

7- During an unconfined compression test on a 3-inch diameter and 6-inch long cylinder of stiff clay, the failure load was 400 lb. The undrained cohesion of the clay is:

a- 4.1 tsf
b- 3.2 tsf
c- 5.6 tsf
d- 1.2 tsf

8- The measured in-field N-values should be corrected for:

a- Rod length
b- Borehole diameter
c- Hammer type
d- All the above

9- During a CPT test, a soil that has a cone point resistance of 1 MN/m^2 and a friction ratio of 2% should be classified, according to the CPT classification chart, as:

a- Sands
b- Silty sands
c- Sandy silts and silts
d- Clayey silts and silty clays

10- According to Terzaghi, the foundation can be considered shallow if:

a- $D_f < 5$ feet

b- $D_f < B$

c- $D_f < 2B$

d- $D_f > 2B$

11- For a continuous footing, the following are given:

Width of the footing = 3 feet

Foundation depth = 3

Angle of shearing resistance = 28 degrees

Cohesion = 400 psf

Unit weight = 110 pcf

Groundwater table is 10 feet below grade

Using Terzaghi’s equation and assuming general shear failure with safety factor of 4, the allowable bearing capacity of the footing is:

a- 3120 psf

b- 5195 psf

c- 10290 psf

d- 1250 psf

12- The effect of the GWT being at the foundation level is to:

a- Increase the overburden pressure at the foundation level

b- Reduce the overburden pressure at the foundation level

c- Reduce soil unit weigh within stresses zones to the submerged unit weight

d- All the above

13- Secondary consolidation settlement comprises most of the settlement in:

a- Sands

b- Silty sands

c- Stiff Clays

d- Organic soils

14- A square footing (5 ft x 5 ft) carries a load of 50 tons and located at a depth of 4.5 feet below ground surface. A clay layer 10 feet thick is located 3 feet below the foundation level. Using 2:1 line method, the stress at the middle of the clay layer is approximately:

a- 592 psf

b- 1562 psf

c- 309 psf

d- 120 psf

15- A foundation measuring 1.5m x 1.5m is supported by a saturated clay layer. Given:

Depth of footing = 1.2 m

Thickness of the clay layer below foundation level = 3 m

Modulus of elasticity of the clay = 600 kN/m^2

Stress increase at foundation level = 150 kN/m^2

The elastic settlement of the foundation is:

a- 24 mm

b- 124 mm

c- 246 mm

d- 760 mm

16- To estimate expected settlement after 10 years using Schmertmann's method, the engineer should consider a creep correction factor of:

a- 1.2

b- 1.4

c- 0.95

d- 2.4

17- For a foundation soils that is mostly sand with fines, the average recorded N_{60}-value was 10. Approximately, the modulus of elasticity is:

a- 1000 ksf

b- 100 ksf

c- 10 ksf

d- 1 ksf

18- For a square shallow foundation supported on sandy soil:

Depth of footing = 4 feet

Width of footing = 6 feet

Tolerable elastic settlement = 1.5 inches

Average N_{60}-value = 12

The net allowable bearing pressure according to Bowles (1977) is:

a- 12400 psf

b- 2450 psf

c- 750 psf

d- 7500 psf

19- The natural moisture content and the liquid limit of a clay sample was estimated in the laboratory to be 35% and 65%, respectively, the compression index and recompression/swelling indices of this clay are approximately:

a- 0.5 and 0.035

b- 0.9 and 0.08

c- 0.1 and 0.07

d- 1.2 and 0.75

20- The liquid limit and the plastic limit of a clay sample are 75% and 20%, respectively; the secondary compression index is approximately:

a- 0.0002

b- 0.002

c- 0.02

d- 0.2