1. Introduction

1.1 Background:

Structural fills and embankments are typically constructed by compacting earthen materials in place, so the compaction properties of the material (optimum water content and maximum dry density) are very important to performance. The compressibility and shear strength are also important measures of the compacted material. In addition, drainage is an important consideration to prevent the loss of shear strength due to saturation.

Structural fill is a screened earthen material used to create a strong and stable base.For example, the native soil at a site may be too weak to support a structure, so the soil is replaced by compacted structural fill to provide the needed structural support. In roadway applications, structural fills are often used as fill for abutments or slabs, backfill for retaining structures, or filling of trenches and other excavations that will support roadways or other structures when completed.

8’The bearing capacity of soil is perhaps the most important of all the topics in soil engineering. Soil behaves in a complex manner when loaded so, it is important to know the bearing capacity of soil. Soil when stressed due to loading, tend to deform. The resistance to deformation of the soil depends upon factors like water content, angle of internal friction, and the manner in which load is applied on the soil. The maximum load per unit area which the soil or rock can carry without yielding or displacement is termed as **the bearing capacity of soil.**

1.2 Research objectives:

The objectives of this research are to:

- Make classification for each type of soil used for the experiments and find their parameter.
- Find the bearing capacity of some layers of soil using plate load test.
- Find field density for layers of soil.
- Estimate values of bearing capacity of the layers of soil using equations.
- Comparing the estimated values of bearing capacity of each layers of soil with those deduced by plate load test.
- Notice the influence of compaction on bearing capacity of layers of soil.

1.3 Methodology:

- Four samples of soil collected from different places, and an experimental work was carried out to it to deduce their parameters.
- **The Unified Soil Classification System (USCS)** was adopted for soil's classification.
- One of samples was chosen for plate load test.
- For experimental work in field a steel box was used, and sample was placed in layers.
- Plate load test was applied for each layer to get actual value of bearing capacity.
- Field density test was carried out to all layers to get densities of soil.
- **Terzaghi's formulae** of ultimate bearing capacity were used to estimate values of bearing capacity to any layer.
- The bearing capacity values according to plate load test were compared by the estimated values from **Terzaghi's formulae**.

1.4 Research layout:

- **Chapter one** is an introductory chapter for the thesis and includes the objectives of the thesis, the methodology adopted, and layout of the thesis.
- **Chapter two is the literature review** about soil in respect to its formation, types, particle size, shape, mass structure, index properties, consistency, and classification.
- **Chapter three is about bearing capacity,** which contains types, reference values due to type of soil, and equations of calculations.
- **Chapter four is the experimental works in laboratory** for structural fills to check materials. The laboratory tests are Atterburg's limits, sieve analysis, compaction test, soil proficiency testing California bearing ratio (CBR), and shear box test.
- **Chapter five is about plate load test (experimental works in field),** which it is a field test for determining the ultimate bearing capacity of soil and the likely settlement under a given load**.**
- **Chapter six is a discussion and comments of the results.**
- **Chapter seven is a conclusion and recommendations of the research.**

Chapter (2)

LITERATURE REVIEW

2.1 Introduction

This chapter contains literature reviews of the soil in respect to it is formation, types, particle size and shape, mass structure, index properties, consistency, and classification.

2.2 Soil in general

The word 'soil' has different meanings for different professions. To the agriculturist, soil is the top thin layer of earth within which organic forces are predominant and which is responsible for the support of plant life. To the geologist, soil is the material in the top thin zone within which roots occur. From the point of view of an engineer, soil includes all earth materials, organic and inorganic, occurring in the zone overlying the rock crust.[1]

Soils are aggregates of mineral particles, and together with air and/or waterin the void spaces, they form three-phase systems. A large portion of theearth's surface is covered by soils, and they are widely used as constructionand foundation materials. Soil mechanics is the branch of engineering thatdeals with the engineering properties of soils and their behavior understress.[8]

The behavior of a structure depends upon the properties of the soil materials on which the structure rests. The properties of the soil materials depend upon the properties of the rocks from which they are derived.[1]

2.3 Formation of soils

Soil is defined as a natural aggregate of mineral grains, with or without organic constituents that can be separated by gentle mechanical means such as agitation in water. By contrast rock is considered to be a natural aggregate of mineral grains connected by strong and permanent cohesive forces. The process of weathering of the rock decreases the cohesive forces binding the mineral grains and leads to the disintegration of bigger masses to smaller and smaller particles. Soils are formed by the process of weathering of the parent rock. The weathering of the rocks might be by mechanical disintegration, and/or chemical decomposition.[1]

2.3.1Mechanical Weathering

Mechanical weathering of rocks to smaller particles is due to the action of such agents as the expansive forces of freezing water in fissures, due to sudden changes of temperature or due to the abrasion of rock by moving water or glaciers. This type of rock weathering takes place in a very significant manner in arid climates where free, extreme atmospheric radiation brings about considerable variation in temperature at sunrise and sunset.[1]

2.3.2 Chemical weathering

Chemical weathering (decomposition) can transform hard rock minerals into soft, easily erodible matter. The principal types of decomposition are *hydmtion, oxidation, carbonation, desilication* and *leaching.* Oxygen and carbon dioxide which are always present in the air readily combine with the elements of rock in the presence of water.[1]

2.4 General types of soils

The individual size of the constituent parts of even the weathered rock might range from the smallest state (colloidal) tothe largest possible (boulders). This implies that all the weathered constituents of a parent rock cannot be termed soil. According to their grain size, soil particles are classified as cobbles, gravel, sand, silt and clay. Grains having diameters in the range of 4.75 to 76.2 mm are called gravel. If the grains are visible to the naked eye, but are less than about 4.75 mm in size the soil is described as sand. The lower limit of visibility of grains for the naked eyes is about 0.075 mm. Soil grains ranging from 0.075 to 0.002 mm are termed as silt and those that are finer than 0.002 mm as clay. This classification is purely based on size which does not indicate the properties of fine grained materials.

2.4.1 Residual and transported soils On the basis of origin of their constituents, soils can be divided into two large groups:

1. Residual soils.
2. Transported soils.

Residual soils are those that remain at the place of their formation as a result of the weathering of parent rocks. The depth of residual soils depends primarily on climatic conditions and the time of exposure. In some areas, this depth might be considerable. In temperate zones residual soils are commonly stiff and stable. An important characteristic of residual soil is that the sizes of grains are indefinite. For example, when a residual sample is sieved, the amount passing any given sieve size depends greatly on the time and energy expended in shaking, because of the partially disintegrated condition.

Transported soils are soils that are found at locations far removed from their place of formation. The transporting agencies of such soils are glaciers, wind and water. The soils are named according to the mode of transportation. *Alluvial* soils are those that have been transported by running water.

The soils that have been deposited in quiet lakes are *lacustrine* soils. *Marine soils* are those deposited in sea water. The soils transported and deposited by wind are *aeolian*soils. Those deposited primarily through the action of gravitational force, as in landslides, are *colluvial* soils. *Glacial* soils are those deposited by glaciers. Many of these transported soils are loose and soft to a depth of several hundred feet. Therefore, difficulties with foundations and other types of construction are generally associated with transported soils. [1]

2.4.2 Organic and Inorganic Soils

Soils in general are further classified as *organic* or *inorganic*. Soils of organic origin are chiefly formed either by growth and subsequent decay of plants such as peat, or by the accumulation of fragments of the inorganic skeletons or shells of organisms. Hence a soil of organic origin can be either organic or inorganic. The term organic soil ordinarily refers to a transported soil consisting of the products of rock weathering with a more or less conspicuous admixture of decayed vegetable matter.[1]

2.4.3 Soils generally used in Practice

Bentonite is clay formed by the decomposition of volcanic ash with a high content of montmorillonite. It exhibits the properties of clay to an extreme degree.

Varved Clays consist of thin alternating layers of silt and fat clays of glacial origin. They possess the undesirable properties of both silt and clay. The constituents of varved clays were transported into fresh water lakes by the melted ice at the close of the ice age.

Kaolin, China Clay is very pure forms of white clay used in the ceramic industry.

Boulder Clay is a mixture of an unstratifiedsedimented deposit of glacial clay, containing unsorted rock fragments of all sizes ranging from boulders, cobbles, and gravel to finely pulverizedClay material.

Calcareous Soil is a soil containing calcium carbonate. Such soil effervesces when tested with weak hydrochloric acid.

Marl consists of a mixture of calcareous sands, clays, or loam.

Hardpan is a relatively hard, densely cemented soil layer, like rock which does not soften when wet. Boulder clays or glacial till is also sometimes named as hardpan.

Caliche is an admixture of clay, sand, and gravel cemented by calcium carbonate deposited from ground water.

Peat is a fibrous aggregate of finer fragments ofdecayedvegetable matter. Peat is very compressible and one should be cautious when using it for supporting foundations of structures.

Loam is a mixture of sand, silt and clay.

Loess is a fine-grained, air-borne deposit characterized by a very uniform grain size, and high void ratio. The size of particles ranges between about 0.01 to 0.05 mm. The soil can stand deep vertical cuts because of slight cementation between particles. It is formed in dry continental regions and its color is yellowish light brown.

Shale is a material in the state of transition from clay to slate. Shale itself is sometimes considered a rock but, when it is exposed to the air or has a chance to take in water it may rapidly decompose. [1]

2.5 Soil particle size and shape

The size of particles as explained earlier may range from gravel to the finest size possible. Their characteristics vary with naked eye or may be examined by means of a hand lens. They constitute the coarser fractions of the soils. Grains finer than 0.075 mm constitute the finer fractions of soils. It is possible to distinguish the grains lying between 0.075 mm and 2 μ (1 μ= 1 micron = 0.001 mm) under a microscope. Grains having a size between 2 μ and 0.1 μ can be observed under a microscope but their shapes cannot be made out. The shape of grains smaller than 1 μ can be determined by means of an electron microscope. The molecular structure of particles can be investigated by means of X-ray analysis.

The coarser fractions of soils consist of gravel and sand. The individual particles of gravel, which are nothing but fragments of rock, are composed of one or more minerals, whereas sand grains contain mostly one mineral which is quartz. The individual grains of gravel and sand may be angular, subangular, sub-rounded, rounded or well-rounded as shown in Fig. 2.1. Gravel may contain grains which may be flat. Some sands contain a fairlyhigh percentage of mica flakes that give them the property of elasticity. Silt and clay constitute the finerfractions of the soil. Any one grain of this fraction generally consists of only one mineral.

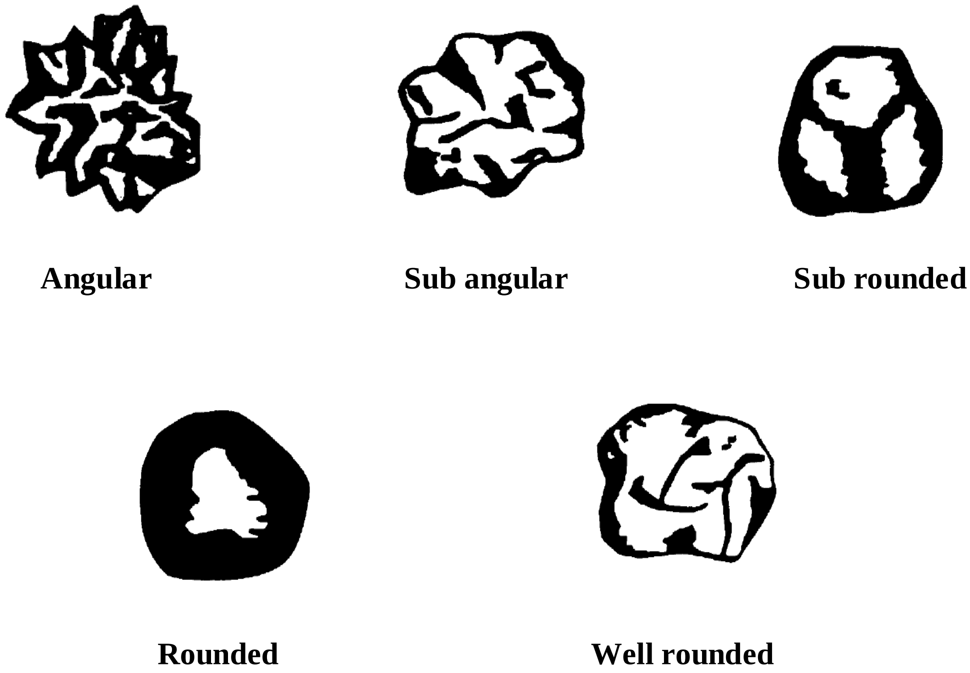


Figure (2.1)   shapes of coarser fractions of soils

The particles may be angular, flake-shaped or sometimes needle like. Table 2.1 gives the particle size classification systems as adopted bysome of the organizations in the USA. The Unified Soil Classification System is now almost universally accepted and has been adopted by the American Society for Testing and Materials (ASTM).[1]

2.5.1 Specific surface

In soils, the dispersed or the solid phase predominates and the dispersion medium, soil water, only helps to fill the pores between the solid particles. The significance of the concept of dispersion becomes more apparent when the relationship of surface to particle size is considered. In the case of silt, sand and larger size particles the ratio of the area of surface of the particles to the volume of the sample is relatively small. This ratio becomes increasingly large as size decreases from 2μ which is the upper limit for clay-sized particles.

A useful index of relative importance of surface effects is the *specific surface* of grain. The specific surface is defined as the total area of the surface of the grains expressed in square centimeters per gram or per cubic centimeter of the dispersed phase.

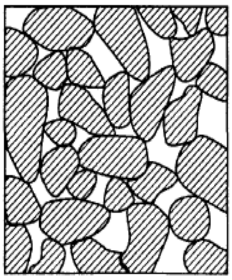
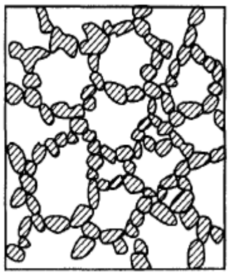
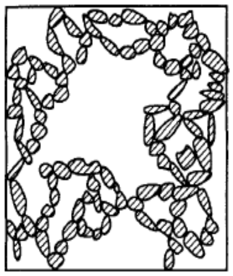
The shape of the clay particles is an important property from a physical point of view. The amount of surface per unit mass or volume varies with the shape of the particles. Moreover, the amount of contact area per unit surface changes with shape. It is a fact that a sphere has the smallest surface area per unit volume whereas a plate exhibits the maximum. The interparticle forces between the surfaces of particles have a significant effect on the properties of the soil mass if the particles in the media belong to the clay fraction. The surface activity depends not only on the specific surface but also on the chemical and mineralogical composition of the solid particles.[1]

Table (2.1) Particle size classification by various systems

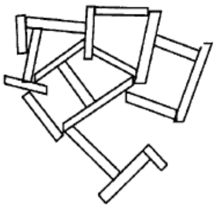
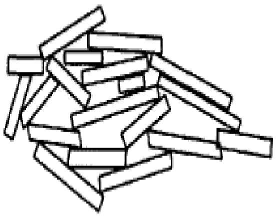
Name of organization	Particle size (mm)			
	Gravel	**Sand**	**Silt**	**Clay**
Massachusetts Institute of Technology(MIT)	>2	2 to 0.06	0.06 to 0.002	<0.002
US Department of agriculture(USDA)	>2	2 to 0.05	0.05 to 0.002	<0.002
American Association of State highway and Transportation Officials(AASHTO)	76.2 to 2	2 to 0.075	0.075to 0.002	<0.002
Unified soil classification system , US Bureau of Reclamation, US Army corps of Engineers and American Society for Testing and materials	76.2 to 4.75	4.75to0.075	Fines(silt and clay)<0.075	

2.6 Soil mass structure

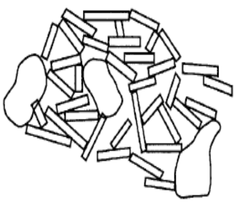
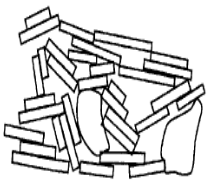
The orientation of particles in a mass depends on the size and shape of the grains as well as upon the minerals of which the grains are formed. The structure of soils that is formed by natural deposition can be altered by external forces. Figure 2.2 gives the various types of structures of soil. Fig. 2.2(a) is a *single grained structure* which is formed by the settlement of coarse grained soils in suspension in water. Fig. 2.2(b) is *a flocculent structure* formed by the deposition of the fine soil fraction in water.

(a)Single grainstructure (b)flocculent structure (c)honeycomb structure

(d) Flocculated type structure (Edge to face contact) (g) disturbed structure (Face to face contact)

(f) Undisturbed salt water deposit (g) Undisturbed fresh water deposit

Figure (2.2) schematic diagrams of various types of structures

Fig. 2.2(c) is a *honeycomb structure* which is formed by the disintegration of a flocculent structure under a superimposed load. The particles oriented in a flocculent structure will have edge-to-face contact as shown in Fig. 2.2(d) whereas in a honeycomb structure, the particles will have face to-face contact as shown in Fig. 2.2(e). Natural clay sediments will have more or less flocculated particle orientations.

Marine clays generally have a more open structure than fresh water clays. Figs. 2.2(f) and (g) show the schematic views of salt water and fresh water deposits. [1]

2.7 Index properties of soils

The various properties of soils which would be considered as index properties are:

1. The size and shape of particles.
2. The relative density or consistency of soil.

The index properties of soils can be studied in a general way under two classes. They are:

1. Soil grain properties.
2. Soil aggregate properties.

The principal soil grain properties are the size and shape of grains and the mineralogical character of the finer fractions (applied to clay soils). The most significant aggregate property of cohesion less soils is the relative density, whereas that of cohesive soils is the consistency. Water content can also be studied as an aggregate property as applied to cohesive soils.

The strength and compressibility characteristics of cohesive soils are functions of water content. As such water content is an important factor in understanding the aggregate behavior of cohesive soils.

By contrast, water content does not alter the properties of a cohesionless soil significantly except when the mass is submerged, in which case only its unit weight is reduced. [1]

2.7.1 The shape and size of particles

The shapes of particles as conceived by visual inspection give only a qualitative idea of the behavior of a soil mass composed of such particles. Since particles finer than 0.075 mm diameter cannot be seen by the naked eye, one can visualize the nature of the coarse grained particles only.

Coarser fractions composed of angular grains are capable of supporting heavier static loads and can be compacted to a dense mass by vibration. The influence of the shape of the particles on the compressibility characteristics of soils are:

1. Reduction in the volume of mass upon the application of pressure.
2. A small mixture of mica to sand will result in a large increase in its compressibility.

The classification according to size divides the soils broadly into two distinctive groups, namely, coarse grained and fine grained. Since the properties of coarse grained soils are, to a considerable extent, based on grain size distribution, classification of coarse grained soils according to size would therefore be helpful. Fine grained soils are so much affected by structure, shape of grain, geological origin, and other factors that their grain size distribution alone tells little about their physical properties. However, one can assess the nature of a mixed soil on the basis of the percentage of fine grained soil present in it. It is, therefore, essential to classify the soil according to grain size.

The classification of soils as gravel, sand, silt and clay as per the different systems of classification is given in Table 2.2. Soil particles which are coarser than 0.075 mm are generally termed as *coarse grained* and the finer ones as silt, clay and peat (organic soil) are considered *finegrained.* From an engineering point of view, these two types of soils have distinctive characteristics. In coarse grained soils, gravitational forces determine the engineering characteristics. Interparticle forces are predominant in fine grained soils.

The dependence of the behavior of a soil mass on the size of particles has led investigators to classify soils according to their size.

The size of the soil grains is of importance in such cases as construction of earth dams or railroad and highway embankments, where earth is used as a material that should satisfy definite specifications. In foundations of structures, data from mechanical analyses are generally illustrative; other properties such as compressibility and shearing resistance are of more importance. The normal method adopted for separation of particles in a fine grained soil mass is the hydrometer analysis and for the coarse grained soils the sieve analysis. [1]

2.7.1.1 Sieve analysis

Sieve analysis is carried out by using a set of standard sieves. Sieves are made by weaving two sets of wires at right angles to one another. The square holes thus formed between the wires provide the limit which determines the size of the particles retained on a particular sieve. The sieve sizes are given in terms of the number of openings per inch. The number of openings per inch varies according to different standards. Thus, an ASTM 60 sieve has 60 openings per inch width with each opening of 0.250 mm. Table 3.2 gives a set of ASTM Standard Sieves (same as US standard sieves).The usual procedure is to use a set of sieves which will yield equal grain size intervals on a logarithmic scale. A good spacing of soil particle diameters on the grain size distribution curve will be obtained if a nest of sieves is used in which each sieve has an opening approximately one-half of the coarser sieve above it in the nest. If the soil contains gravel, the coarsest sieve that can be used to separate out gravel from sand is the No. 4 Sieve (4.75 mm opening). To separate out the silt-clay fractions from the sand fractions, No. 200 sieve may be used. The intermediate sieves between the coarsest and the finest may be selected on the basis of the principle explained earlier. The nest of sieves consists of Nos 4 (4.75 mm), 8 (2.36 mm), 16 (1.18 mm) 30 (600 µm), 50 (300 µm), 100 (150 µm), and 200 (75 µm). [1]

Table 2.2 US Standard sieves

Designation	Opening(mm)	Designation	Opening(mm)
2 in	50.80	35	0.50
$1^1/_2$ in	38.10	40	0.425
$^3/_4$ in	19.00	50	0.355
$^3/_8$ in	9.51	60	0.250
4	4.75	70	0.212
8	2.36	80	0.180
10	2.00	100	0.150
14	1.40	120	0.125
16	1.18	170	0.090
18	1.00	200	0.075
30	0.60	270	0.053

2.7.1.2 The hydrometer analysis

The hydrometer analysis is a widely used method to obtain the distribution of particle size in the silt range (2-63µm),and the percentage of clay mineral <2µm.the test is usually not performed if less than 10% of the material passes the 63µm sieve. The hydrometer analysis utilizes the relationship among the velocity of fall of spheres in a fluid, the diameter of the sphere, the specific weights of the sphere and of the fluid, and of the viscosity of the fluid as expressed by stokes low.

Each density reading taken on the hydrometer must first be expressed as a hydrometer reading, Rh' corresponding to the level of the upper rimof the meniscus. This is done by subtracting 1 from the density and moving the decimal point three places to the right.

$$\mathbf{Rh=Rh' + C_m} \qquad \text{equation (2.1)}$$

Scale calibration of hydrometer:

The effective depth, HR (mm), is corresponding to each of the major calibration marks, Rh from the equation:

$$\mathbf{H_R = H+0.5(h- (V_h/900)*L)} \qquad \text{equation (2.2)}$$

Where:

H ≡ length from the neck of the bulb to graduation Rh.

h ≡ length of the bulb =159mm for B.S.hydrometer.

$\mathbf{V_h}$≡ volume ~~o~~f hydrometer bulb=70 ml for B.S.hydrometer.

L ≡ distance between the 100ml and the 1000ml scale marking of the sedimentation cylinder.[2]

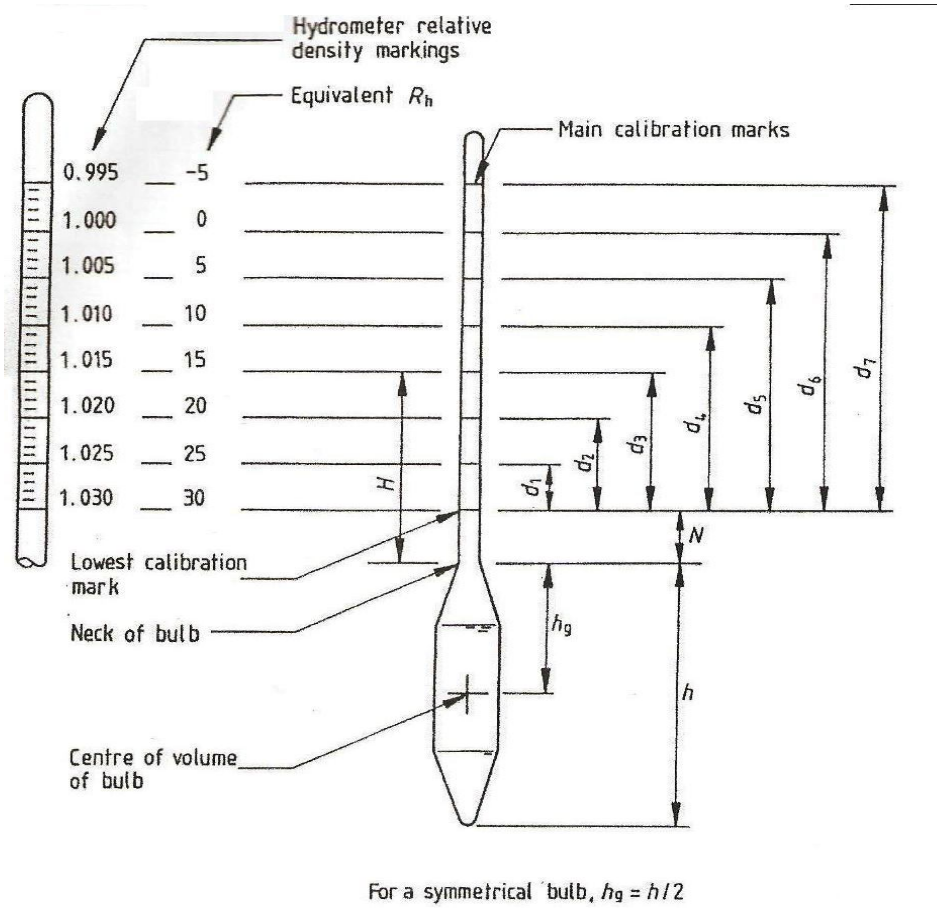


Figure (2.3) Essential measurement for calibration of hydrometer method

2.7.1.3 Grain size distribution curves

A typical set of grain size distribution curves is given in Fig. 2.4 with the grain size D as the abscissa on the logarithmic scale and the percent finer P as the ordinate on the arithmetic scale. On the curve $C_1$the section AB represents the portion obtained by sieve analysis and the section $B'C'$ by hydrometer analysis. Since the hydrometer analysis gives equivalent diameters which are generally less than the actual sizes, the section $B'C'$ will not be a continuation of AB and would occupy a position shown by the dotted curve.

If we assume that the curve *BC* is the actual curve obtained by sketching it parallel to*B'C'*, then at any percentage finer, say 20 per cent, the diameters *Da* and *De* represent the actual and equivalent diameters respectively.

The shapes of the curves indicate the nature of the soil tested. On the basis of the shapes we can classify soils as:

1. Uniformly graded or poorly graded.
2. Well graded.
3. Gap graded.

Uniformly graded soils are represented by nearly vertical lines as shown by curve C2 in Fig. 2.4. Such soils possess particles of almost the same diameter. A well graded soil, represented by curve Cp possesses a wide range of particle sizes ranging from gravel to clay size particles. A gap graded soil, as shown by curve C3 has some of the sizes of particles missing.

The grain distribution curves as shown in Fig. 2.4 can be used to understand certain grain size characteristics of soils. Hazen (1893) has shown that the permeability of clean filter sands in a loose state can be correlated with numerical values designated D10, the effective grain size. The effective grain size corresponds to 10 per cent finer particles. Hazen found that the sizes smaller than the effective size affected the functioning of filters more than did the remaining 90 per cent of the sizes.

To determine whether a material is uniformly graded or well graded, Hazen proposed the following equation:

$$C_u = D_{60}/D_{10} \qquad \textbf{equation (2.3)}$$

Where D60 is the diameter of the particle at 60 per cent finer on the grain size distribution curve. The *uniformity coefficient, Cu,* is about one if the grain size distribution curve is almost vertical, and the value increases with gradation.

For all practical purposes we can consider the following values for granular soils.

Cu >4 for well graded gravel

Cu >6 for well graded sand

Cu<4 for uniformly graded soil containing particles of the same size.

There is another step in the procedure to determine the gradation of particles. This is based on the term called the *coefficient of curvature* which is expressed as

$$\mathbf{Cc=D_{30}^{2}/\left(D_{10}*D_{60}\right)} \qquad \textbf{equation (2.4)}$$

Where in D30 is the size of particle at 30 percent finer on the gradation curve. The soil is said to be well graded if Cc lies between 1 and 3 for gravels and sands.[1]

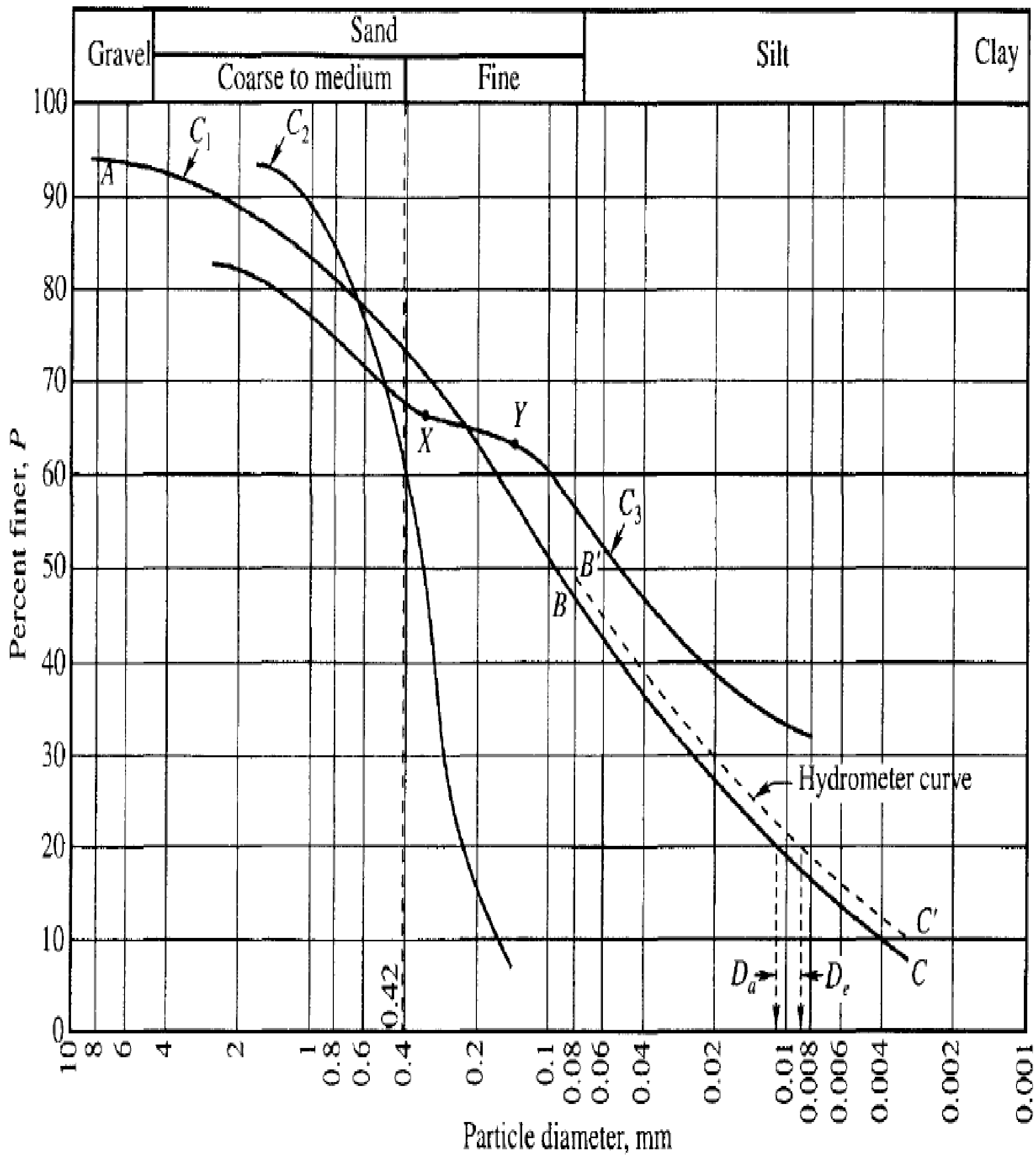
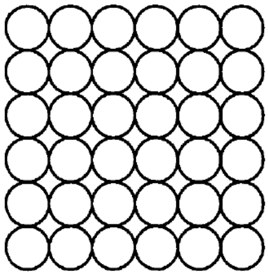


Figure (2.4) grain size distribution curves

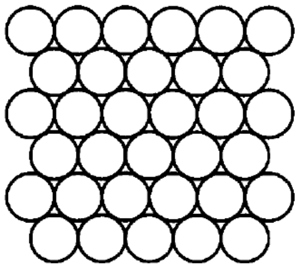
2.7.2 Relative density of cohesionless soils

The density of granular soils varies with the shape and size of grains, the gradation and the manner in which the mass is compacted. If all the grains are assumed to be spheres of uniform size and packed as shown in Fig. 2.5(a), the void ratio of such a mass amounts to about 0.90. However, if the grains are packed as shown in Fig. 2.5(b), the void ratio of the mass is about 0.35.

The soil corresponding to the higher void ratio is called loose and that corresponding to the lower void ratio is called dense.



(a)Loosest state



(b) Densest state

Figure (2.5) packing of grains of uniform size

If the soil grains are not uniform, then smaller grains fill in the space between the bigger ones the void ratios of such soils are reduced to as low as 0.25 in the densest state. If the grains are angular, they tend to form looser structures than rounded grains because their sharp edges and points hold the grains further apart. If the mass with angular grains is compacted by vibration, it forms a dense structure.

The change in void ratio would change the density and this in turn changes the strength characteristics of granular soils. Void ratio or the unit weight of soil can be used to compare the strength characteristics of samples of granular soils of the same origin. [1]

2.8 Consistency of clay soil

Consistency is a term used to indicate the degree of firmness of cohesive soils. The consistency of natural cohesive soil deposits is expressed qualitatively by such terms as very soft, soft, stiff, very stiff and hard. The physical properties of clays greatly differ at different water contents. A soil which is very soft at a higher percentage of water content becomes very hard with a decrease in water content. However, it has been found that at the same water content, two samples of clay of different origins may possess different consistency. One clay may be relatively soft while the other may be hard. Further, a decrease in water content may have little effect on one sample of clay but may transform the other sample from almost a liquid to a very firm condition. Water content alone, therefore, is not an adequate index of consistency for engineering and many other purposes. Consistency of a soil can be expressed in terms of:

1. Atterberg limits of soils
2. Unconfmed compressive strengths of soils.

2.8.1 Atterberg limits:

Atterberg, a Swedish scientist, considered the consistency of soils in 1911, and proposed a series of tests for defining the properties of cohesive soils. These tests indicate the range of the plastic state (plasticity is defined as the property of cohesive soils which possess the ability to undergo changes of shape without rupture) and other states. He showed that if the water content of a thick suspension of clay is gradually reduced, the clay water mixture undergoes changes from a liquid state through a plastic state and finally into a solid state. The different states through which the soil sample passes with the decrease in the moisture content are depicted in Fig. 3.9. The water contents corresponding to the transition from one state to another are termed as *Atterberg Limits* and the tests required determining the limits are the *Atterberg Limit Tests.* The testing procedures of Atterberg were subsequently improved by A. Casagrande (1932).

The transition state from the liquid state to a plastic state is called the *liquid limit*, w_lAt this stage all soils possess a certain small shear strength. This arbitrarily chosen shear strength isprobably the smallest value that is feasible to measure in a standardized procedure. The transition from the plastic state to the semisolid state is termed *the plastic limit,* w_P .At this state the soil rolled into threads of about 3 mm diameter just crumbles. Further decrease of the water contents of the same will lead finally to the point where the sample can decrease in volume no further. At this point the sample begins to dry at the surface, saturation is no longer complete, and further decrease inwater in the voids occurs without change in the void volume. The color of the soil begins to change from dark to light. This water content is called the *shrinkage limit,* w_s. The limits expressed above are all expressed by their percentages of water contents. The range of water content between the liquid and plastic limits, which is an important measure of plastic behavior, is called *the plasticity index*, I_P}.[1]

$$I_P=w_l-w_P \qquad \textbf{equation (2.5)}$$

2.9 General considerations for classification of soils

It has been stated earlier that soil can be described as gravel, sand, silt and clay according to grain size. Most of the natural soils consist of a mixture of organic material in the partly or fully decomposed state. The proportions of the constituents in a mixture vary considerably and there is no generally recognized definition concerning the percentage of, for instance, clay particles that a soil must have to be classified as clay, etc.

When a soil consists of the various constituents in different proportions, the mixture is then given the name of the constituents that appear to have significant influence on its behavior, and then other constituents are indicated by adjectives. Thus sandy clay has most of the properties of clay but contains a significant amount of sand.The individual constituents of a soil mixture can be separated and identified as gravel, sand, silt and clay on the basis of mechanical analysis. The clay mineral that is present in a clay soil is sometimes a matter of engineering importance.

According to the mineral present, the clay soil can be classified as kaolinite, montmorillonite or illite. The minerals present in clay can be identified by either X-ray diffraction or differential thermal analysis.

The behavior of a soil mass under load depends upon many factors such as the properties of the various constituents present in the mass, the density, the degree of saturation, the environmental conditions etc. If soils are grouped on the basis of certain definite principles and rated according to their performance, the properties of a given soil can be understood to a certain extent, on the basis of some simple tests. [1]

2.9.1 Field identification of soils

The methods of field identification of soils can conveniently be discussed under the coarse-grained and fine-grained soil materials.

2.9.1.1 Coarse-grained soil materials

The coarse-grained soil materials are mineral fragments that may be identified primarily on the basis of grain size. The different constituents of coarse-grained materials are sand and gravel. As described in the earlier sections, the size of sand varies from 0.075 mm to 4.75 mm and that of gravel from 4.75 mm to 80 mm. Sand can further be classified as coarse, medium and fine.

The engineer should have an idea of the relative sizes of the grains in order to identify the various fractions. The description of sand and gravel should include an estimate of the quantity of material in the different size ranges as well as a statement of the shape and mineralogical composition of the grains. The mineral grains can be rounded, subrounded, angular or subangular. The presence of mica or a weak material such as shale affects the durability or compressibility of the deposit.

A small magnifying glass can be used to identify the small fragments of shale or mica. The properties of a coarse grained material mass depend also on the uniformity of the sizes of the grains. Well-graded sand is more stable for a foundation base as compared to a uniform or poorly graded material.

2.9.1.2 Fine-grained soil materials

Inorganic Soils: The constituent parts of fine-grained materials are the silt and clay fractions. Since both these materials are microscopic in size, physical properties other than grain size must be used as criteria for field identification. The classification tests used in the field for preliminary identification are

1. Dry strength test
2. Shaking test
3. Plasticity test
4. Dispersion test

Dry strength: The strength of a soil in a dry state is an indication of its cohesion and hence of its nature. It can be estimated by crushing a 3 mm size dried fragment between thumb and forefinger. A clay fragment can be broken only with great effort, whereas a silt fragment crushes easily.

Shaking test: The shaking test is also called as dilatancy test. It helps to distinguish silt from clay since silt is more permeable than clay. In this test a part of soil mixed with water to a very soft consistency is placed in the palm of the hand. The surface of the soil is smoothed out with a knife and the soil pat is shaken by tapping the back of the hand.

If the soil is silt, water will rise quickly to the surface and give it a shiny glistening appearance. If the pat is deformed either by squeezing or by stretching, the water will flow back into the soil and leave the surface with a dull appearance.

Since clay soils contain much smaller voids than silts and are much less permeable, the appearance of the surface of the pat does not change during the shaking test.

An estimate of the relative proportions of silt and clay in an unknown soil mixture can be made by noting whether the reaction is rapid, slow or nonexistent.

Plasticity test: If a sample of moist soil can be manipulated between the palms of the hands and fingers and rolled into a long thread of about 3 mm diameter, the soil then contains a significant amount of clay. Silt cannot be rolled into a thread of 3 mm diameter without severe cracking.

Dispersion test: This test is useful for making a rough estimate of sand, silt and clay present in a material. The procedure consists in dispersing a small quantity of the soil in water taken in a glass cylinder and allowing the particles to settle. The coarser particles settle first followed by finer ones. Ordinarily sand particles settle within 30 seconds if the depth of water is about 10 cm.Silt particles settle in about 1/2 to 240 minutes, whereas particles of clay size remain in suspension for at least several hours and sometimes several days.

Organic soils: Surface soils and many underlying formations may contain significant amounts of solid matter derived from organisms. While shell fragments and similar solid matter are found at some locations, organic material in soil is usually derived from plant or root growth and consists of almost completely disintegrated matter, such as muck or more fibrous material, such as peat. The soils with organic matter are weaker and more compressible than soils having the same mineral composition but lacking in organic matter.

The presence of an appreciable quantity of organic material can usually be recognized by the dark-grey to black color and the odor of decaying vegetation which it lends to the soil.

Organic silt: It is a fine grained more or less plastic soil containing mineral particles of silt size and finely divided particles of organic matter. Shells and visible fragments of partly decayed vegetative matter may also be present.

Organic clay: It is a clay soil which owes some of its significant physical properties to the presence of finely divided organic matter. Highly organic soil deposits such as peat or muck may be distinguished by a dark-brown to black color, and by the presen fibrous particles of vegetable matter in varying states of decay. The organic odor is a distinguishing characteristic of the soil. The organic odor can sometimes be distinguished by a slight amount of heat. [1]

2.9.2 Classification of soils

Soils in nature rarely exist separately as gravel, sand, silt, clay or organic matter, but are usually found as mixtures with varying proportions of these components. Grouping of soils on the basis of certain definite principles would help the engineer to rate the performance of a given soil either as a sub-base material for roads and airfield pavements, foundations of structures, etc.

The classification or grouping of soils is mainly based on one or two index properties of soil. The methods that are used for classifying soils are based on one or the other of the following two broad systems:

1. A textural system which is based only on grain size distribution.
2. The systems that are based on grain size distribution and limits of soil.

Many systems are in uses that are based on grain size distribution and limits of soil. The systems that are quite popular amongst engineers are the AASHTO Soil Classification System and the Unified Soil Classification System.

2.9.2.1 U.S. Department of agriculture system (USDA)

The boundaries between the various soil fractions of this system are given in Table 2.3.

By making use of the grain size limits mentioned in the table for sand, silt and clay, a triangular classification chart has been developed as shown in Fig. 2.6 for classifying mixed soils. The first step in the classification of soil is to determine the percentages of sand, silt and clay-size materials in a given sample by mechanical

analysis. With the given relative percentages of the sand, silt and clay, a point is located on the triangular chart as shown in Fig. 2.6. The designation given on the chart for the area in which the point falls is used as the classification of the sample.
This method of classification does not reveal any properties of the soil other than grain-size distribution. Because of its simplicity; it is widely used by workers in the field of agriculture. One significant disadvantage of this method is that the textural name as derived from the chart does not always correctly express the physical characteristics of the soil.
For example, since some clay size particles are much less active than others, a soil described as clay on the basis of this system may have physical properties more typical of silt. [1]

Table (2.3) Soil fraction as per U.S. department of agriculture

Soil fraction	Diameter in mm
Gravel	>2.00
Sand	2 - 0.05
Silt	0.05 - 0.002
Clay	<0.002

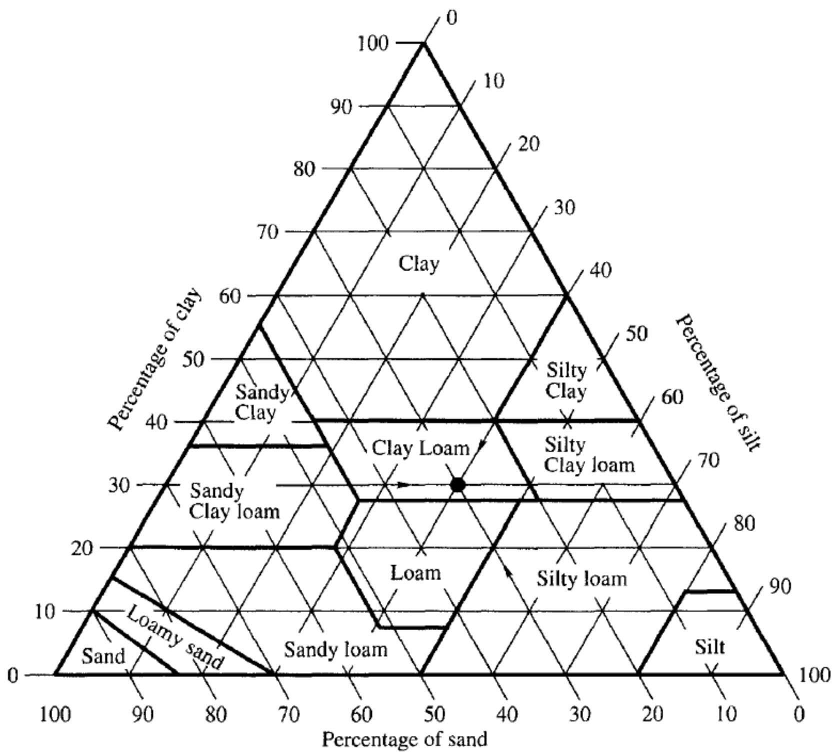


Figure (2.6) U.S. Department of Agriculture textural classification

2.9.2.2 AASHTO soil classification system

This system was originally proposed in 1928 by the U.S. Bureau of Public Roads for use by highway engineers. A Committee of highway engineers for the Highway Research Board, met in 1945. This system is known as the AASHTO (American Association of State Highway and Transportation Officials) System (ASTM D-3242, AASHTO Method M 145).

The revised system comprises seven groups of inorganic soils, A-l to A-7 with 12 subgroups in all. The system is based on the following three soil properties:

1. Particle-size distribution
2. Liquid Limit
3. Plasticity Index.

A Group Index is introduced to further differentiate soils containing appreciable fine-grained materials. The characteristics of various groups are defined in Table 2.4.

The Group Index may be determined from the equation.

$$(GI) = 0.2a + 0.005ac + 0.01\,bd \qquad \textbf{equation (5.2a)}$$

In which,

a = that portion of percentage of soil particles passing No. 200 (ASTM) sieve greater than 35 = (F-35).

b = that portion of percentage of soil particles passing No. 200 sieve, greater than 15 = (F -15).

c = that portion of the liquid limit greater than 40 = (w_I- 40).

d = that portion of the plasticity index greater than 10 = (I_p-10).

F = percent passing No. 200 sieve. If F <35, use (F -35) = 0

It may be noted here that if GI <0, use GI = 0. There is no upper limit for GI. When calculating the GI for soils that belong to groups A-2-6 and A-2-7, use the partial group index *(PGI)* only, that is (From Eq. 2.5a)

$$PGI = 0.01bd = 0.01(F - 15)\,(I_p - 10) \qquad \textbf{equation (2.5b)}$$

Figure 2.7 provides a rapid means of using the liquid and plastic limits (and plasticity index I_p) to make determination of the A-2 subgroups and the A-4 through A-7 classifications.

Figure 2.7 is based on the percent passing the No. 200 sieve (whether greater or less than 35 percent).

The group index is a means of rating the value of a soil as a subgrade material within its own group.

It is not used in order to place a soil in a particular group that is done directly from the results of sieve analysis, the liquid limit and plasticity index. The higher the value of the group index, the poorer is the quality of the material. The group index is a function of the amount of material passing the No. 200 sieve, the liquid limit and the plasticity index.

If the pertinent index value for a soil falls below the minimum limit associated with *a, b, c* or *d,* the value of the corresponding term is zero, and the term drops out of the group index equation. The group index value should be shown in parenthesis after a group symbol such as A-6(12) where 12 is the group index. [1]

2.9.2.2.1 Classification procedure

With the required data in mind, proceed from left to right in the chart. The correct group will be found by a process of elimination. The first group from the left consistent with the test data is the correct classification. The A-7 group is subdivided into A-7-5 or*A-l-6* depending on the plasticity index, I_p.

For A-7-5, $I_p < w_l - 30$

ForA-7-6, $I_p > w_l - 30$

Table (2.4) AASHTO Soil Classification

General classification	Granular Materials (35 percent or less of total sample passing No. 200)							Silt-clay Materials (More than 35 percent of total sample passing No. 200)			
Group classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve analysis percent passing											
No. 10	50 max										
No. 40	30 max	50 max	51 min								
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 No. 40											
Liquid limit				40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index	6 max		N.P.	10 max	10 max	11 min	11 max	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			

rved.

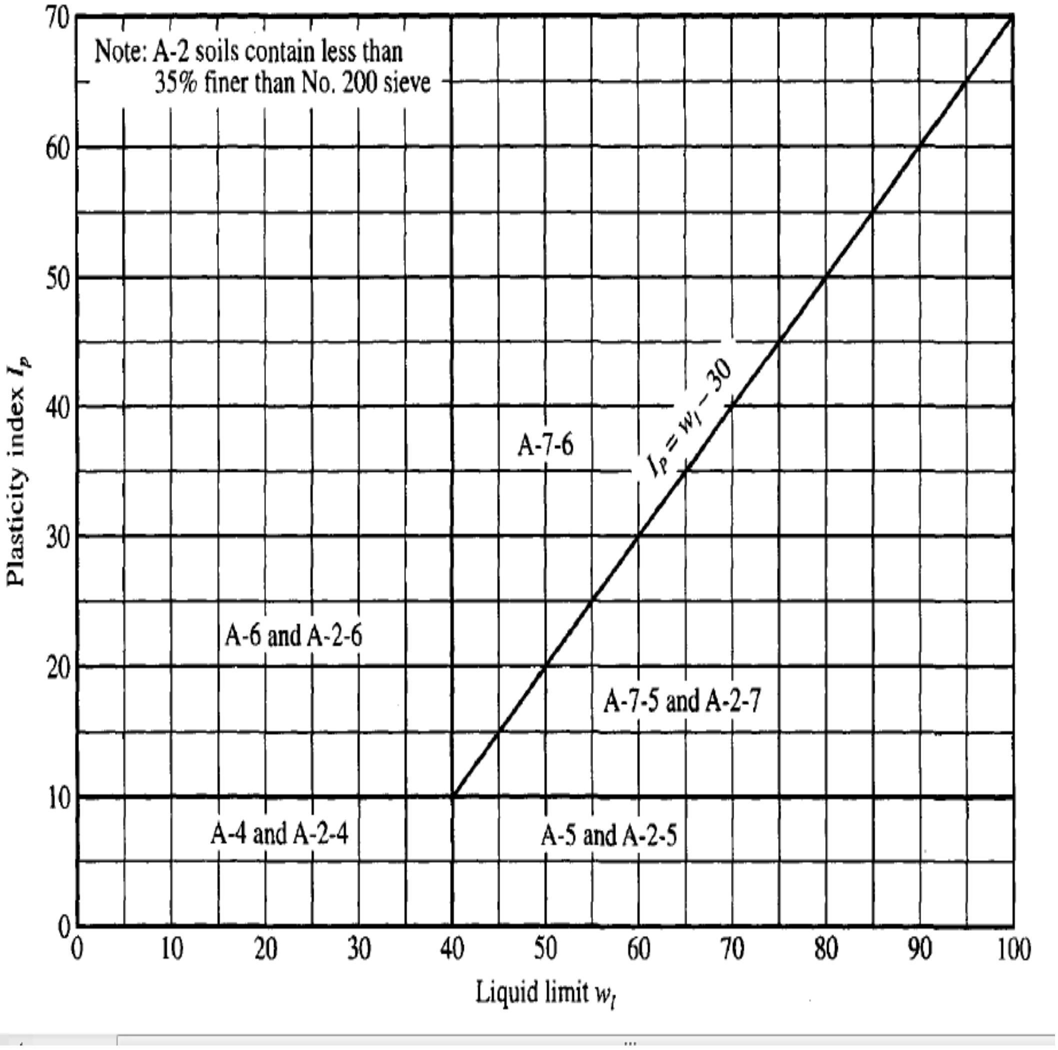


Figure (2.7) Chart for use in AASHTO soil classification system[1]

2.9.2.3 Unified Soil Classification System (USCS)

The Unified Soil Classification System is based on the recognition of the type and predominance of the constituents considering grain-size, gradation, plasticity and compressibility. It divides soil into three major divisions: coarse-grained soils, fine grained soils, and highly organic (peaty) soils. In the field, identification is accomplished by visual examination for the Coarse-grained soils and a few simple hand tests for the fine-grained soils. In the laboratory, the grain-size curve and the Atterberg limits can be used.

The Unified Soil Classification System is a modified version of A.Casagrande's Airfield Classification (AC) System developed in 1942 for the Corps of Engineers. Since 1942 the original classification has been expanded and revised in cooperation with the Bureau of Reclamation, so that it applies not only to airfields but also to embankments, foundations, and other engineering features. This system was adopted in 1952. In 1969 the American Society for Testing and Materials (ASTM) adopted the Unified System as a standard method for classification for engineering purposes (ASTM Test Designation D-2487).

Table 2.5 presents the primary factors to consider in classifying a soil according to the Unified Soil Classification system.

The following subdivisions are considered in the classification:

1. Gravels and sands are GW, GP, SW, or SP if less than 5 percent of the material passes the No. 200 sieve; *G* = gravel; *S* = sand; *W* = well-graded; *P* = poorly-graded. The well- or poorly-graded designations depend on Cc and Cu.

2. Gravels and sands are GM, GC, SM, or SC if more than 12 percent passes the No. 200 sieve; *M* = silt; *C* = clay. The silt or clay designation is determined by performing the liquid and plastic limit tests on the (-) No. 40 fraction and using the plasticity chart of Fig. 2.8.This chart is also a Casagrande contribution to the USC system, and the A line shown on this chart is sometimes called Casagrande's A line. [1]

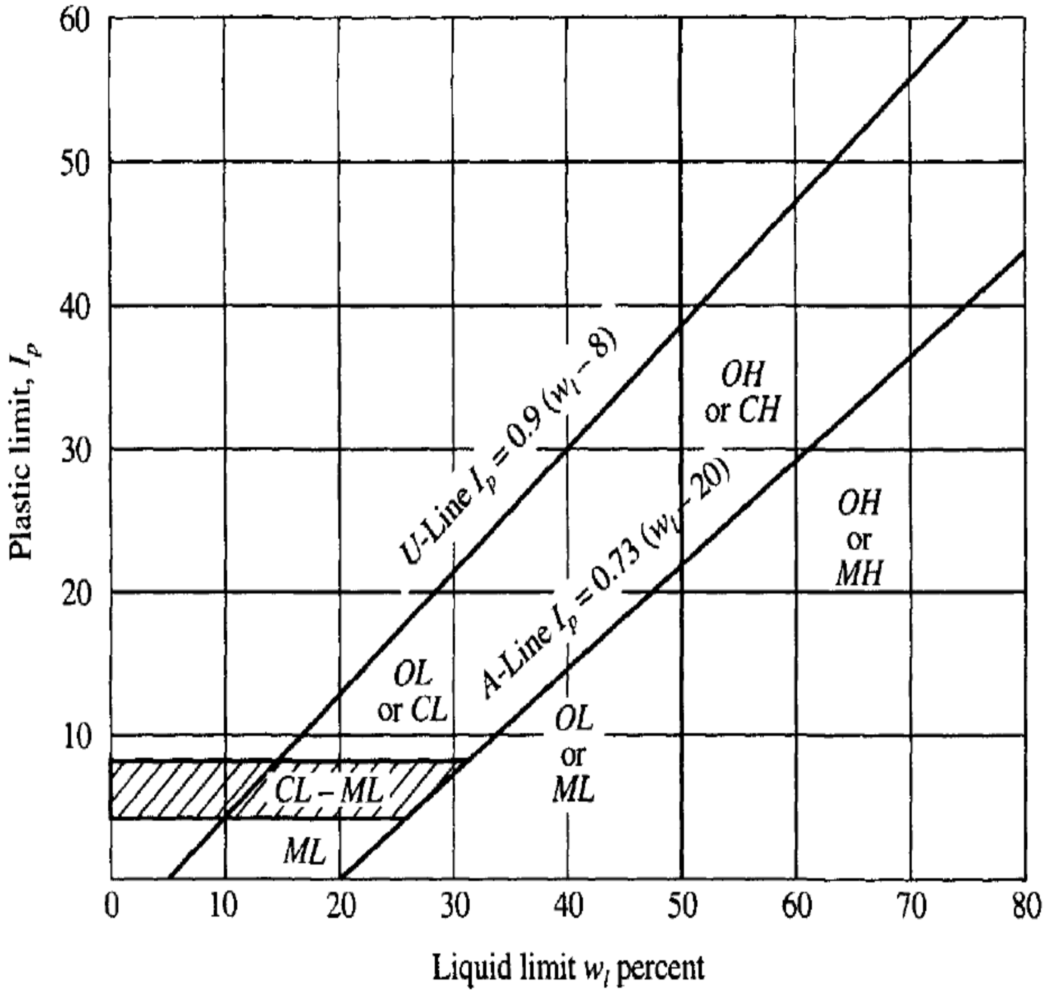


Figure (2.8) plasticity chart for fine - grained soils[1]

Table (2.5) the unified soil classification system

Major divisions			Group symbol	Typical names	Classification criteria for coarse-grained soils	
Coarse-grained soils (more than half of material is larger than No. 200)	Gravels (more than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	$C_u \geq 4$ $1 \leq C_c \leq 3$	
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines	Not meeting all gradation requirements for GW ($C_u < 4$ or $1 > C_c > 3$)	
		Gravels with fines (appreciable amount of fines)	GM $\frac{d}{u}$	Silty gravels, gravel-sand-silt mixture	Atterberg limits below *A* line or $I_p < 4$	Above A line with $4 < I_p < 7$ are borderline cases requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixture	Atterberg limits above *A* line with $I_p > 7$	
	Sands (more than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines	$C_u \geq 6$ $1 \leq C_c \leq 3$	
			SP	Poorly graded sands, gravelly sands, little or no fines	Not meeting all gradation requirements for SW ($C_u < 6$ or $1 > C_c > 3$)	
		Sands with fines (appreciable amount of fines)	SM $\frac{d}{u}$	Silty sands, sand-silt mixture	Atterberg limits below *A* line or $I_p < 4$	Above A line with $4 \leq I_p \leq 7$ are borderline cases requiring use of dual symbols
			SC	Clayey sands, sand-silt mixture	Atterberg limits above *A* line with $I_p > 7$	
Fine-grained soils (more than half of material is smaller than No. 200)	Silts and clays (liquid limit < 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity	1. Determine percentages of sand and gravel from grain-size curve. 2. Depending on percentages of fines (fraction smaller than 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 dual symbols	
			CL	Inorganic clays of very low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays		
			OL	Organic silts and organic silty clays of low plasticity		
	Silts and clays (liquid limit > 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts		
			CH	Inorganic clays or high plasticity, fat clays		
			OH	Organic clays of medium to high plasticity, organic silts		
	Highly organic soils		Pt	Peat and other highly organic soils		

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

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

The chart as presented here has been slightly modified based on the Corps of Engineers findings that no soil has so far been found with coordinates that lie above the "upper limit" or *U* line shown.

This chart and lines are part of the ASTM D 2487 standard.

3. Gravels and sands are GW-GC SW-SC GP-GC SP-SC, or GW-GM SW-SM GP-GM SP-SM if between 5 and 12 percent of the material passes the No. 200 sieve. It may be noted that the M or C designation is derived from performing plastic limit tests and using Casagrande's plasticity chart.

4. Fine-grained soils (more than 50 percent passes the No. 200 sieve) are:

ML, OL, or CL

If the liquid limits are < 50 percent; M = silt; O = organic soils; C = clay. L = Less than 50 percent for w_I

5. Fine grained soils are

MH, OH, or CH

If the liquid limits are > 50 percent; H = Higher than 50 percent. Whether a soil is Clay (C), Silt (M), or Organic (O) depends on whether the soil coordinates plot above or below the A line on Fig. 2.8.

The organic (O) designation also depends on visual appearance and odor in the USC method. In the ASTM method the O designation is more specifically defined by using a comparison of the air-dry liquid limit wI and the oven-dried w'I. If the oven dried value is 0.75w and the appearance and odors indicates "organic" then classify the soil as O. The liquid and plastic limits are performed on the (-) No. 40 sieve fraction of all of the soils, including gravels, sands, and the fine-grained soils. Plasticity limit tests are not required for soils where the percent passing the No. 200 sieve < 5 percent.

The identification procedures of fine grained soils are given in Table 2.6.

A visual description of the soil should accompany the letter classification. The ASTM standard includes some description in terms of sandy or gravelly, but color is also very important.[1]

Certain areas are underlain with soil deposits having a distinctive color (e.g., Boston blue clay, Chicago blue clay) which may be red, green, blue, grey, black, and so on. Geotechnical engineers should become familiar with the characteristics of this material so the color identification is of considerable aid in augmenting the data base on the soil.[1]

Table (2.6) Unified Soil Classification System-fine-grained soils (more than half of material is larger than No.200 sieve size)

Soil	**Major division**	**Group symbols**	**Identification procedures on fraction smaller than No.40 sieve size**		
			Dry strength	**Dilatancy**	**Toughness**
Silt and Clay	Liquid limit less than 50	ML	None to slight	Quick to slow	None
		CL	Medium to high	None to very slow	Medium
		OL	Slight to medium	Slow	Slight
	Liquid limit more than 50	MH	Slight to medium	Slow to none	Slight to medium
		CH	High to very high	None	High
		OH	Medium to high	None to very slow	Slight to medium
Highly organic soil		Pt Readily identified by color,oder,spongy feel and frequently by fibrous texture			

2.10 Comments on the systems of soil classification

The various classification systems described earlier are based on:

1. The properties of soil grains.

2. The properties applicable to remolded soils.

The systems do not take into account the properties of intact materials as found in nature. Since the foundation materials of most engineering structures are undisturbed, the properties of intact materials only determine the soil behavior during and after construction. The classification of a soil according to any of the accepted systems does not in itself enable detailed studies of soils to be dispensed with altogether. Solving flow, compression and stability problems merely on the basis of soil classification can lead to disastrous results. However, soil classification has been found to be a valuable tool to the engineer. It helps the engineer by giving general guidance through making available in an empirical manner the results of field experience. [1]

Chapter (3)

BEARING CAPACITY

3.1 Bearing capacity:

Bearing capacity can be defined as is the ability of soil to safely carry the pressure placed on the soil from any engineering structure without undergoing a shear failure with accompanying large settlement.

When excessive load is transmitted to the soil by structure foundation, the settlement of the foundation takes place which can endanger the stability of the structure. The settlement due to load is caused basically on account of two factors:

i. The soil below footing gets compressed by a certain amount.
ii. Since the foundations cover only a limited area there is a possibilitythat the concentrated stresses developed are so high as to cause actual rupture (shear failure) and displacement of soil below.

Past experience shows that very often a structure fails due to unequal settlement or differential settlement. This happens when a part or building is founded on compressible stratum and the remaining part rests on firm soil strata. Thus the part of the building on compressible soil settles at a rate well in excess of the part of building on firm soil leading to the differential settlement. Differential settlement can also occur when one part of the building is loaded much more than other or intensity of load is varying and is more than the bearing capacity of soil incase, however the settlement is uniform and small in magnitude it does not endanger the structure in any way. [2]

3.2 Bearing capacity failure:

A bearing capacity failure is defined as a foundation Failure that occurs when the shear stresses in the soil exceed the shear strength of the soil.

Bearing capacity failures of foundation can be grouped in to three categories:-

1. General Shear:-

A general shear failure involves total rupture of the under lying soil. There is a continuous shear failure of the soil from below the footing to the ground surface. A general shear failure occurs for soils that are in a dense or hard state.

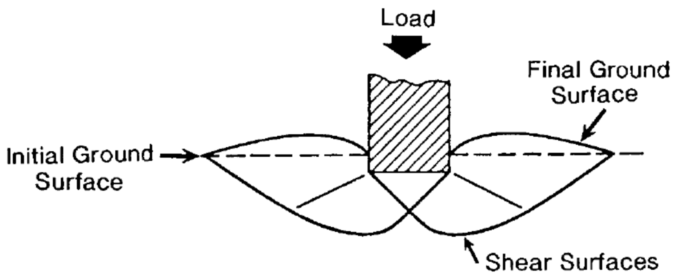


Figure (3.1) General Shear

2. Punching shear:-

A punching shear failure does not develop the distinct shear surface associated with a general shear failure. For punching shear, the soil outside the loaded area remains relatively uninvolved and there is minimal movement of soil on both sides of the footing.

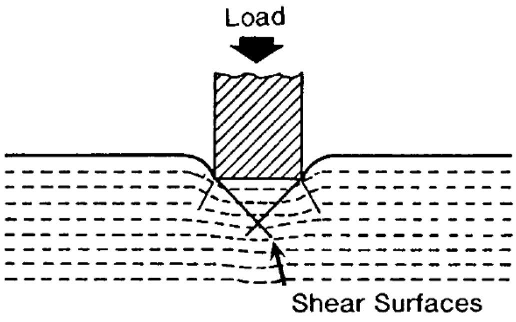


Figure (3.2) Punching Shear

3. Local shear failure:-

Local shear failure involves of the soil only immediately below the footing. There is soil building on bath side of the footing, but the building is not as a significant as in general shear. Local shear failure can be considered as a transition phase between general shear and punching shear. [3]

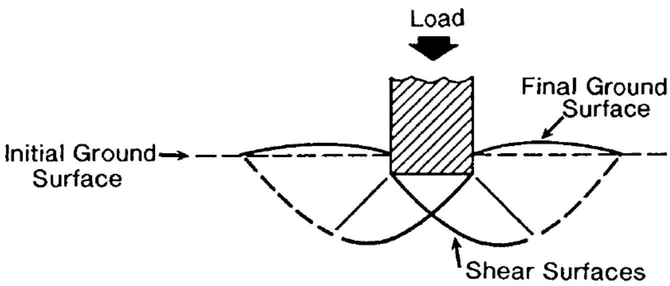


Figure (3.3) Local Shear Failure

3.3 Bearing capacity terms:-

The various terms which are used in connection with the bearing capacity of soils are:

3.3.1 Ultimate bearing capacity of soil:

The intensity of loading at the base of foundation, at which soil support fails in shear, is called ultimate bearing capacity of soils. [2]

3.3.2 Net ultimate bearing capacity:

It is defined as the ultimate pressure per unit area of the foundation that can be supported by the soil in excess of the pressure censed by the surrounding soil at the foundation level. [4]

$$q_{net(u)} = q_u - q \qquad \textbf{equation(3.1)}$$

Where:

$q_{net(u)}$ ≡ Net ultimate bearing capacity.

q_u ≡ Ultimate bearing capacity.

q ≡ Equivalent surcharge = γD_f.

3.3.3 Allowable load bearing capacity:

It is the maximum allowable net loading intensity which can be applied to the soil taking into account the ultimate bearing capacity, the amount and kind of settlement expected and the ability of the given structure to with stand the settlement. It is, therefore, dependent upon both the sub-soil. And the type of building proposed to be erected thereon.

The allowable bearing pressure adopted in the design of foundation is lesser of the following two values:

(a) The safe bearing capacity of soil.
(b) The maximum allowable bearing pressure that the soil can take without exceeding the specified limits of permissible settlement.[2]

$$q_{all(net)} = \frac{q_u - q}{Fs} \quad \textbf{equation (3.2)}$$

To calculate net allowable bearing capacity with respect to shear failure, the developed cohesion and the angle of friction are:

$$C_d = \frac{c}{Fs\,shear} \quad \textbf{equation (3.3)}$$

$$Q_d = \frac{tan^{-1}(\tan \emptyset)}{Fs\,shear} \quad \textbf{equation (3.4)}$$

The factor of safety should be at safety should be at least 3 in all cases. Another type of factor of safety for the bearing capacity of shallow foundation is often used. It is the factor of safety for the bearing capacity of shallow foundation is often used. It is the factor of safety with respect to shear failure (FS shear). In most cases, a rule of FS shear = 1.4 to 1.6 is desirable a long with a minimum factor of safety of 3 to 4 against net ultimate bearing capacity.[4]

3.3.4 Safe bearing capacity:

The maximum intensity of loading that the soil will safely carry without risk of shear failure is called safe bearing capacity of soil. This is obtained by dividing the ultimate bearing capacity by a certain factor of safety, and it is the value which is used in the design of foundation. The bearing capacity study of shallow footing is a subject with a very long reference list. The basic structure of formula used for calculations of bearing capacity today however, is no different from that proposed by Terzaghi in 1943. the first important contribution are due to Prandtl (1921) and Reissner (1924), who considered a punch over a weightless semi finite space, and Sokolovski (1965), in regard to a ponder able soil, all under plane strain conditions. Meyerhof (1951) obtained, with a similar technique of Terzaghi's approach, approximate solution to the plastic equilibrium of shallow foundations and deep foundation, assuming a different failure mechanism and like Terzaghi, expressing the results in the form of bearing capacity factors in terms of the angle of internal friction Ø. [2]

3.4 Bearing capacity values for different types of soil:

Table (3.1):

The International Building Code, like the CABO code (The Council of American Building Officials) , lists presumed bearing strengths for different types of soils. Very fine soils (clays and silts) typically have lower capacities than coarse granular soils (sands and gravels).[13]

Class of materials	Load-bearing pressure (pound per square foot)
Crystalline bedrock	12000
Sedimentary rock	6000
Sandy gravel or gravel	5000
Sand, silty sand, clayey sand, silty gravel, and clayey gravel	3000
Clay, sandy clay, silty clay, and clayey silt	2000

Table (3.2):

Presumptive bearing capacity: Building codes of various organizations in different countries gives the allowable bearing capacity that can be used for proportioning footings. These are “Presumptive bearing capacity values based on experience with other structures already built. Generally these values are conservative and can be used for preliminary design or even for final design of small unimportant structure. IS1904-1978 recommends that the safe bearing capacity should be calculated on the basis of the soil test.[5]

Types	**Safe/allowable bearing capacity(KN/m^2)**
Rock	3240
Soft rocks	440
Coarse sand	440
Medium sand	245
Fine sand	100
Soft shale/stiff clay	440
Soft clay	100
Very soft clay	50

Table (3.3):

Soil properties to be considered in foundation design for various types of soil[5]:

Type of soil	Limit bearing capacity (Kg/m^2)
Normal dry soil: (a) Without undercut (b) With undercut	 25000 25000
Wet soil due to presence of subsoil water/surface water	12500
Black cotton soil: (a) In dry portion (b) In wet portion	 12500 12500
Sandy soil: (a) With clay content 0-5% (b) With clay content 5-10%	25000 25000
Fissured rock/soft rock (with undercut): (a) In dry portion (b) In wet portion	 62500 62500
Hard rock	125000
Normal hard dry soil (Morrum) with undercut	40000

Table (3.4):

Other examples of soil bearing capacities (SBC) due to soil description [5]:

Soil description	SBC-ton/ft^2	SBC-KN/m^2
Hardpan overlaying rock	12	1290
Very compact sandy gravel	10	1075
Loose gravel and sandy gravel, compact sand and gravelly sand, very compact sand-inorganic	6	645
Hard, dry, consolidated clay	5	537
Loose coarse to medium sand, medium compact fine sand	4	430
Compact sand clay	3	322
Loose, fine sand, medium compact sand- inorganic silt soils	2	215
Firm or stiff clay	1.5	161
Loose, saturated sand-clay soils, medium soft clay	1	107

3.5 Terzaghi,s bearing capacity theory:

Terzghi (1943) was the first to present a comprehensive theory for the evaluation of the ultimate bearing capacity of rough shallow foundation according to this theory, a foundation is shallow if its depthD_f, is less than or equal to its width. Later investigations, however, have suggested that foundations with $D_f$equal to 3 to 4 times their width may be defined as shallow foundations.

Terzaghi suggested that for a continuous, or strip foundation the failure surface in soil at ultimate load may be assumed to be similar to that shown in figure 3.4.

The effect of soil above the bottom of the foundation may also be assumed to be replaced by an equivalent surcharge q = γ D_f (where γ is a unit weight of soil).[4]

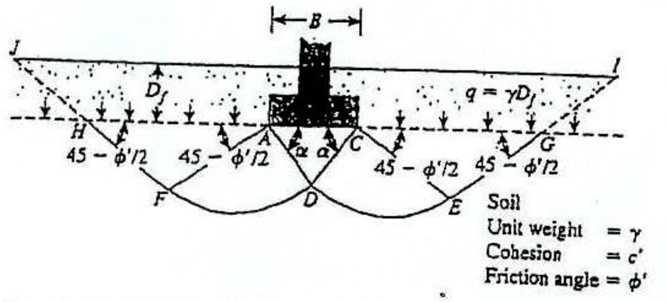


Figure (3.4) failure surface

3.5.1 Terzaghi,s bearing capacity equations:-

Terzaghi,s equation is an approximate solution which uses the superposition technique to combine the effect of cohesion soil weight and surcharge .These contribution are expressed through three factors of bearing capacity,N_c , N_q , N_γ using equilibrium analysis, terzaghi expressed the ultimate bearing capacity in the form.[4]

$$q_u = cN_c + qN_q + \frac{1}{2}\gamma BN_\gamma$$ **(Continuous or strip foundation)** **equation (3.5)**

Where:

C ≡ cohesion of soil.

γ≡ unit weight of soil.

$q = \gamma D_f$

N_c,N_q, N_γ≡ bearing capacity factors that are non-dimensional and are functions only of the soil frictions angle Ø.

The bearing capacity factorsN_c,N_q andN_γ , are defined by:

$$N_c = \cot\emptyset \left[\frac{e^{2(3\pi/4-\emptyset/2)tan\emptyset}}{2cos^2\left(\frac{\pi}{4}+\frac{\emptyset}{2}\right)} - 1\right] = \cot\emptyset(N_q - 1)$$ **equation (3.6)**

$$N_q = \frac{e^{2(3\pi/4-\emptyset/2)tan\emptyset}}{2cos^2\left(45+\frac{\emptyset}{2}\right)}$$ **equation (3.7)**

$$N_\gamma = \frac{1}{2}\left(\frac{K_{P\gamma}}{cos^2\emptyset} - 1\right) tan\emptyset$$ **equation (3.8)**

Where:

$K_{P\gamma}$ = passive pressure coefficient

The variations of the bearing capacity factors defined by Eqs. (3.6), (3.7), and (3.8) are given in table 3.4.

Table (3.5) Terzaghi's Bearing Capacity Factors

Ø	N_c	N_q	N_γ	Ø	N_c	N_q	N_γ
0	5.70	1.00	0.00	26	27.09	14.21	9.84
1	6.00	1.10	0.01	27	29.24	15.90	11.60
2	6.30	1.22	0.04	28	31.61	17.81	13.70
3	6.62	1.35	0.06	29	34.24	19.98	16.18
4	6.97	1.49	0.10	30	37.16	22.46	19.13
5	7.34	1.64	0.14	31	40.41	25.28	22.65
6	7.73	1.81	0.20	32	44.04	28.52	26.87
7	8.15	2.00	0.27	33	48.09	32.23	31.94
8	8.60	2.21	0.35	34	52.64	36.50	38.04
9	9.09	2.44	0.44	35	57.75	41.44	45.41
10	9.61	2.69	0.56	36	63.53	47.16	54.36
11	10.16	2.98	0.69	37	70.01	53.80	65.27
12	10.76	3.29	0.85	38	77.50	61.55	78.61
13	11.41	3.63	1.04	39	85.97	70.61	95.03
14	12.11	4.02	1.26	40	95.66	81.27	115.31
15	12.86	4.45	1.52	41	106.81	93.85	140.51
16	13.68	4.92	1.82	42	119.67	108.75	171.99
17	14.60	5.45	2.18	43	134.58	126.50	211.56
18	15.12	6.04	2.59	44	151.95	147.74	261.60
19	16.56	6.70	3.07	45	172.28	173.28	325.34
20	17.69	7.44	3.64	46	196.22	204.19	407.11
21	18.92	8.26	4.31	47	224.55	241.80	512.84
22	20.27	9.19	5.09	48	258.22	287.85	650.67
23	21.75	10.23	6.00	49	298.71	344.63	831.99
24	23.36	11.40	7.08	50	347.50	415.14	1072.80
25	25.13	12.72	8.34				

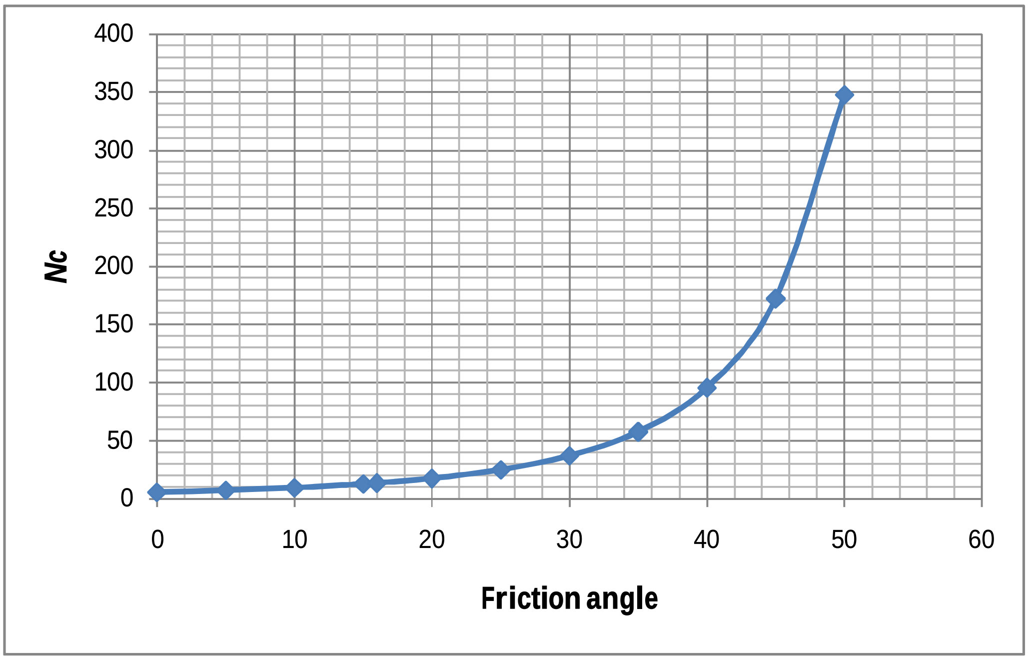


Figure (3.5)Terzaghi's Bearing Capacity Factor *Nc*

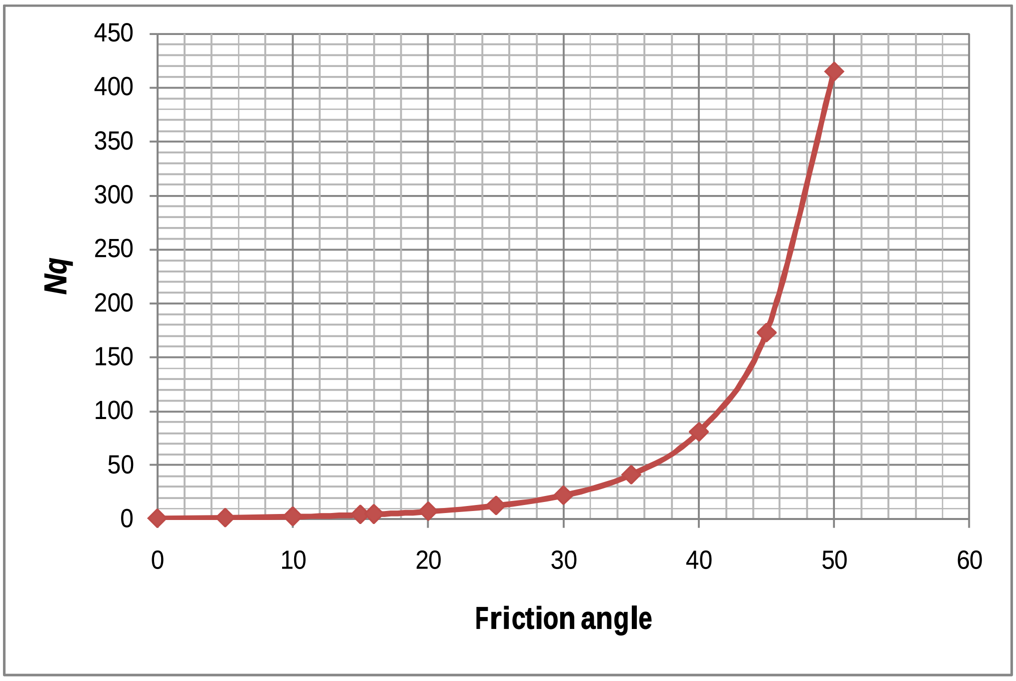


Figure (3.6)Terzaghi's Bearing Capacity Factor *Nq*

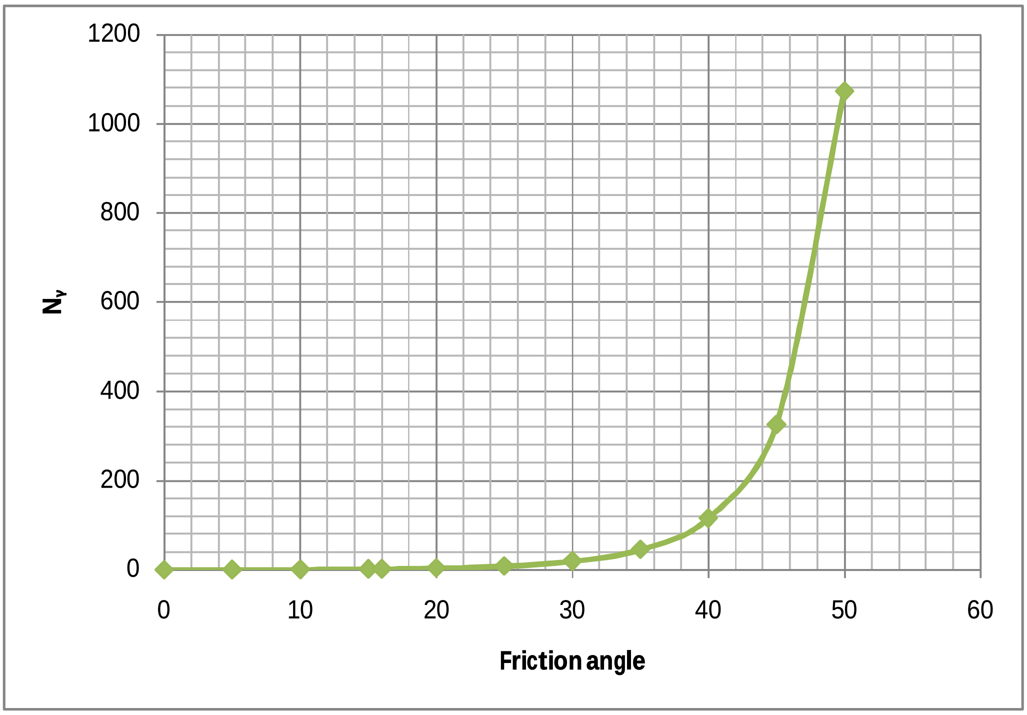


Figure (3.7)Terzaghi's Bearing Capacity Factor N_γ

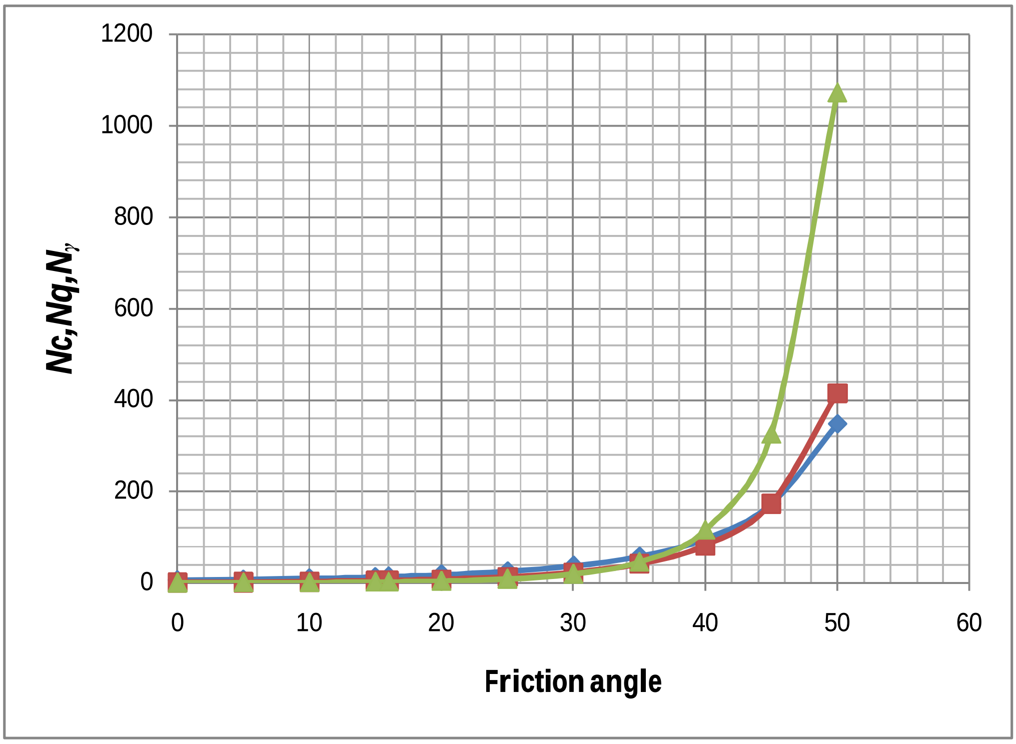


Figure (3.8) Terzaghi's Bearing Capacity Factors

To estimate the ultimate bearing capacity of square and circular foundations, next equations expressed that:

$$q_{u=}1.3cN_c + qN_q + 0.4\gamma BN_\gamma$$ **(Square foundation)** **equation (3.9)**

$$q_{u=}1.3cN_c + qN_q + 0.3\gamma BN_\gamma$$ **(Circular foundation)** **equation (3.10)**

For foundations that exhibit the local shear failure mode in soil, Terzaghi suggested the following modifications to Eqs (3.11), (3.12), and (3.13):

$$q_{u=} \frac{2}{3}c\grave{N}_c + q\grave{N}_q + \frac{1}{2}\gamma B\grave{N}_\gamma$$ **(Strip foundation)** **equation (3.11)**

$$q_{u=}0.867c\grave{N}_c + q\grave{N}_q + 0.4\gamma B\grave{N}_\gamma$$ **(Square foundation)** **equation (3.12)**

$$q_{u=}0.867c\grave{N}_c + q\grave{N}_q + 0.3\gamma B\grave{N}_\gamma$$ **(Circular foundation)** **equation (3.13)**

$\grave{N}_c$, $\grave{N}_q$, and $\grave{N}_\gamma$, the modified bearing capacity factors can be calculated by using Eqs (3.6), (3.7) and (3.8) by replacing $\emptyset$ by $\emptyset' = \tan^{-1}(\frac{2}{3}tan\emptyset)$.

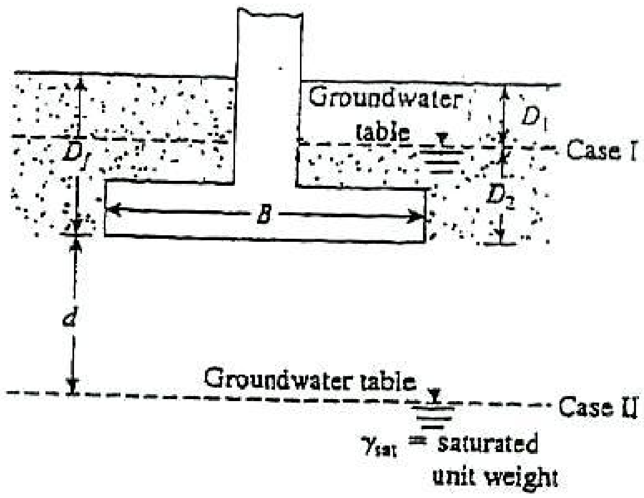
The variations of $\grave{N}_c$, $\grave{N}_q$ and $\grave{N}_\gamma$ with the soil frictions angle $\emptyset$ is given in table 3.5.

Table 3.6Terzaghi's modified Bearing Capacity Factors

$\emptyset'$	$\grave{N}_c$	$\grave{N}_q$	$\grave{N}_\gamma$	$\emptyset'$	$\grave{N}_c$	$\grave{N}_q$	$\grave{N}_\gamma$
0	5.70	1.00	0.00	26	15.53	6.05	2.59
1	5.90	1.07	0.005	27	16.30	6.54	2.88
2	6.10	1.14	0.02	28	17.13	7.07	3.29
3	6.30	1.22	0.04	29	18.03	7.66	3.76
4	5.51	1.30	0.055	30	18.99	8.31	4.39
5	6.74	1.39	0.074	31	20.03	9.03	9.03
6	6.97	1.49	0.10	32	21.16	9.82	5.51
7	7.22	1.59	0.128	33	22.39	10.69	6.32
8	7.47	1.70	0.16	34	23.72	11.67	7.22
9	7.74	1.82	0.20	35	25.18	12.75	8.35
10	8.02	1.94	0.24	36	26.77	13.97	9.41
11	8.32	2.08	0.30	37	28.51	15.32	10.90
12	8.63	2.22	0.35	38	30.43	16.85	12.72
13	8.96	2.38	0.42	39	32.53	18.56	14.71
14	9.31	2.55	0.48	40	34.87	20.50	17.22
15	9.67	2.73	0.57	41	37.45	22.70	19.75
16	10.06	2.92	0.67	42	40.33	25.21	22.50
17	10.47	3.13	0.76	43	43.54	28.06	26.25
18	10.90	3.36	0.88	44	47.13	31.34	30.40
19	11.36	3.61	1.03	45	51.17	35.11	36.00
20	11.85	3.88	1.12	46	55.73	39.48	41.70
21	12.37	4.17	1.35	47	60.91	44.45	49.30
22	12.92	4.48	1.55	48	66.80	50.46	59.25
23	13.51	4.82	1.74	49	73.55	57.41	71.45
24	14.14	5.20	1.97	50	81.31	65.60	85.75
25	14.80	5.60	2.25				

3.5.2 Modifications of bearing capacity equations of water table:-

Equations (3.4) and (3.9) through (3.13) give the ultimate bearing capacity, based on the assumption that the water table is located well below the foundation. However, if the water table is close to the foundations, some modifications of the bearing capacity equations:



(Figure 3.9) location of water table

Case I. if the water table is located so that $0 \leq D_1 \leq D_f$, the factor q in bearing capacity equations takes the form:

$$q = effective\ surcharge = D_1\gamma + D_2\ (\gamma_{sat} - \gamma_w\ ) \qquad \textbf{equation (3.14)}$$

Where:

γ_{sat} = saturated unit weight of soil

γ_w = unit weight of water

The factor γ in the last term of the equations has to be replaced by

$$\gamma' = \gamma_{sat} - \gamma_w$$

Case II. For a water table located so that $0 \leq d \leq B$,

$$q = \gamma D_f \qquad \textbf{equation (3.15)}$$

The factor γ in the last term must be replaced by the factor:

$$\bar{\gamma} = \grave{\gamma} + \frac{d}{B}(\gamma - \grave{\gamma})$$

Case III. When the water table is located so that d ≥ B, the water will have no effect on the ultimate Bering capacity. [4]

3.6 Meyerhof's bearing capacity theory:

The ultimate bearing capacity equations (Terzaghi,s equations) are for continuous, square, and circular foundations (0 < B/L < 1). Also, the equations do not take in to account the shearing resistance a long the failure surface in soil above the bottom of the foundation. In addition, the load on the foundation may be inclined.[4]

3.6.1 Meyerhof's bearing capacity equations:

To account for all these short coming above Meyerhof (1963) suggested the following from of the bearing capacity equation.

$$q_u = cN_cF_{cs}F_{cd}F_{ci} + qN_qF_{qs}F_{qd}F_{qi} + \frac{1}{2}\gamma BN_\gamma F_{\gamma s}F_{\gamma d}F_{\gamma i} \qquad \textbf{(3.16)}$$

Where:

C ≡ cohesion

q ≡ Effective stress at the level of the bottom of the foundation

γ≡ Unit weight of soil

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

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

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

F_{ci}, F_{qi},$F_{\gamma i}$≡ load inclination factors.

Bearing capacity factor:

$$N_q = tan^2(45 + \frac{\emptyset}{2})e^{\pi tan\emptyset}$$ **equation (3.17)**

$$N_c = (N_q - 1)cot\emptyset$$ **equation (3.18)**

$$N_\gamma = 2(N_q + 1)tan\emptyset$$ **equation(3.19)**

Shape Factors:

$$F_{cs} = (\frac{B}{L})(\frac{N_q}{N_c})$$ **equation (3.20)**

$$F_{qs} = 1 + \left(\frac{B}{L}\right)tan\emptyset$$ **equation (3.21)**

$$F_{\gamma s} = 1 - 0.4(\frac{B}{L})$$ **equation (3.22)**

Where:

L ≡ length of foundation (L > B).

Depth factors:

For $(D_f/B) \leq 1)$:

$$F_{cd} = 1 + 0.4(\frac{D_f}{B}) \qquad \textbf{equation (3.23)}$$

$$F_{qd} = 1 + 2tan\emptyset(1 - sin\emptyset)^2 \frac{D_f}{B} \qquad \textbf{equation (3.24)}$$

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

For $(D_f/B) > 1)$:

$$F_{cd} = 1 + (0.4)\tan^{-1}\left(\frac{D_f}{B}\right) \qquad \textbf{equation (3.25)}$$

$$F_{qd} = 1 + 2tan\emptyset(1 - sin\emptyset)^2 \tan^{-1}(\frac{D_f}{B}) \qquad \textbf{equation (3.26)}$$

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

The Factor $\tan^{-1}\left(\frac{D_f}{B}\right)$ is in radians in Eqs (3.25) and (3.26)

Inclination factors:

$$F_{ci} = F_{qi} = \left(1 - \frac{\beta}{90}\right)^2 \qquad \textbf{equation (3.27)}$$

$$F_{\gamma i} = (1 - \frac{\beta}{\emptyset})^2 \qquad \textbf{equation (3.28)}$$

Where:

$\beta \equiv$Inclination of the load on the foundation with .respect to the vertical.

Table 3.7 Bearing Capacity Factors

$\emptyset'$	$\grave{N}_c$	$\grave{N}_q$	$\grave{N}_\gamma$	$\emptyset'$	$\grave{N}_c$	$\grave{N}_q$	$\grave{N}_\gamma$
0	5.14	1.00	0.00	26	22.25	11.85	12.54
1	5.38	1.09	0.07	27	23.94	13.20	14.47
2	5.63	1.20	0.15	28	25.80	14.72	16.72
3	5.90	1.31	0.24	29	27.86	16.44	19.34
4	6.19	1.43	0.34	30	30.14	18.40	22.40
5	6.49	1.57	0.45	31	32.67	20.63	25.99
6	6.18	1.72	0.57	32	35.49	23.18	30.22
7	7.16	1.88	0.71	33	38.64	26.09	35.19
8	7.53	2.06	0.86	34	42.16	29.44	41.06
9	7.92	2.25	1.03	35	46.12	33.30	48.03
10	8.35	2.47	1.22	36	50.59	37.75	56.31
11	8.80	2.71	1.44	37	55.63	42.92	66.19
12	9.28	2.97	1.69	38	61.35	48.93	78.03
13	9.81	3.28	3.26	39	67.87	55.96	92.25
14	10.37	3.59	2.29	40	75.31	64.20	109.41
15	10.98	3.94	2.65	41	73.86	73.90	130.22
16	11.63	4.34	3.06	42	93.11	85.38	155.55
17	12.34	4.77	3.53	43	105.11	99.02	186.54
18	13.10	5.26	4.07	44	118.37	115.31	224.64
19	13.93	5.80	4.68	45	133.88	134.88	271.76
20	14.83	6.40	5.39	46	152.10	158.51	330.35
21	15.82	7.07	6.20	47	173.64	187.21	403.67
22	16.88	7.13	7.13	48	199.26	222.31	496.01
23	18.05	8.66	8.20	49	229.93	265.51	613.16
24	19.32	9.60	9.44	50	266.89	319.07	762.89
25	20.72	10.66	10.88				

3.6.2 Meyerhof's bearing capacity, shape, depth, and inclination factors: -

Table (3.7) is summary of those factors:

Table 3.8Meyerhof's Bearing Capacity, shape, depth, and inclination factors

Factors		Relationship
	Bearing capacity	
N_c		Equation (3.17)
N_q		Equation (3.18)
N_γ		(Nq-1) tan $(1.4\phi')$;see table(3.5)
	Shape	
For $\phi = 0$,		
Fcs		1 + 0.2 (B/L)
$F_{gs} = F_{\gamma s}$		1
For $\phi' \geq 10$,		
F_{cs}		1 +0.2 ((B/L) $\tan^2 (45 + \phi'/2)$
$F_{qs} = F_{\gamma s}$		1 +0.1 ((B/L) $\tan^2 (45 + \phi'/2)$
	Depth	
For $\phi = 0$,		
F_{cd}		1 + 0.2 (D_f/L)
$F_{qd} = F_{\gamma d}$		1
For $\phi \geq 10$,		1 +0.2 (D_f/B) tan $(45 + \phi'/2)$
F_{cd}		1 +0.1 (D_f/B) tan $(45 + \phi'/2)$
$F_{qd} = F_{\gamma d}$		1 + 0.2 (B/L)
	Inclination	
$F_{ci} = F_{qi}$		Equation (3.27)
$F_{\gamma i}$		Equation (3.28)

Table 3.9 Meyerhof's Bearing capacity factor $N_\gamma = (N_q - 1) \tan (1.4Ø)$

$\emptyset'$	N_γ	$\emptyset'$	N_γ	$\emptyset'$	N_γ	$\emptyset'$	N_γ
0	0.00	14	0.92	28	11.19	42	139.32
1	0.002	15	1.13	29	13.24	43	171.14
2	0.01	16	1.38	30	15.67	44	211.41
3	0.02	17	1.66	31	18.65	45	262.74
4	0.04	18	2.00	32	22.02	46	328.73
5	0.07	19	2.40	33	26.17	47	414.32
6	0.11	20	2.87	34	31.15	48	526.44
7	0.15	21	3.42	35	37.15	49	674.91
8	0.21	22	4.07	36	44.43	50	873.84
9	0.28	23	4.82	37	53.27	51	1143.93
10	0.37	24	5.72	38	64.07	52	1516.05
11	0.47	25	6.77	39	77.33	53	2037.26
12	0.60	26	8.00	40	93.69		
13	0.74	27	9.46	41	113.99		

Chapter (4)

MATERIALS AND TESTING

4.1 Introduction

This chapter contains structural fills in respect to it is laboratory works that used to check materials.

The laboratory works are Atterburg's limits, sieve analysis, compaction test, soil proficiency testing California bearing ratio (CBR), and shear box test.

4.2 Structural Fills

Structural fill is typically a screened earthen material used to create a strong, stable base. For example, the native soil at a site may be too weak to support a structure, so the native soil is replaced by compacted structural fill to provide the needed bearing capacity. Another common application is the filling of trenches and other excavations that will support roadways or other structures when completed. Structural fills are usually constructed by compacting earthen materials in place, so the compaction properties (optimum water content and maximum dry density) are very important to the performance.

The compressibility and shear strength are also important measures of the compacted material. Traditionally, fill materials have been composed of soil and natural aggregates.Structural fill used below foundations should consist of a mineral soil free of organic material, loam, debris, frozen soil or other deleterious material which may be compressible or which cannot be properly compacted.

Traditional structural fill materials tend to be sandy soils that compact well and have good drainage properties. There are a number of materials that have similar particle size gradations and mechanical properties, which them very good materials for structural fill applications. Foundry sands have performed well in the field, which is expected because the material is essentially high quality sand. Foundry sand has been used in a number of projects, and it has been found that design specifications for traditional materials will also apply foundry sands, which means no special equipment is required. Similarly, the use of coal fl y ash in embankments and fills is actually the second highest use of this material, with more than 7 million tons placed in 2006. It behaves like a fine sand material but has a lower density. Embankments and fills are also the highest use application of coal bottomash.

The use of crushed concrete in embankments and fill applications may not provide the best value, but crushed concrete makes a good embankment material and sometimes the specific situation provides the best use of the material. Tire shreds provide an excellent low density fill material, typically one-third the weight of gravel, and provide good drainage.

Structural fill should be placed in layers no thicker than 8 inches, as placed, and compacted with suitable compaction equipment to at least 95 percent of maximum dry density.

Common fill may be used below pavement and landscaped areas. Common fill should consist of granular soil free of organic material, topsoil, debris, frozen soil or other deleterious material that cannot be properly compacted. Common fill should contain stones no larger than 6inches and should have no more than 35 percent of material passing the No. 200 sieve. Common fill should be placed in layers not to exceed 12 inches, as placed, and compacted with suitable vibratory compaction equipment to at least 92 percent of maximum dry density. [5]

Compacted soil

Compacted soil reduces the rate of water distribution through the soil to plant roots. In addition, compacted soil reduces air space available for plant roots and will therefore stunt root growth. And if poor drainage from an excess of water being held in the soil and poor aeration aren't enough to convince you of the undesirability of compacted soil, increased erosion will make you think again, as the water landing on the surface runs off elsewhere because the hard surface on the soil prevents penetration of the water. This article is a basic primer in getting started on ridding your garden or land of compacted soil. [5]

Picture (4.1) structural fill samples

4.3 Laboratory Tests

4.3.1 Atterburg's Limits

Liquid Limit by the Use of Fall Cone Penetrometer

Figure 4.1 shows the arrangement of the apparatus. The soil whose liquid limit is to be determined is mixed well into a soft consistency and pressed into the cylindrical mold of 5 cm diameter and 5 cm high. The cone which has a central angle of 31° and a total mass of 148 g will be kept free on the surface of the soil.

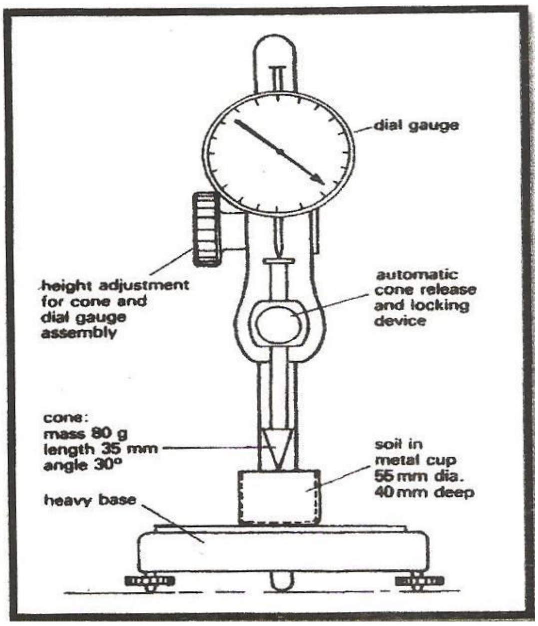


Figure (4.1) the cone penometer devices picture (4.2)(cone penometeter)

The depth of penetration y of the cone is measured in mm on the graduated scale after 30 sec of penetration. The liquid limit w_l may be computed by using the formula:

$$w_l = w_y + 0.01(25 - y)(w_y + 15) \qquad \text{equation (4.1)}$$

Where:

$w_y \equiv$ is the water content corresponding to the penetration y.

The procedure is based on the assumption that the penetration lies between 20 and 30 mm. [1]

Plastic Limit

About 15 g of soil, passing through a No. 40 sieve, is mixed thoroughly. The soil is rolled on a glass plate with the hand, until it is about 3 mm in diameter. This procedure of mixing and rolling is repeated till the soil shows signs of crumbling. The water content of the crumbled portion of the thread is determined. This is called the plastic limit. [1]

Plasticity Index *I_p*

Plasticity index *I_p* indicates the degree of plasticity of a soil. The greater the difference between liquid and plastic limits, the greater is the plasticity of the soil. Cohesion less soil has zero plasticity indexes. Such soils are termed non-plastic. Fat clays are highly plastic and possess a high plasticity index. Soils possessing large values of w, and / are said to be highly plastic or fat. Those with low values are described as slightly plastic or lean. Atterberg classifies the soils according to their plasticity indices as in Table 4.1. [1]

Table (4.1) soil classifications according to plasticity index

Plasticity index	plasticity
0	Non-plastic
<7	Low plastic
7-17	Medium plastic
>17	Highly plastic

4.3.1.1 Hattab (1) sample: penetration and moisture content four trails is dotted ,an intermediate line is drawn between the dotes , liquid limit is the moisture content causes a penetration of 20 mm, as shown in appendix (A) calculation result was as shown below:

The sample is founded to be Medium plastic.

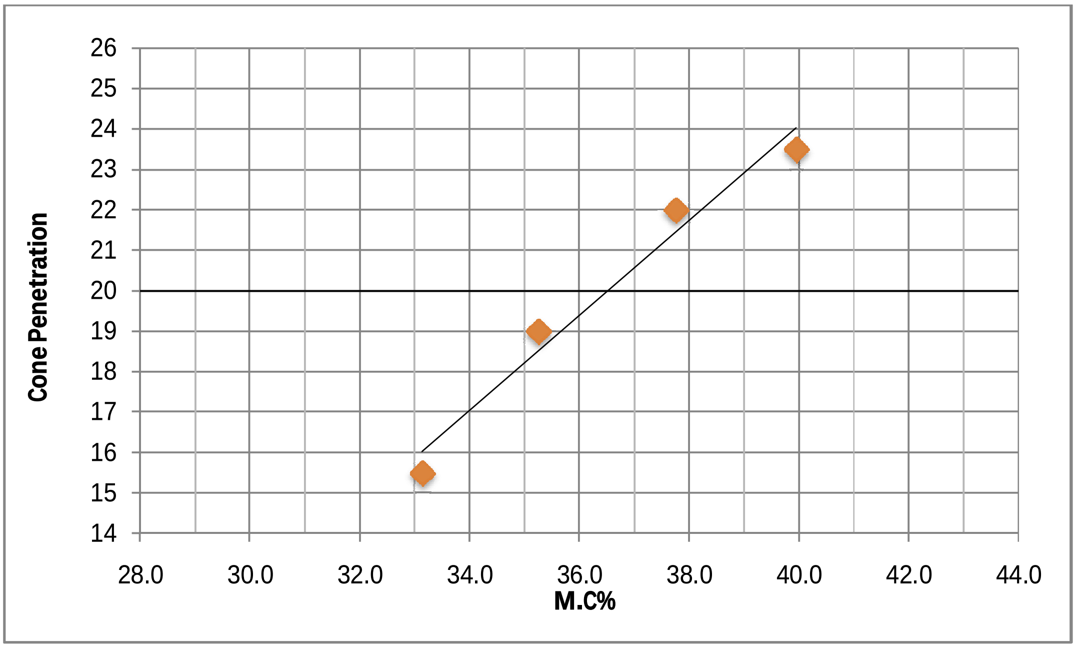


Figure (4.2) liquid limit test for hattab(1) sample

Liquid Limit (L.L)	**37.0**
Plastic Limit(P.L)	25
Plasticity Index(P.I)	12

4.3.1.2 Hattab (2) sample: as shown in appendix (A) calculation result was as shown below:

The sample is founded to be Medium plastic.

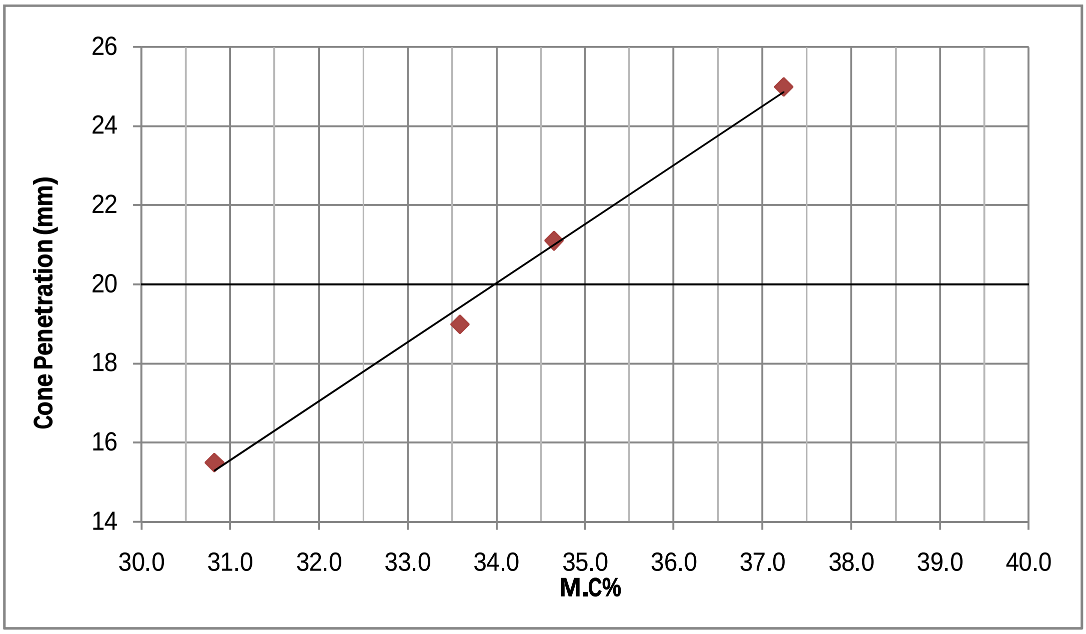


Figure (4.3) liquid limit test for hattab (2) sample

Liquid Limit (L.L)	**37.0**
Plastic Limit(P.L)	19
Plasticity Index(P.I)	18

4.3.1.3 El housh sample: as shown in appendix (A) calculation result was as shown below:

The sample is founded to be Medium plastic.

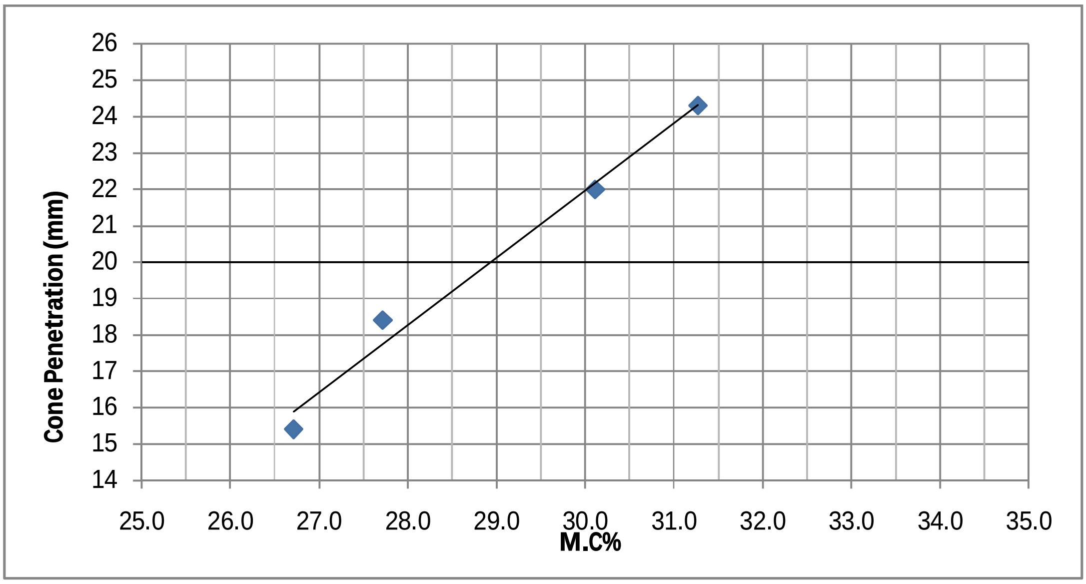


Figure (4.4) liquid limit test for El housh sample

Liquid Limit (L.L)	**37.0**
Plastic Limit(P.L)	16
Plasticity Index(P.I)	21

4.3.1.4 Jabil Toria sample: as shown in appendix (A) calculation result was as shown below:

The sample is founded to be Medium plastic.

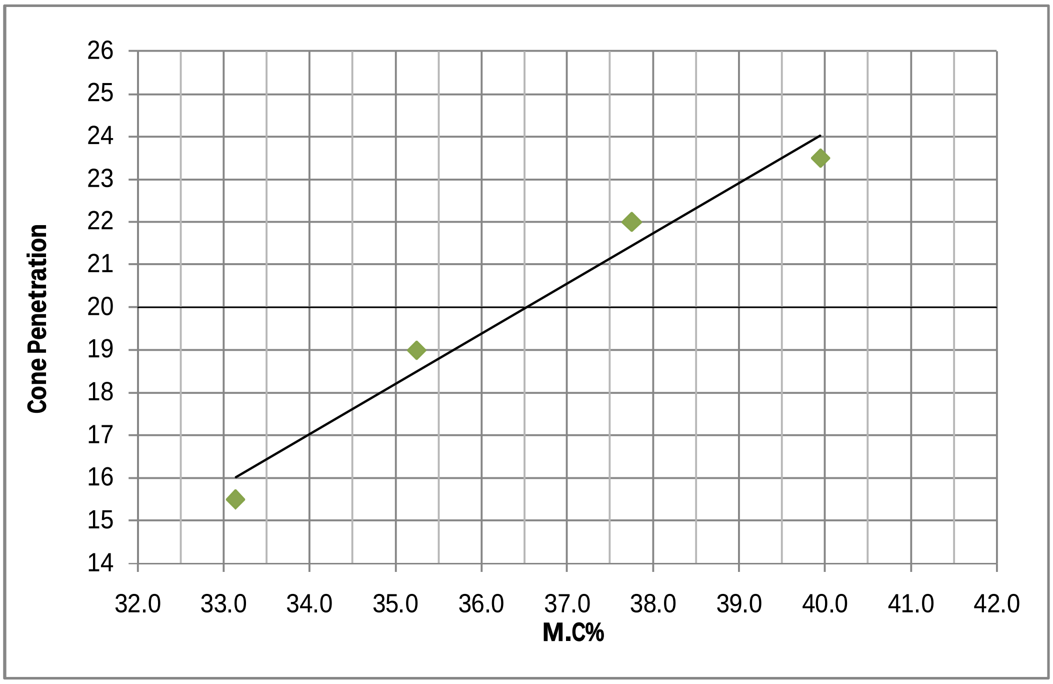
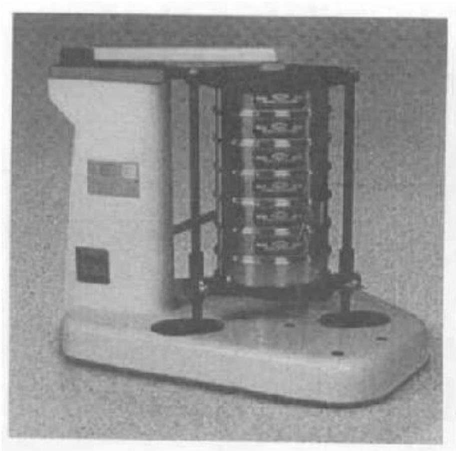


Figure (4.5) liquid limit test for Jabil Toria sample

Liquid Limit (L.L)	**37.0**
Plastic Limit(P.L)	21
Plasticity Index(P.I)	16

4.3.2Sieve Analysis

The sieve analysis is carried out by sieving a known dry mass of sample through the nest of sieves placed one below the other so that the openings decrease in size from the top sieve downwards, with a pan at the bottom of the stack as shown in picture4.3. The whole nest of sieves is given a horizontal shaking for about 10 minutes (if required, more) till the mass of soil remaining on each sieve reaches a constant value (the shaking can be done by hand or using a mechanical shaker, if available). The amount of shaking required depends on the shape and number of particles. If a sizable portion of soil is retained on the No. 200 sieve, it should be washed. This is done by placing the sieve with a pan at the bottom and pouring clean water on the screen. A spoon may be used to stir the slurry. The soil which is washed through is recovered, dried and weighed .The mass of soil recovered is subtracted from the mass retained on the No. 200 sieve before washing and added to the soil that has passed through the No. 200 sieve by dry sieving. The mass of soil required for sieve analysis is of oven-dried soil with all the particles separated out by some means. [1]



Picture (4.3) the nest of sieve figure (4.6) the vibration machine of sieves

4.3.2.1 Hattab (1) sample: As shown in appendix (B) the test results show that the sample is clayey gravel of low plasticity.

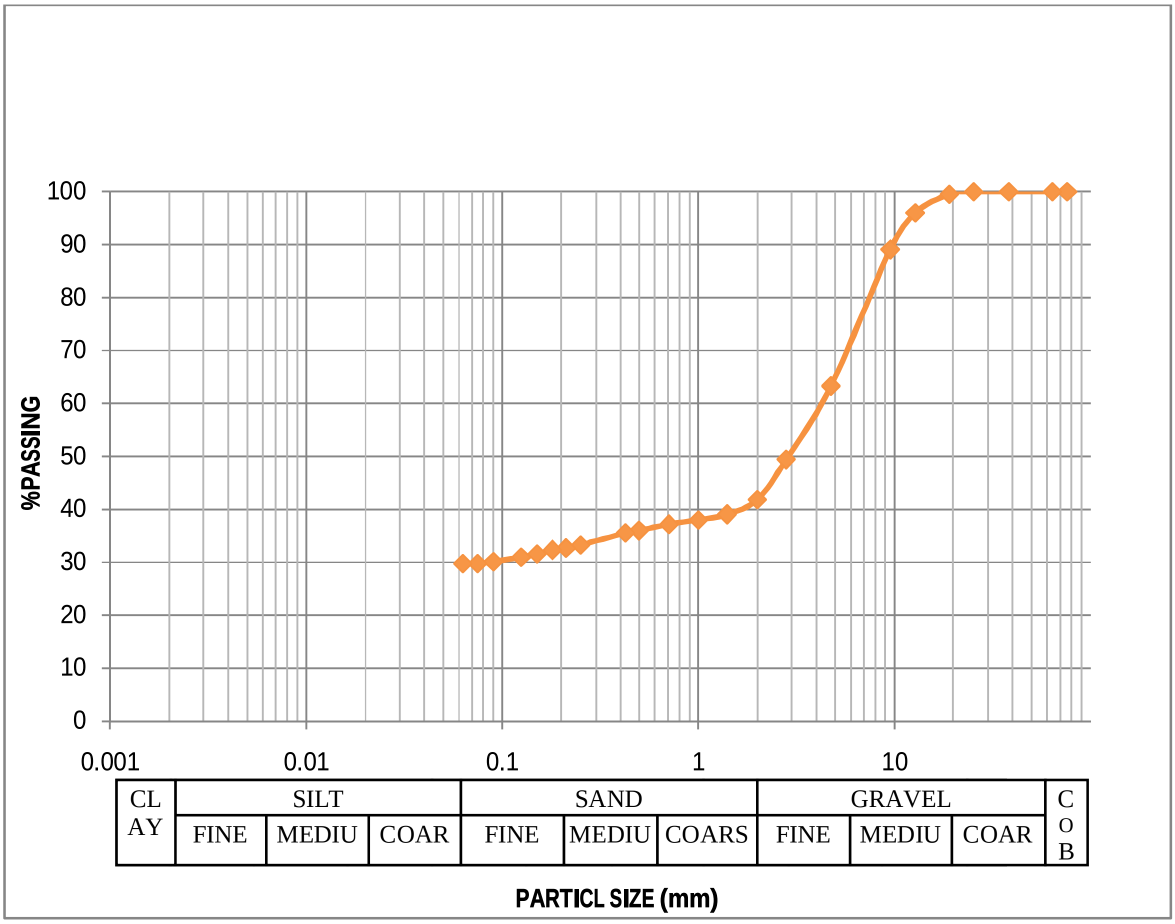


Figure (4.7) Sieve Analysis test for hattab (1) sample

4.3.2.2Hattab (2) sample: As shown in appendix(B) the test results show that the sample is clayey gravel of low plasticity.

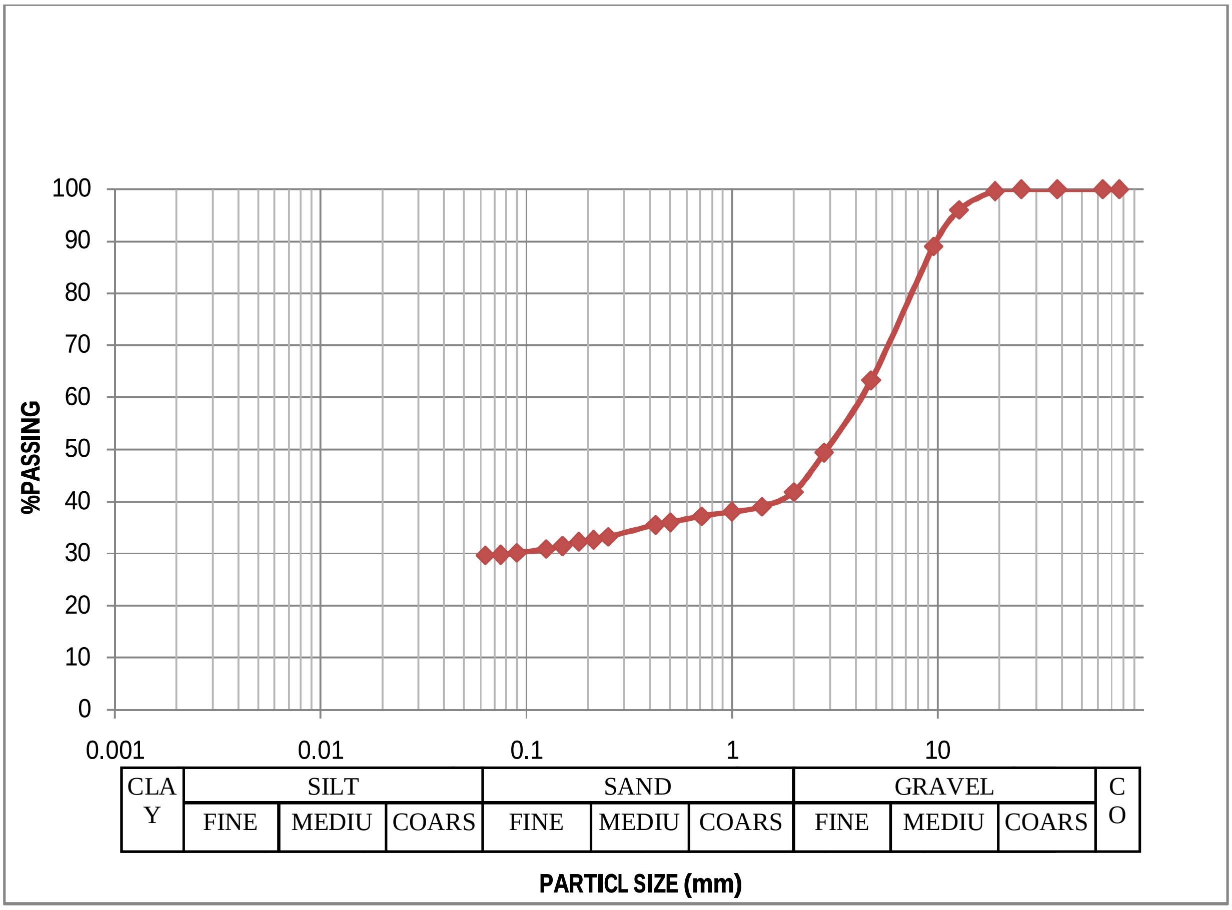


Figure (4.8) Sieve Analysis test for hattab (2) sample

4.3.2.3 El houshsample: As shown in appendix(B) the test results show that the sample is clayey gravel of low plasticity.

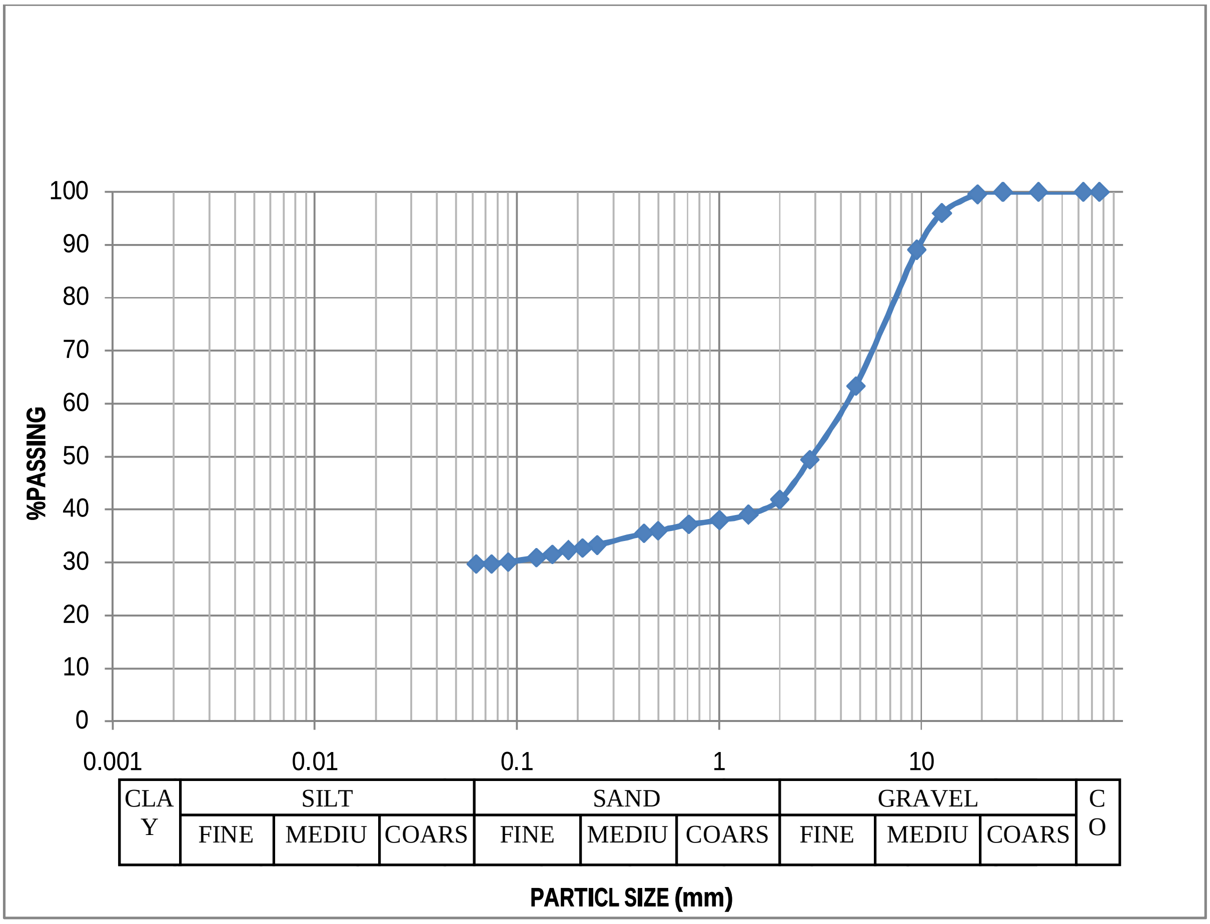


Figure (4.9) Sieve Analysis test for El houshsample

4.3.2.4 Jabil Toriasample: As shown in appendix(B) the test results show that the sample is clayey gravel of low plasticity.

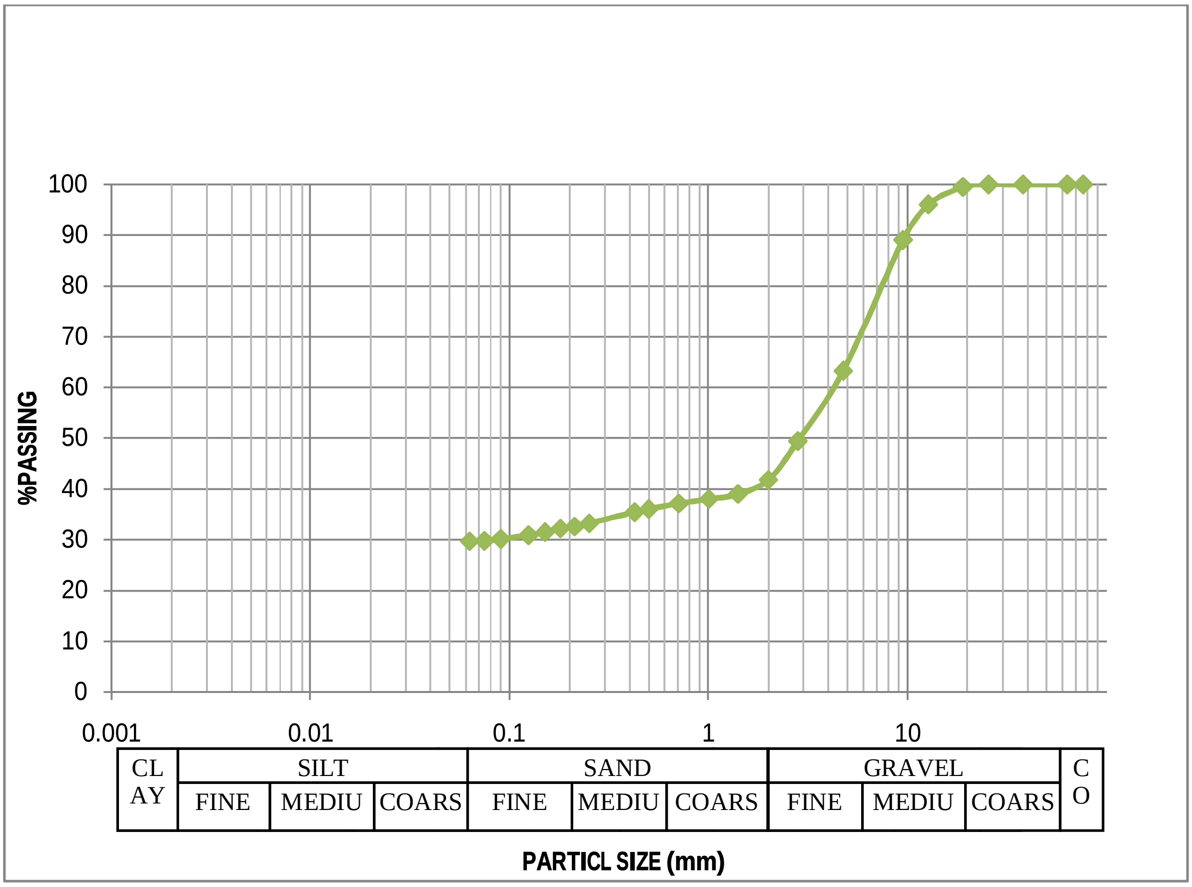


Figure (4.10) Sieve Analysis test for Jabil Toriasample

4.3.3 Compaction Test

Soil compaction is defined as the method of mechanically increasing the density of soil. In construction, this is a significant part of the building process. If performed improperly, settlement of the soil could occur and result in unnecessary maintenance costs or structure failure.

Almost all types of building sites and construction projects utilize mechanical compaction techniques.

There are five principle reasons to compact soil:

1-Increases load-bearing capacity.

2-Prevents soil settlement and frost damage.

3-Provides stability.

4-Reduces water seepage, swelling and contraction.

5-Reduces settling of soil.

There are four types of compaction effort on soil or asphalt:

1-Vibration

2-Impact

3-Kneading

4-Pressure

To determine if proper soil compaction is achieved for any specific construction application, several methods were developed. The most prominent by far is soil density. Soil testing accomplishes the following:

1-Measures density of soil for comparing the degree of compaction with specs.

2-Measures the effect of moisture on soil density with specs.

3-Provides a moisture density curve identifying optimum moisture.

Tests to determine optimum moisture content are done in the laboratory. The most common is the Proctor Test, or Modified Proctor Test. A particular soil needs to have an ideal (or optimum) amount of moisture to achieve maximum density. This is important not only for durability, but will save money because less compaction effort is needed to achieve the desired results.[1]

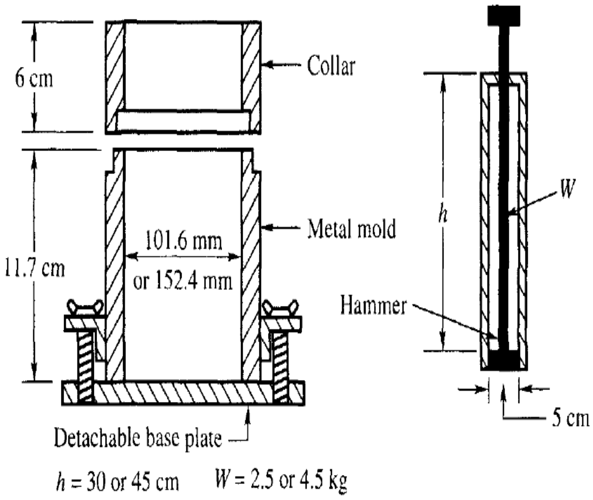
Standard Proctor Compaction Test

Proctor (1933) developed this test in connection with the construction of earth fill dams in California. The standard size of the apparatus used for the test is given in Fig 4.10.

Test Procedure

A soil at selected water content is placed in layers into a mold of given dimensions (Fig. 4.3), with each layer compacted by 25 or 56 blows of a 5.5 Ib (2.5 kg) hammer dropped from a height of 12 in (305 mm), subjecting the soil to a total compactive effort of about 12,375 fl-lb/ft3 (600 kNm/m3). The resulting dry unit weight is determined.

The procedure is repeated for a sufficient number of water contents to establish a relationship between the dry unit weight and the water content of the soil. This data, when, plotted, represents a curvilinear relationship known as the compaction curve or moisture-density curve. The values of water content and standard maximum dry unit weight are determined from the compaction curve.[1]



Picture (4.4)mould and hammerFigure (4.11)compaction test devices

4.3.3.1 Hattab (1) sample: As shown in appendix (C) calculation results was as shown below:

- Optimum Moisture Content (O.M.C) = 6.40%

- Maximum Dry Density (M.D.D) = 2.223gm/cm^3

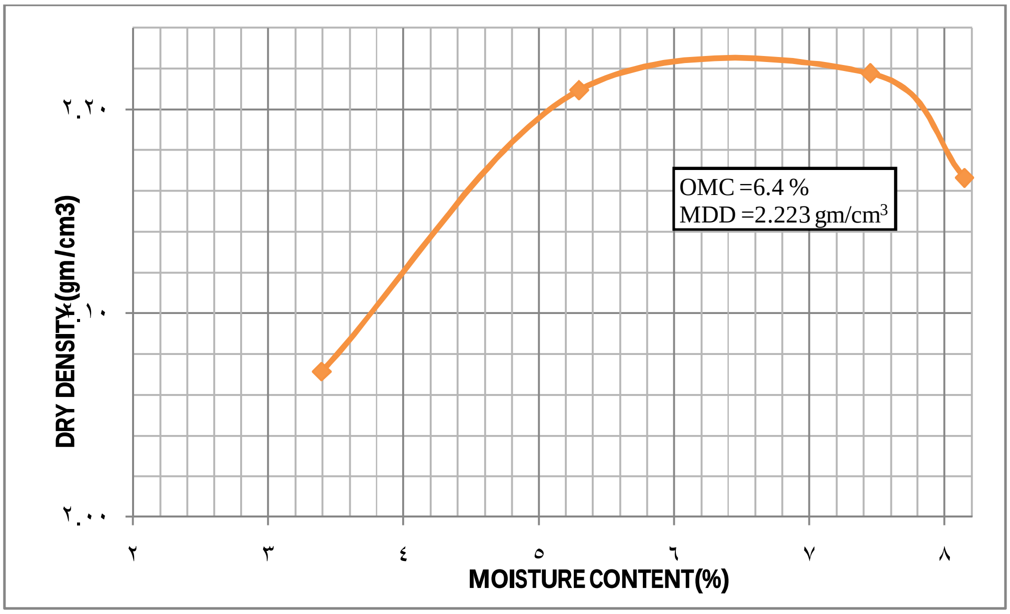


Figure (4.12) compaction test for Hattab (1) sample

4.3.3.2Hattab (2) sample: As shown in appendix (C) calculation results was as shown below:

- Optimum Moisture Content (O.M.C) = 7.20%

- Maximum Dry Density (M.D.D) = 2.18gm/cm^3

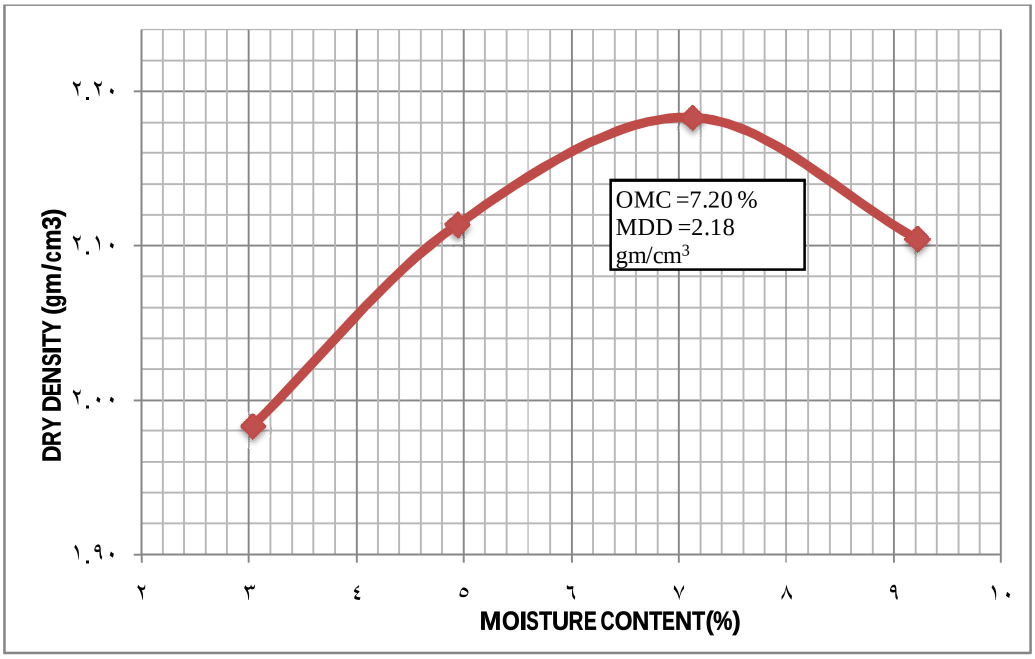


Figure (4.13) compaction test for Hattab (2) sample

4.3.3.3 El houshsample: As shown in appendix (C) calculation results was as shown below:

- Optimum Moisture Content (O.M.C) = 7.03%

- Maximum Dry Density (M.D.D) = 2.18gm/cm^3

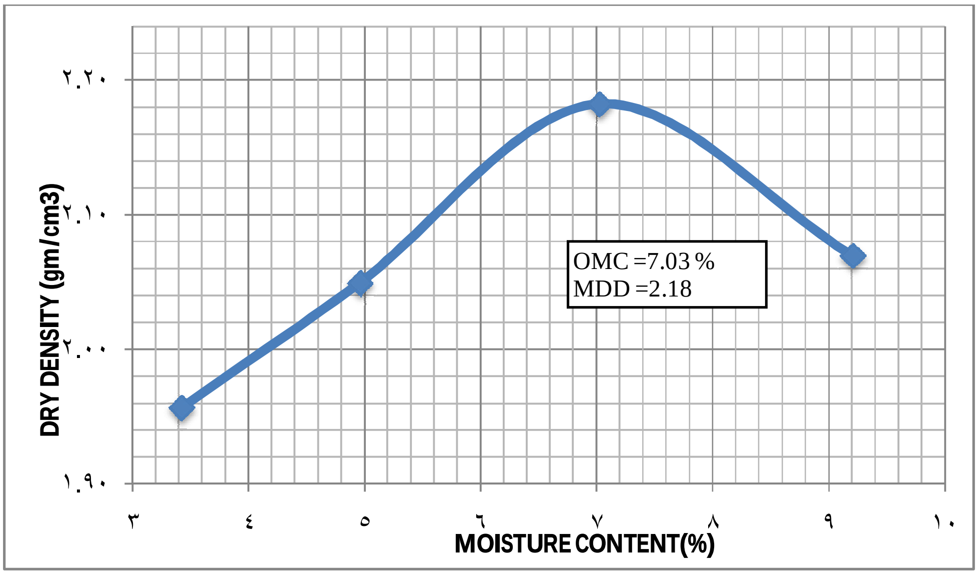


Figure (4.14) compaction test for El housh sample

4.3.3.4 Jabil Toria sample: As shown in appendix (C) calculation results were as shown below:

- Optimum Moisture Content (O.M.C) = 6.60%

- Maximum Dry Density (M.D.D) = 2.12gm/cm^3

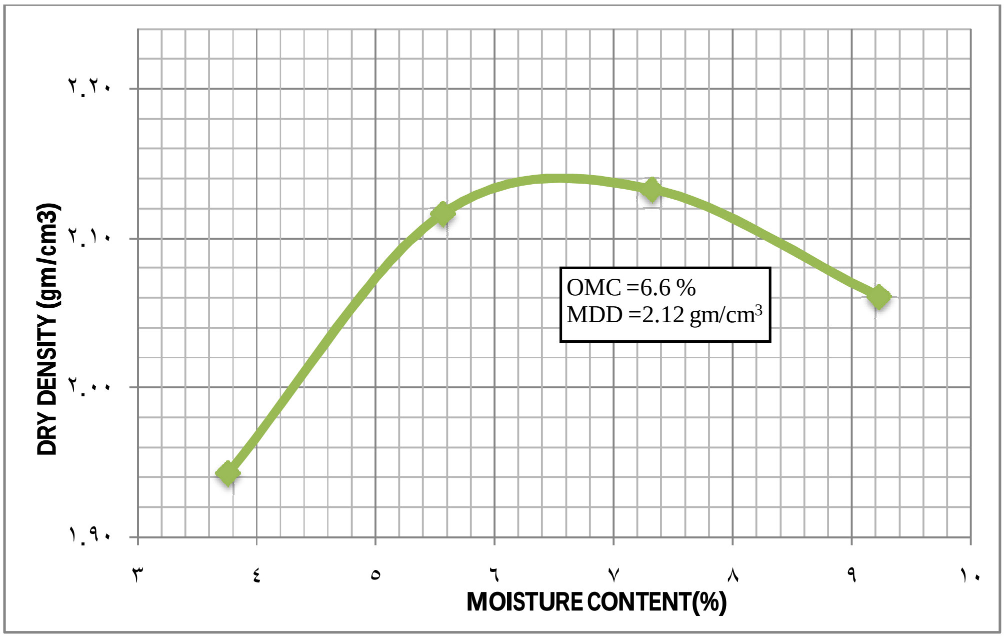


Figure (4.15) compaction test for Jabil Toriasample

4.3.4 SOILS Proficiency Testing California Bearing Ratio (CBR)

The California Bearing Ratio (CBR) is a penetration test for evaluation of the mechanical strength of road sub grades and base courses. It was developed by the California Department of transportation in the 1930's. The CBR is used for measuring the load-bearing capacity for new pathways, road and airstrips or for soils already under paved areas. [5]

CBR

The California bearing ratio test, usually shortened to CBR, is a penetration test where in a standardized piston, having an end area of 3 square inches, is caused to penetrate the soil at a standard area of 0.05 inch per minute. The unit load at each 1/10 inch penetration up to o.5 inch is recorded and the CBR computed as the ratio of an arbitrarily selected unit load to that of a standard. The unit load generally taken for design is at 0.1 inch penetration; however in some cases other penetration values are used. As a general rule, the CBR will decrease as the penetration value increases. In some cases, however, the value at 0.2 inch penetration may be higher than that at the 0.1 inch penetration. If this happens the value at 0.2 inch penetration is used.[7]

CBR is a measure of resistance of material to penetration of a plunger under controlled density and moisture conditions. A standard piston is used to penetrate the soil at a standard rate. The pressure up to a penetration of 10mm and its ratio to the bearing value of a standard crushed rock is termed as the CBR.[5]

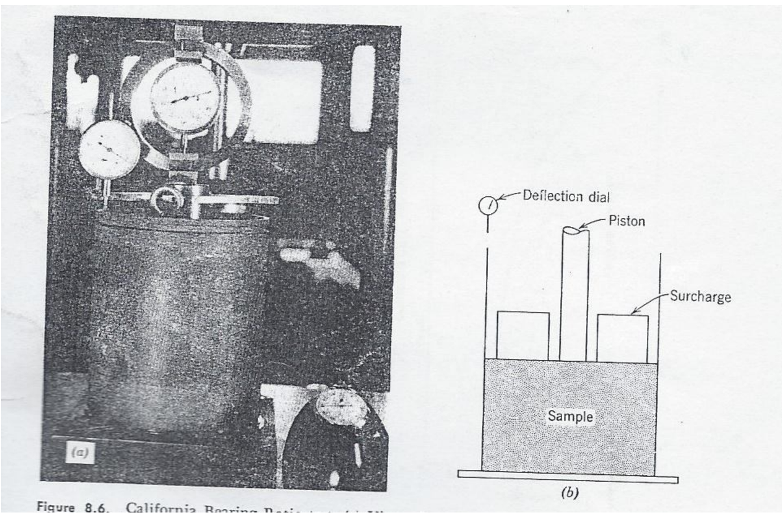


Figure (4.16) California bearing ratio test.(a) View of cylinder and dials; (b) schematic diagram.

Test Procedure

-The soil is sieved over a 19mm sieve and the percentage of sample remaining on the sieve is calculated. This will determine the size of mould to be used in the CBR test. The material passing the 19mm sieve is split up for determining maximum dried density, optimum moisture content and CBR.

-The specimen is mixed with enough water to dampen it to achieve the required laboratory moisture ratio. It is then left to cure for as long as it takes for the water to be thoroughly mixed into and uniformly distributed.

-Determine mass of mould and soil. Place in oven and dry to a constant mass, reweigh. Calculate maximum dried density and optimum moisture content.

-The specimen is mixed with enough water to dampen it to achieve the required laboratory moisture ratio. It is then left to cure for as long as it takes for the water to be thoroughly mixed into and uniformly distributed.

-Compact into mould using modified compaction.

-Load is applied on the sample by a standard plunger at the rate of 1 ±0.2 mm/min. [5]

Results

-Plot the load penetration curve.

-Read the force value in N at penetrations of 2.5mm and 5.0mm and calculate the bearing ratio for each by dividing by 13.2kN and 19.8kN respectively, t hen multiplying by 100.

-CBR value is expressed as a percentage of the actual load causing the penetrations of 2.5 mm or 5.0 mm to the standard loads mentioned above. The greatest value calculated for penetrations at 2.5 mm and 5.0mm will be recorded as the CBR. When the graph begins to concave up this is likely due to surface irregularities and so the zero point must be adjusted. [5]

Problems faced in Proficiency Testing

-Human error in calculations may play a small role in the problems faced in PT programs, however, with modern technology it is unlikely that calculations are performed free hand but rather through an excel spreadsheet or program.

-It seems the most common problem simply comes down to segregation and the handling of samples.

-Strength test needs a nice smooth distribution of particle sizes.

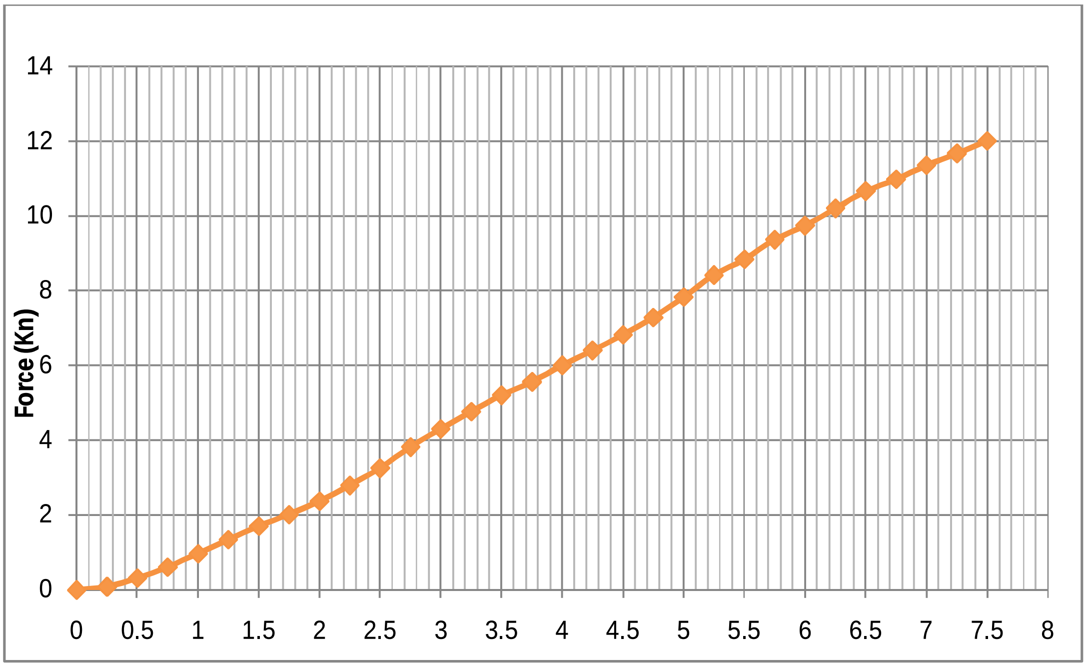
-Plunger may only contact larger particles and thus give a higher strength that is not representative of the sample.

When performing CBR must ensure there is a good mix of the sample before compaction. [5]

4.3.3.1 Hattab (1) sample: As shown in appendix (D) calculation results by B.S method was as shown below:

CBR @ 2.5 mm = 33%

CBR @ 5 mm = 44%



Penetration (mm)

Figure (4.17) CBR test for Hattab (1) sample

4.3.3.2 Hattab (2) sample: As shown in appendix (D) calculation results by B.S method was as shown below:

CBR @ 2.5 mm = 35%

CBR @ 5 mm = 37%

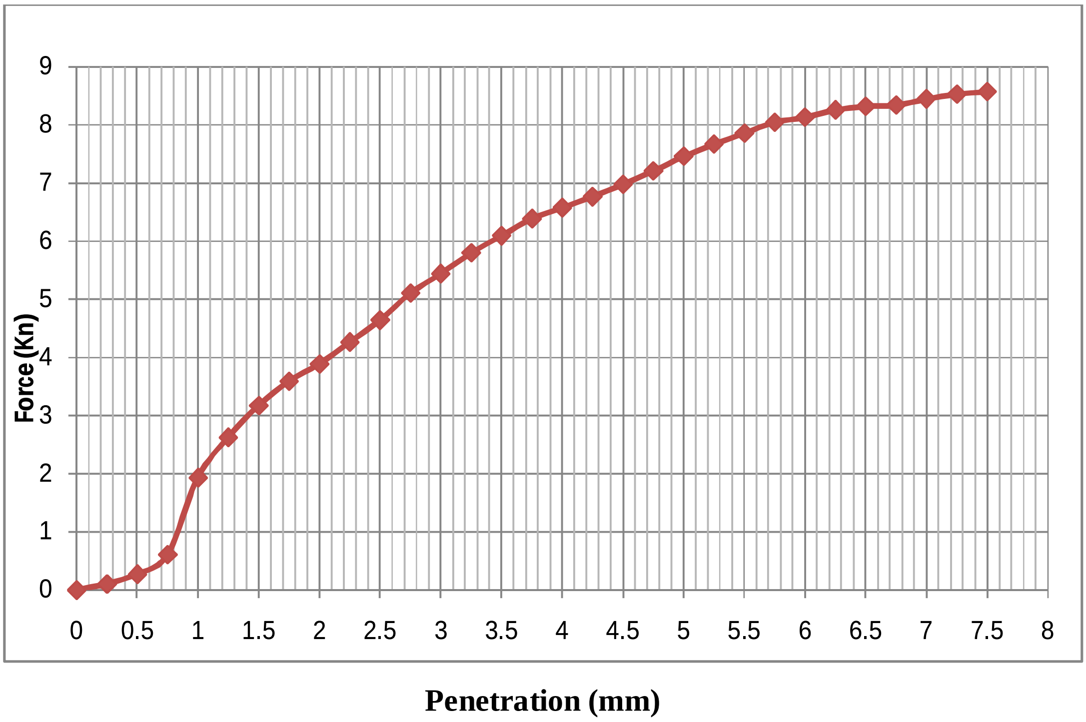


Figure (4.18) CBR test for Hattab (2) sample

4.3.3.3 El housh sample: As shown in appendix (D) calculation results by B.S method was as shown below:

CBR @ 2.5 mm = 14%

CBR @ 5 mm = 14%

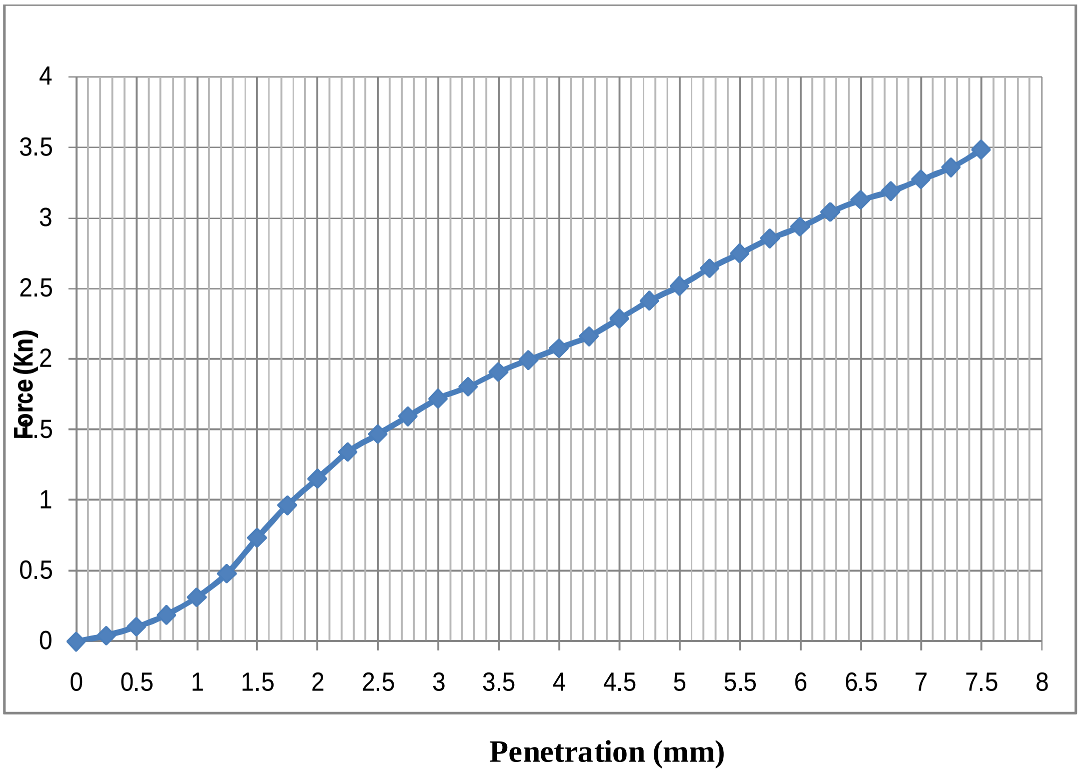


Figure (4.19) CBR test for El housh sample

4.3.5 Direct Shear Test

The direct shear test (Figure 4.21) is the earliest method for testing soil shearingstrength. The Box Shear apparatus consists of a rectangular box with a top that canslide over the bottom half. Normal load is applied vertically at the top of the boxas shown in Figure (4.21).

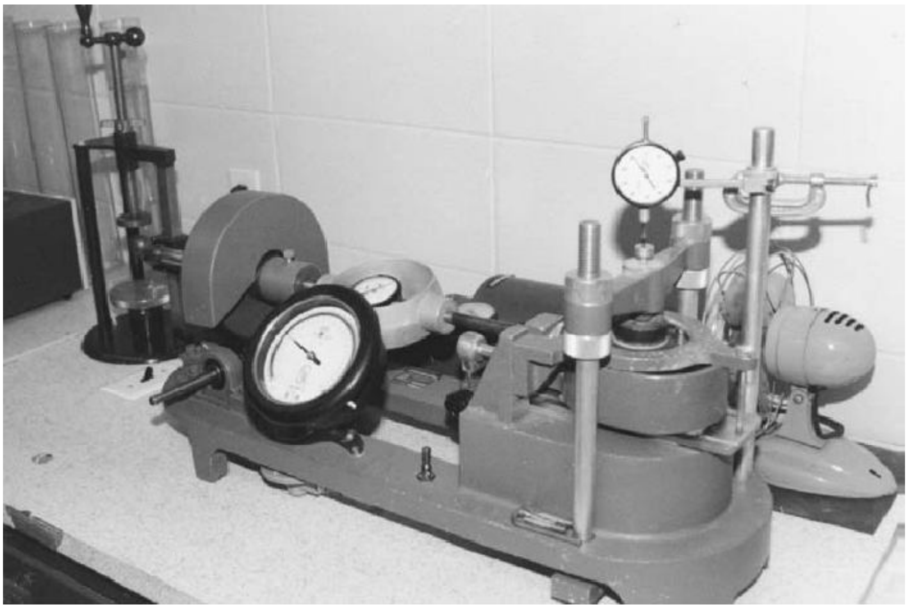


Figure (4.21) direct shear test

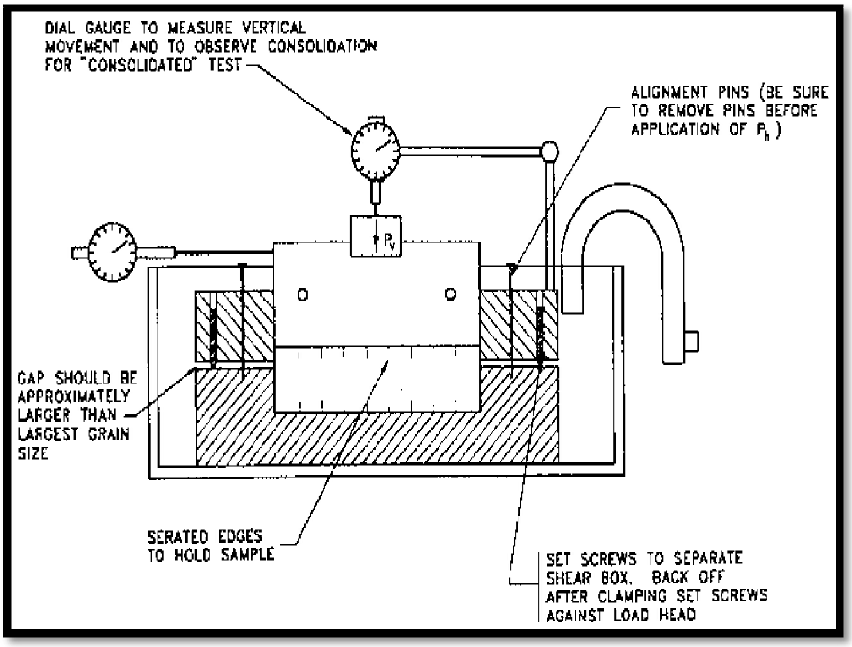


Figure (4.22) direct shear apparatus

A shearing force is applied to the top half of the box, shearing the sample along the horizontal surface, and the shear stress that produces the shear failure is recorded.The operation is repeated several times under different normal loads. The resultingvalues of shearing strength against normal loads are plotted and the angle of internal friction and cohesion value determined. The commonly used sample size for the direct shear test is 4 in* 4 in. Such an undisturbed sample can be obtained by the use of a 6-in. Shelby tube. There is an unequal distribution of stresses over the shear surface. The stress is greater at the edges and less at the center. The strength indicated by the test will often be too low.

Irrespective of the many shortcomings of the direct shear test, its simplicity led to wide adoption of the test by most consulting engineers' laboratories.[9]

Some typical values of soil cohesion are given below for different soil types. The soil cohesion depends strongly on the consistence, packing, and saturation condition. The values given below correspond to normally consolidated condition unless otherwise stated. These values should be used only as guideline for geotechnical problems.[5]

For Inorganic clays soil, silty clay soil, sandy clays of low plasticity, cohesion value = 4[kPa], from Swiss Standard SN 670 010b, Characteristic Coefficients of soils, Association of Swiss Road and Traffic Engineers (reference).[12]

For Inorganic clays soil, silty clay soil, sandy clays of low plasticity, Soil friction angle [°], min=27, and max = 35, from Swiss Standard SN 670 010b, Characteristic Coefficients of soils, Association of Swiss Road and Traffic Engineers (reference).So, the values in this reference are suggested for use if further information is not available, then :

C (cohesion) = 4 [KPa].

Φ (internal friction angle) = 31[°] (average).

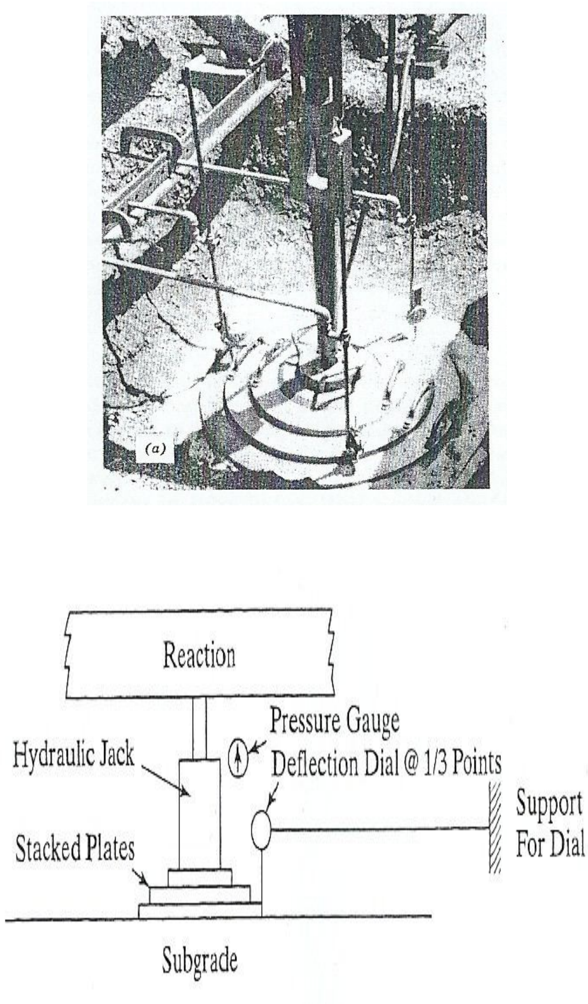
Chapter (5)

PLATE LOAD TEST

5.1 Introduction

Plate load test is a field test for determining the ultimate bearing capacity of soil and the likely settlement under a given load. The plate load test basically consists of loading a steel plate placed at the foundation level and recording the settlements corresponding to each load increment. The test load is gradually increased till the plate starts to sink at a rapid rate. The total value of load on the plate in such a stage divided by suitable factor of safety (which varies from 2 to 3)to arrive at the value of safe bearing capacity of soil**.** [2]

The plate-loading test is one made to evaluate the supporting power of subgrades, bases, and. in some cases, completed pavements by utilizing relatively large diameter plates. Circular plates are used. The reaction for the load is supplied by heavy mobile equipment tied to a steel beam. The load is applied to the plates by means of hydraulic jacks. Deflection of the plate is measured by means of deflection dials placed usually at the one-third points of the plate near its outer edge. To minimize bending, a series of stacked plates should be used. [7]



(b)

Figure (5.1) plate-bearing test.(a) View of plates and dials;(b) schematic diagram

Figure (5.2) plate load test

5.2 Test setup

A test pit is dug at site up to the depth at which the foundation is proposed to be laid. The width of the pit should be at least5 times the width of the test plate. At the center of the pit a small square depression or hole is made whose size is equal to the size of the test plate and bottom level of which corresponds to the level of actual foundation. The depth of the hole should be such that the ratio of depth to width of the loaded area is approximately the same as the ratio of the actual depth to width of the foundation.

The mild steel plate (also known as bearing plate) used in the test should not be less than 25 mm in thickness and it is size may vary from 300 to 700 mm. The plate could be square or circular in shape. Circular plate is adopted in case of circular footing and square plate is used in all other types of footings. The plate is machined on side and edges. [2]

5.3 Testing procedure

The load is applied to the test plate through a centrally placed column. The test load is transmitted to the column by one of the following two methods:

5.3.1 Gravity loading or reaction loading method

In case of gravity loading method, a loading platform is constructed over the column placed on the test plate and test load is applied by placing dead weight in the form of sand bags, pig iron, concrete blocks, lead bars, etc. many hydraulic jack is placed between the loading platform and the column top for applying the load to the test plate, the reaction of the hydraulic jack being borne by the loaded platform. This form of loading is termed as reaction loading. [2]

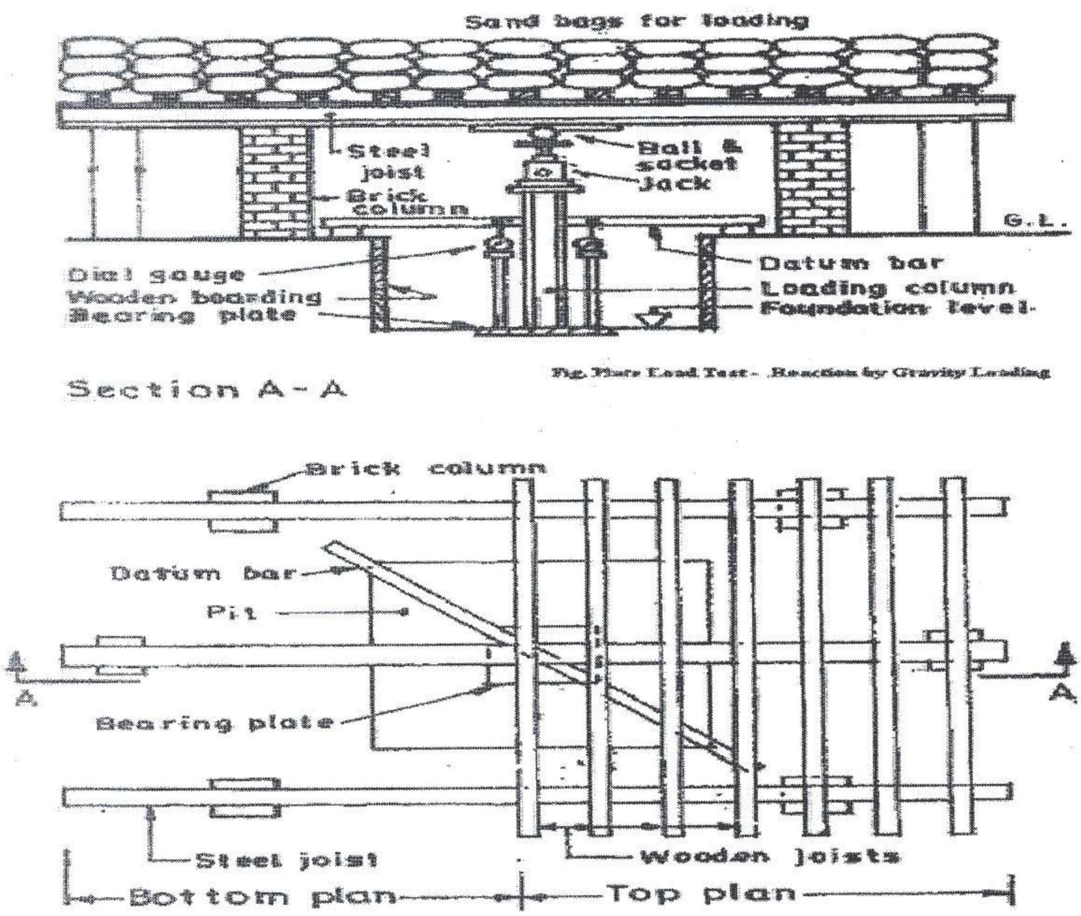


Figure (5.3) plate load test-reaction by gravity loading

5.3.2 Reaction truss method

In case of reaction truss method, instead of constructing a loading platform, a steel truss of suitable size is provided to bear the reaction of the hydraulic jack. The truss is firmly anchored to the ground by means of steel anchors and guy ropes are provided for ensuring it is lateral stability. When the load is applied to the test plate, it starts sinking slowly. The settlement of the plate is recorded to an accuracy of 0.02 mm with the help of sensitive dial gauges. At least two dial gauges are placed at diametrically opposite ends of the plate and one dial guage is mounted on independently supported references beam or datum rod. As the plate sinks,the ram of the dial guage moves down and the settlement is recorded. The magnitude of load is applied in regular increment of about 2kn or $1/5^{th}$ of the expected ultimate bearing capacity, whichever is less.Settlment should be observed for each increment of load after an interval of 1, 4,10,20,40 and 60 minutes and thereafter at hourly intervals until the rate of settlement becomes less than 0.02 mm per hour. The maximum load to be applied for the test should be about 15 times the expected ultimate bearing capacity of the soil. [2]

In case of clayey soils the, time settlement curve should be plotted at each load stage and load should be increased to next stage either when the curve indicates that the settlement has exceeded 70 to 80% of the probable ultimate settlement at that stage or at the end of 24 hour period. [2]

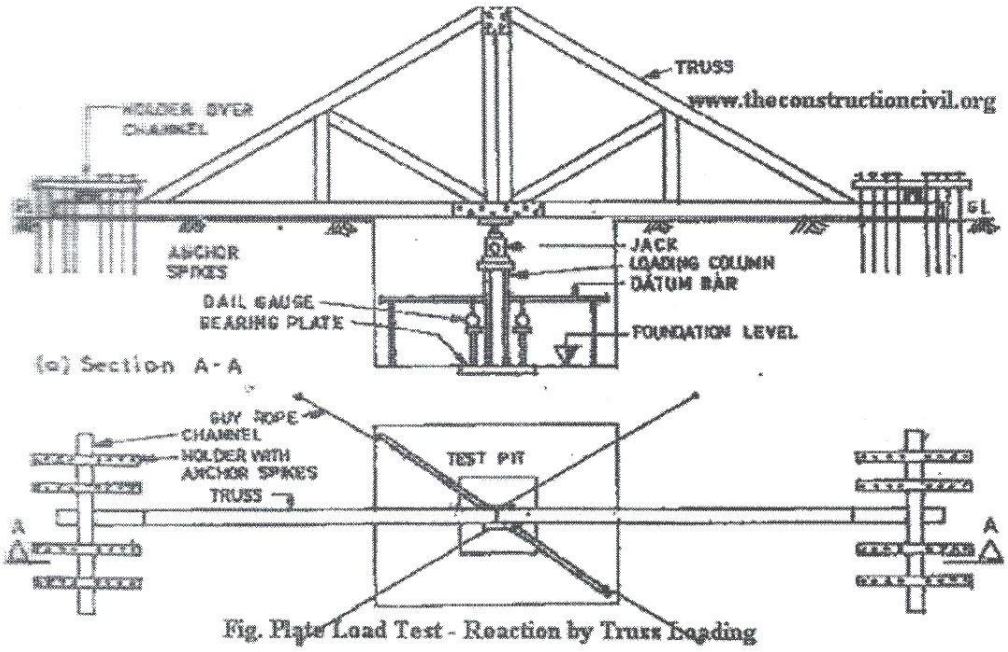


Figure (5.4) plate load test-Reaction by truss

For soils other than clayey soils, each load increment should be kept for not less than one hour or up to a time when the rate of settlement gets appreciably reduced (to a value of 0.02 mm per mm). The next increment of load should then be applied and observations repeated. The test is continued till a settlement of 25mm under normal circumstances or 50 mm in special cases (such as dense gravel, and sand mixture).[2]

The following abbreviation to procedures of the test:

1. The test site is carefully leveled, and the plate beded into the layer being tested using plaster of Paris and/or bedding sand.
2. Load is applied to the plate using a hydraulic jack in a series of pre-determined steps. This application of load and the maximum load applied must be designed to conform to the anticipated structural loads.
3. Plate settlement is usually measured by means of dial gauges. The dial gauges are usually fixed to a beam supported by posts, bearing on the soil, some distance from the loaded area to avoid the readings being influenced by the settlement of the plate.[10]

5.4 Interpretations of results

The load intensity and settlement observations of the plate load test are plotted in the form of load settlement curves.

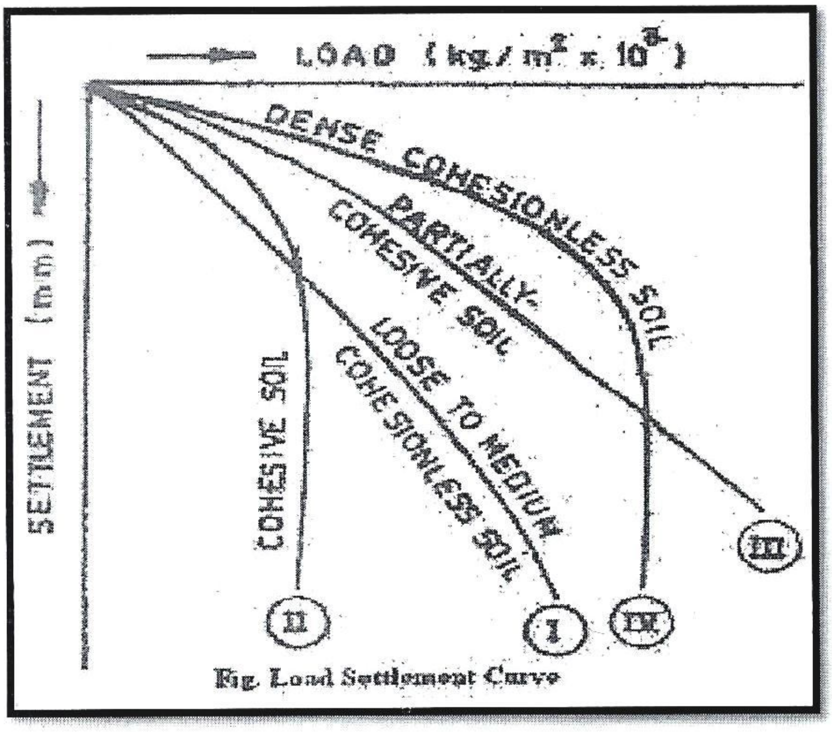


Fig. Load Settlement Curve

Figure (5.5) load settlement curve

The figure above shows four typical curves applied to different soils:

Curve I is typical for loose to medium non-cohesive soils. It can be seen that initially this curve is a straight line, but as the load increases it flattens out. There is no clear point of shear failure.

Curve II is typical for cohesive soils. This may not be quite straight in the initial stages and leans towards settlement axis as the settlement increases.

Curve III is typical for partially cohesive soils.

Curve IV is typical for purely dense non-cohesive soil. [2]

5.5 limitations of plate load test

The plate load test, though very useful in obtaining necessary information about soil for design of foundation has following limitation:

(a) The test results reflect only the character of the soil located within a depth of less than twice the width of bearing plate. Normally the foundations are larger than the test plates, the settlement and shear resistance of soil against shear failure will depend on the properties of much thicker stratum. Thus the results of test could be misleading if the character of the soil changes at shallow depths.

(b) The plate load test being of short duration does not give the ultimate settlements particularly in case of cohesive soils.

(c) For clayey soils the bearing capacity (from shear consideration) for a large foundation, is almost same as that for the smaller test plate. But in dense sandy soils the bearing capacity increases with the size of the foundation and hence the test with smaller size test plate tends to give conservative values in dense sandy soils. [2]

5.6 plate load test

The plate load test made four times, at four layers of structural fills of soil. The device used for this test was CONTROL'S Digital plate bearing test device (Model: 35- T0116/BE).

The model

The model is steel box (86 cm length, 76 cm width, and 68 cm high) fixed on cement base with steel beam to supply the reaction for the load.

Picture (5.2) model of plate load test

The sample

The sample is structural fill from hattab area (sample hattab (2)).

Picture (5.3) sample of plate load test

The procedure

The sample was put in the steel box as four layers, any layer (12 cm thickness) like steps below:

(I) first layer was put in steel box, water added to it (m.c), a little compaction was done to stabilize the layer, plate load test was worked to get bearing capacity of soil (circular plate 20 cm was used, the load was applied to plate by mean of hydraulic jack, deflection of plate was measured by means of deflection dials placed at two points of the plate near its outer edge, results were registered by data loger, and transmitted to a computer), and density test was done on the layer.

(a)

(b)

(c)

(d)

Picture (5.4): (a) a layer (b) circular plate (c) setting up of plate load test (d) hydraulic jack

As shown in appendix (E) soil in first layer has:

Ultimate bearing capacity (b.c) = 280.89 KN/m^2

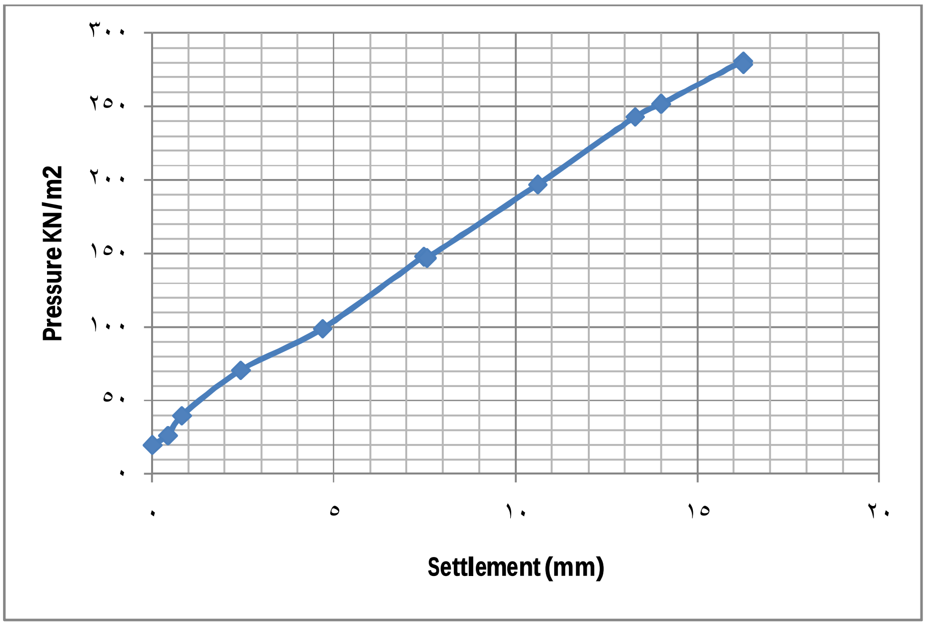


Figure (5.6) plate load test for first layer

(II) The first layer extracted out,a new first layer of structural fill was put in the steel box, water added to it, a little compaction was done to stabilize the layer, second layer of structural fill was put in the steel box, water added to this layer (m.c), a little compaction was done to stabilize it, plate load test was worked to second layer to get bearing capacity of soil, and density test was done to this layer.

As shown in appendix (E) soil in second layer has:

Ultimate bearing capacity (b.c) = 236.94 KN/m^2

Figure (5.7) plate load test for second layer

(III) The new first layer and second layer were extracted out, a new first layer was put in the steel box , water added to it (m.c), a little compaction was done to stabilize the layer, a new second layer was put in steel box also, water added to it (m.c), a little compaction was done to stabilize it, a third layer was put in the steel box, water added to it (m.c), a little compaction was done to it to stabilize this layer, plate load test was worked to it to get bearing capacity of soil, and density test was done to this layer.

As shown in appendix (E) soil in third layer has:

Ultimate bearing capacity (b.c) = 290.13 KN/m^2

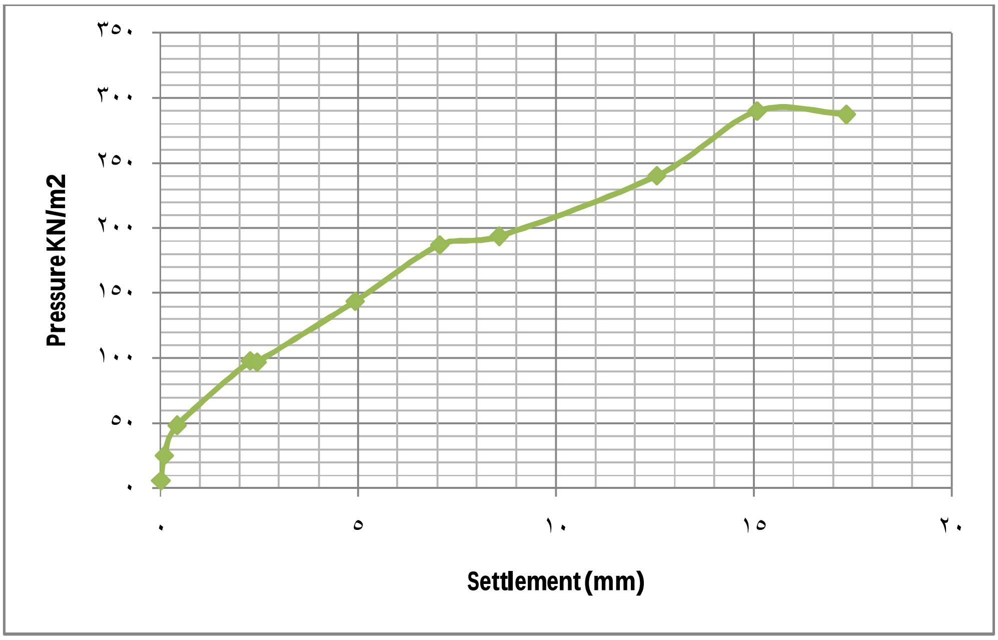


Figure (5.8) plate load test for third layer

(IV) The new first layer, new second layer, and third layer were extracted out, a new first layer was put in the steel box , water added to it (m.c), a little compaction was done to stabilize the layer, a new second layer was put in steel box, water added to it (m.c), a little compaction was done to stabilize it, a new third layer was put in the steel box, water added to it (m.c), a little compaction was done to it to stabilize this layer, fourth layer was put in the steel box, water added to it (m.c), a little compaction was done to it to stabilize this layer, plate load test was worked to it to get bearing capacity of soil, and density test was done to this layer.

As shown in appendix (E) soil in fourth layer has:

Ultimate bearing capacity (b.c) = 477.07 KN/m^2

Figure (5.9) plate load test for fourth layer

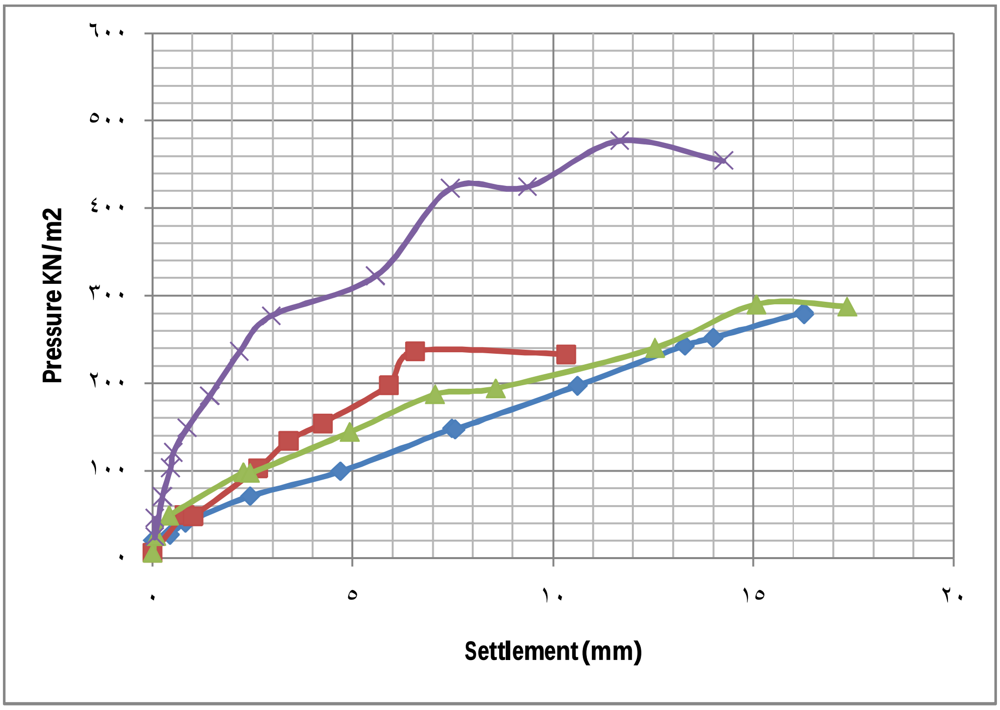


Figure (5.10) plate load test for four layers

5.7Sand Cone Test (ASTM D1556-90)

A small hole (6” x 6” deep) is dug in the compacted material to be tested. The soil is removed and weighed, then dried and weighed again to determine its moisture content. A soil’s moisture is figured as a percentage. The specific volume of the hole is determined by filling it with calibrated dry sand from a jar and cone device. The dry weight of the soil removed is divided by the volume of sand needed to fill the hole. This gives us the density of the compacted soil in lbs per cubic foot. This density is compared to the maximum Proctor density obtained earlier, which gives us the relative density of the soil that was just compacted. [See Figure 5.6]. [11]

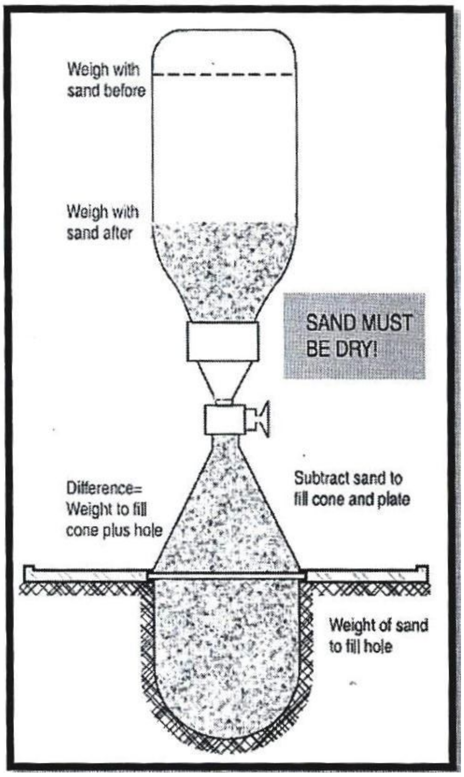


Figure (5.11) sand cone test

5.7.1 Layer (1): As shown in appendix (F) calculation results was as shown below:

- Dry Density of soil (g/cm^3) = 1.44

5.7.2 Layer (2): As shown in appendix (F) calculation results was as shown below:

- Dry Density of soil (g/cm^3) = 1.45

5.7.3Layer (3): As shown in appendix (F) calculation results was as shown below:

- Dry Density of soil (g/cm^3) = 1.59

5.7.4 Layer (4): As shown in appendix (F) calculation results was as shown below:

- Dry Density of soil (g/cm^3) = 1.51

5.8Estimations of bearing capacities values from equations:

Layer (1):

From Terzaghi,s bearing capacity equations,

$\boldsymbol{q_{u=}1.3cN_c + qN_q + 0.4\gamma BN_\gamma}$ (Equation (3.9))

C = 4 Kps

B = 76 cm (width of the model)

q = γ*D

γ = 1.44 g/cm^3

D = 12 cm (thickness of soil under plate)

$\boldsymbol{N_c}$ = 40 (from figure (3.5), due to Φ=31[°])

$\boldsymbol{N_q}$= 25 (from figure (3.6), due to Φ= 31[°])

$\boldsymbol{N_\gamma}$= 30 (from figure (3.7), due to Φ= 31[°])

So,

$q_{u=}$1.3*4*40 + (1.44*10^{-6}*2.205*0.12*25) / (10^{-6}) + (0.4*1.44* 10^{-6}*2.205*0.76*30) / (10^{-6}) =246.49 KN/m^2.

Layer (2):

γ = 1.45 g/cm^3

D = 24 cm

$q_{u=}$1.3*4*40 + (1.45*10^{-6}*2.205*0.24*25) / (10^{-6}) + (0.4*1.45* 10^{-6}*2.205*0.76*30) / (10^{-6}) =256 KN/m^2.

Layer (3):

γ = 1.59 g/cm^3

D = 36 cm

$q_{u=}$1.3*4*40 + (1.59*10^{-6}*2.205*0.36*25) / (10^{-6}) + (0.4*1.59* 10^{-6}*2.205*0.76*30) / (10^{-6}) =271.52 KN/m^2.

Layer (4):

γ = 1.51g/cm^3

D = 48 cm

$q_{u=}$1.3*4*40 + (1.51*10^{-6}*2.205*0.48*25) / (10^{-6}) + (0.4*1.51* 10^{-6}*2.205*0.76*30) / (10^{-6}) =278.32KN/m^2.

5.9 Summary of tests results

Test	**Samples**			
	Hattab (1)	**Hattab (2)**	**El housh**	**Jabil Toria**
Atterburg's Limits	Medium plastic. L.L=37 P.L=25 P.I=12	Medium plastic. L.L=37 P.L=19 P.I=18	Medium plastic. L.L=37 P.L=16 P.I=21	Medium plastic. L.L=37 P.L=21 P.I=16
Sieve Analysis	Low plasticity clay (CL).	Low plasticity clay (CL).	Low plasticity clay (CL).	Low plasticity clay (CL).
Compaction	OMC =6.4 % MDD=2.223 gm/cm^3	OMC =7.20 % MDD=2.18 gm/cm^3	OMC =7.03 % MDD =2.18 gm/cm^3	OMC =6.6 % MDD=2.12 gm/cm^3
CBR	CBR @ 2.5 mm = 33% CBR @ 5 mm = 44%	CBR @ 2.5 mm = 35% CBR @ 5 mm = 37%	CBR @ 2.5 mm = 14% CBR @ 5 mm = 14%	CBR@ 90% of M.D.D = 2% CBR@ 95% of M.D.D =3% CBR@ 98% of M.D.D = 3%
Layers				
	Layer (1)	**Layer (2)**	**Layer (3)**	**Layer (4)**
Shear box	C= 4KPa $\Phi=31^{\circ}$	C= 4KPa $\Phi=31^{\circ}$	C= 4KPa $\Phi=31^{\circ}$	C= 4KPa $\Phi=31^{\circ}$
Site density	Dry Density= 1.44 g/cm^3	Dry Density= 1.45 g/cm^3	Dry Density= 1.59 g/cm^3	Dry Density= 1.51 g/cm^3
Plate load	**B.C=280.89 KN/m^2**	**B.C=236.94 KN/m^2**	**B.C=290.13 KN/m^2**	**B.C=477.07 KN/m^2**
Estimated B.C	**B.C=246.49 KN/m^2**	**B.C=256 KN/m^2**	**B.C=271.52 KN/m^2**	**B.C=278.32 KN/m^2**

6. Discussion and comments:

Testing results and problems faced in this research will be discussed in this chapter.

- The selected samples were four samples from hattab (1), hattab (2), el housh, and jabil toria.
- All samples are medium plastic.
- The sieve analysis shows that all samples are clayey gravel of low plasticity from classification.
- The sieve analysis shows that all samples contain 12% sand and 58% gravel from curves of sieve analysis.
- Hattab (2) sample was chosen for plate load test in the model because it has good results from tests , also values of it is CBR make it good for structural fill’s layers.
- The values of angle of internal friction and cohesion for samples are suggestedfrom reference (12) depend on type of soil, because shear box (large size) not available in laboratory for direct shear test, even proper shear box in other laboratories did not work yet.
- The chosen sample for plate load test was put in the model (steel box) as four layers.
- Plate load test and density test were done to all layers.
- At site density test, in order to stabilize samples, some water was added, and moisture content recorded.
- At plate load test, plate was put at center of the layer actually due to narrow area in the box.

* At plate load test, layer (4) had the maximum value of bearing capacity, while layer (2) had the minimum value of bearing capacity, and the value of bearing capacity for layer (1) and layer (3) was almost equal.

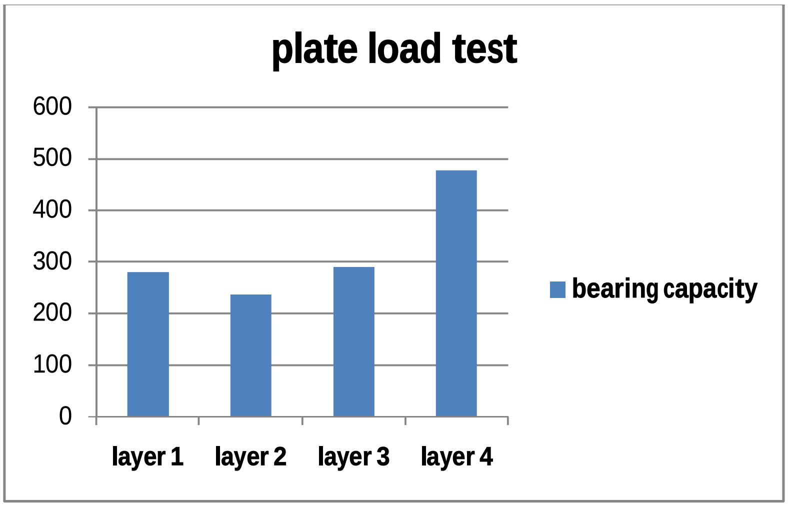


Figure (6.1) bearing capacity from plate load test

* From Terzaghi,s bearing capacity equations, layer (4) had the maximum value of bearing capacity, while layer (1) had the minimum value of bearing capacity, and increase to layer (2) and to layer (3).

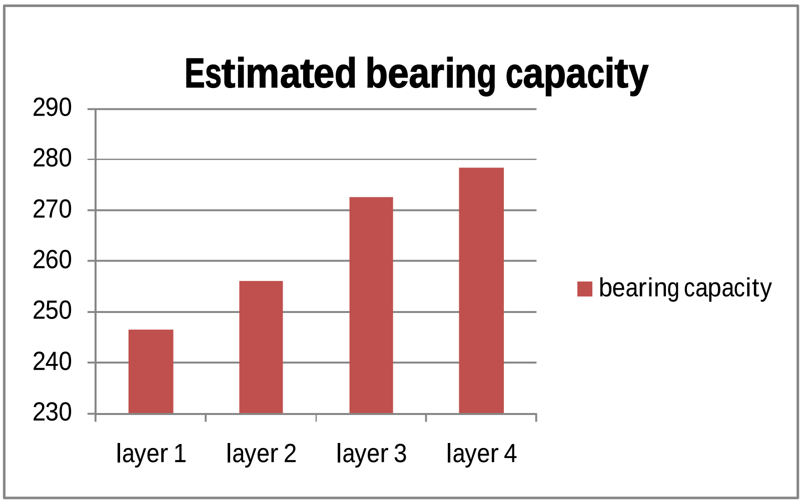


Figure (6.2) Estimated bearing capacity

- In Relative density test compaction degree for layer (3) was 102%, which mean that the filed compaction for this layer was perfect thus the bearing capacity by plate load test is approximately most near value to the estimated ultimate bearing capacity.
- When comparing the estimated values of bearing capacity with plate load test values of bearing capacity, it shows that:
  - For layer (1), estimated value was 87.8 % of plate load test value of bearing capacity.
  - For layer (2), estimated value was 108 % of plate load test value of bearing capacity.
  - For layer (3), estimated value was 93.6% of plate load test value of bearing capacity.
  - For layer (4), estimated value was 58.3% of plate load test value of bearing capacity.
- From plate load test, value of bearing capacity for layer (1) was 280.89 KN/m2, then decrease to 236.94 KN/m2 in layer (2),after that layer (3) compressed above layer (1) and (2), so that value of bearing capacity increase to 290.13 KN/m2, also layer (4) compressed above layers (1),(2), and (3) , so that the value of bearing capacity increased very much and reach to 477.07 KN/m2.

7.1 Conclusion:

This chapter contains summary for results of all tests done to structural fill material and recommendations.

From the results obtained in this study the following conclusions can be summarized:

In the research four different types of structural fills had been chosen namely: Hattab (1) sample, Hattab (2) sample,El housh sample, and Jabil Toria sample).All necessary laboratory works were done to classify there types (liquid limits and sieve analysis).Compaction test was done to it to find optimum moisture content and maximum dry density also CBR test was carried out for all samples of soil.

The model is steel box (86 cm length, 76 cm width, and 68 cm high) fixed on cement base with steel beam to supply the reaction for the load. It was prepared to be filled with four layers from the chosen sample which is Hattab (2).Plate load test was carried out on each layer.Sand cone test also was done to each layer, and shear box test was carried out for this sample.

(1) The following parameters were deduced for the four samples tested:

- All samples contains clayey gravel of low plasticity Optimum moisture content (O.M.C): min (6.40%) _ max (7.2%).
- Maximum dry Density (M.D.D): min (2.18gm/cm^3) _ max (2.223gm/cm^3).
- CBR (By B.S. method):
  _CBR @ 2.5 mm: min (14%) _ max (35%).
  _CBR @ 5 mm: min (14%) _ max (44%).

(2) The following points can be concluded on the sample used for plate load test.

- Dry density of soil (g/cm^3): min (1.44) _ max (1.59).
- Compaction degree: min (102%) _ max (92 %).
- C=4 [KPa], ϕ= 31[°].
- The plate load test bearing capacity: min (236.94 KN/m2) _ max (477.07 KN/m2).
- The estimated ultimate bearing capacity: min (246.49 KN/m2) _ max (278.32 KN/m2).

(3) For plate load test:

- Layer (1) had a large value of bearing capacity in the beginning, and that is referring to strong steel base of the box.
- Value of bearing capacity increased gradually in layer (2) , layer (3) and layer (4) due to increasing of the thickness of the compacted fill layer and the compression on them.
- Layer (2) had the minimum value of bearing capacity compare to layer (1), this is due to compaction of the latter layer on small thickness and the existence of the steel plate as a hard stratum.

(4) Considering the theoretical estimation **of bearing capacity values:**

- Layer (1) had the minimum value of bearing capacity.
- Layer (4) had the maximum value of bearing capacity.
- The sequence of increasing for estimated values of bearing capacity to layer (1), layer (3), and layer (4), symmetric with that from plate load test (figure (6.1) and (6.2)).

- The filed compaction for layer (3) was perfect (degree of compaction=102%), thus the bearing capacity by plate load test is approximately near to the estimated ultimate bearing capacity.

7.2 Recommendations:

Suggestions for further work:

Extension of this study can be as follows:

- Plate load test would be done in the side in trench on many layers of structural fills to give real values of bearing capacity.
- Three transducers can be used to read settlement and take the average.
- Multiple samples must be used for each type of soil.
- The compaction on soil can be made by different ways to notice the influence of compaction on bearing capacity.
- Plate load test must be expanded to use as a fast method for finding the bearing capacity and a useful instrument for quality controlling issues.
- Computer applications should be entered on bearing capacity estimations depend on theories of scientists.
- Shear box test would be done to any size of soil particles to give cohesion and angle of internal friction values which help to calculate bearing capacity.
- Quality must be controlled through testing materials.

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13. *Source: Table 401.4.1; CABO One- and Two- Family Dwelling Code; 1995.*

Appendixes

Appendix (A)

Liquid Limit Test Results

Hattab (1) sample:

Test No.	1	2	3	4	5	6
Cone Penet.	**16.6**	**18.6**	**22.5**	**24.5**	P.L	P.L
WtSoil+Cont.	29.79	33.20	33.91	32.91	25.01	26.84
Dry Soil+Cont.	27.61	29.95	29.49	30.36	24.87	21.05
Mass of Cont.	21.54	21.38	21.42	21.22	9.21	9.23
M.C%	**35.9**	**37.9**	**54.8**	**27.9**	0.9	49.0

Liquid Limit (LL):	**37.0**
Plastic Limit(PL)	25
Plasticity Index(PI):	12

Hattab (2) sample:

Test No.	1	2	3	4	5	6
Cone Penet.	**15.5**	**19.0**	**21.1**	**25.0**	P.L	P.L
WtSoil+Cont.	36.10	35.50	33.68	36.63	22.22	24.50
Dry Soil+Cont.	32.66	31.95	30.52	32.56	20.10	22.03
Mass of Cont.	21.50	21.38	21.40	21.63	8.82	9.17
M.C%	**30.8**	**33.6**	**34.6**	**37.2**	18.8	19.2

Liquid Limit (LL):	**37.0**
Plastic Limit(PL)	19
Plasticity Index(PI):	18

El housh sample:

Test No.	1	2	3	4	5	6
Cone Penet.	**15.4**	**18.4**	**22.0**	**24.3**	P.L	P.L
WtSoil+Cont.	30.27	35.89	32.98	34.58	21.03	22.18
Dry Soil+Cont.	28.40	32.73	30.27	31.45	19.32	20.35
Mass of Cont.	21.40	21.33	21.27	21.44	8.77	9.06

M.C%	**26.7**	**27.7**	**30.1**	**31.3**	16.2	16.2

Liquid Limit (LL):	**37.0**
Plastic Limit(PL)	16
Plasticity Index(PI):	21

Jabil Toria sample:

Test No.	1	2	3	4	5	6
Cone Penet.	**15.5**	**19.0**	**22.0**	**23.5**	P.L	P.L
WtSoil+Cont.	39.82	36.64	40.13	43.04	24.06	23.67
Dry Soil+Cont.	35.22	32.66	34.98	36.86	21.47	21.04
Mass of Cont.	21.34	21.37	21.34	21.39	9.02	8.71

M.C%	**33.1**	**35.3**	**37.8**	**39.9**	20.8	21.3

Liquid Limit (LL):	**37.0**
Plastic Limit(PL)	21
Plasticity Index(PI):	16

Appendix (B)

Sieve Analysis Test Result

Hattab (1) sample:

Total Weight		10000.0	
Weight Passing # 4		3413	
Weight Washed		182.9	
Factor		18.660	
B.S sieve	**Ret By Wet**		**%age Passing**
76.2	0	0	100.0
63.5	0	0	100.0
38.1	0	0	100.0
25.4	435	435	95.7
19.0	916	1351	86.5
12.7	1377	2728	72.7
9.50	1028	3756	62.4
4.75	2818	6574	34.3
2.80	58.2	1086.0	23.4
2.00	81.5	1520.8	19.1
1.40	95.4	1780.2	16.5
1.00	104.5	1950.0	14.8
0.710	110.8	2067.6	13.6
0.500	116.9	2181.4	12.4
0.425	119	2220.6	12.1
0.250	126.4	2358.7	10.7
0.212	128	2388.5	10.4
0.180	128.8	2403.5	10.2
0.150	131.4	2452.0	9.7
0.125	133	2481.8	9.4
0.090	135.3	2524.8	9.0
0.075	136.2	2541.6	8.8
0.063	136.6	2549.0	8.8

Hattab (2) sample:

Total Weight		10000.0	
Weight Passing # 4		4426	
Weight Washed		256.8	
Factor		17.235	
B.Ssieve	**Ret By Wet**		**%age Passing**
76.2	0	0	100.0
63.5	0	0	100.0
38.1	0	0	100.0
25.4	652	652	93.5
19.0	467	1119	88.8
12.7	1190	2309	76.9
9.50	985	3294	67.1
4.75	2265	5559	44.4
2.80	78	1344.3	31.0
2.00	112	1930.3	25.1
1.40	130	2240.6	22.0
1.00	139.1	2397.4	20.4
0.710	146.5	2525.0	19.2
0.500	154.5	2662.8	17.8
0.425	157.1	2707.7	17.3
0.250	164.8	2840.4	16.0
0.212	166	2861.0	15.8
0.180	168.1	2897.2	15.4
0.150	169.5	2921.4	15.2
0.125	170.1	2931.7	15.1
0.090	173.1	2983.4	14.6
0.075	174	2998.9	14.4
0.063	174.3	3004.1	14.4

El housh sample:

Total Weight		10000.0	
Weight Passing # 4		6785	
Weight Washed		294.4	
Factor		23.047	
B.Ssieve	**Ret By Wet**		**%age Passing**
76.2	0	0	100.0
63.5	0	0	100.0
38.1	0	0	100.0
25.4	197	197	98.0
19.0	151	348	96.5
12.7	570	918	90.8
9.50	593	1511	84.9
4.75	1703	3214	67.9
2.80	58.1	1339.0	54.5
2.00	84.7	1952.1	48.3
1.40	100.1	2307.0	44.8
1.00	111.3	2565.1	42.2
0.710	123	2834.8	39.5
0.500	139.6	3217.3	35.7
0.425	146	3364.8	34.2
0.250	168	3871.9	29.1
0.212	170.9	3938.7	28.5
0.180	175.3	4040.1	27.5
0.150	178.4	4111.6	26.7
0.125	181.3	4178.4	26.1
0.090	186.1	4289.0	25.0
0.075	187.5	4321.3	24.6
0.063	188	4332.8	24.5

Jabil Toria sample:

Total Weight		10000.0	
Weight Passing # 4		6310	
Weight Washed		130	
Factor		48.538	
B.Ssieve	**Ret By Wet**		**%age Passing**
76.2	0	0	100.0
63.5	0	0	100.0
38.1	0	0	100.0
25.4	0	0	100.0
19.0	45	45	99.6
12.7	350	395	96.1
9.50	704	1099	89.0
4.75	2576	3675	63.3
2.80	28.5	1383.3	49.4
2.00	44	2135.7	41.9
1.40	50	2426.9	39.0
1.00	52	2524.0	38.0
0.710	53.7	2606.5	37.2
0.500	56.1	2723.0	36.0
0.425	57.2	2776.4	35.5
0.250	61.8	2999.7	33.3
0.212	63	3057.9	32.7
0.180	63.8	3096.8	32.3
0.150	65.4	3174.4	31.5
0.125	66.6	3232.7	30.9
0.090	68.2	3310.3	30.1
0.075	69	3349.2	29.8
0.063	69.1	3354.0	29.7

Appendix (C)

Compaction Test Results

Hattab (1) sample:

Mould wt.	3349
Mould vol.	2126

	1	2	3	4
wt.of wet soil+mould(g)	7902	8295	8415	8330
wt.ofmould(g)	3349	3349	3349	3349
wt.ofwet soil(g)	4553	4946	5066	4981
volume of mould(cc)	2126	2126	2126	2126
wet density(g/cc)	2.140	2.330	2.380	2.340
wt.ofcont(g)	21.99	23.11	23.65	19.61
wt.of wet soil+cont(g)	219.41	223.90	240.27	249.95
wt.of dry soil+cont(g)	212.93	213.80	225.25	232.60
Moisture content (%)	**3.39**	**5.30**	**7.45**	**8.15**
dry density(g/cc)	**2.07**	**2.21**	**2.21**	**2.164**

Hattab (2) sample:

	1	2	3	4
wt.of wet soil+mould(g)	7694	8065	8321	8236
wt.ofmould(g)	3349	3349	3349	3349
wt.ofwet soil(g)	4345	4716	4972	4887
volume of mould(cc)	2126	2126	2126	2126
wet density(g/cc)	2.040	2.220	2.340	2.300
wt.ofcont(g)	28.22	22.93	27.94	22.95
wt.of wet soil+cont(g)	307.47	227.00	236.94	235.39
wt.of dry soil+cont(g)	299.25	217.40	223.04	217.46
Moisture content(%)	**3.03**	**4.94**	**7.12**	**9.22**
dry density(g/cc)	**1.98**	**2.12**	**2.18**	**2.106**

El housh sample:

	1	2	3	4
wt.of wet soil+mould(g)	7651	7922	8315	8154
wt.ofmould(g)	3349	3349	3349	3349
wt.ofwet soil(g)	4302	4573	4966	4805
volume of mould(cc)	2126	2126	2126	2126
wet density(g/cc)	2.020	2.150	2.340	2.260
wt.ofcont(g)	25.21	28.18	28.87	23.77
wt.of wet soil+cont(g)	233.85	232.17	238.46	232.00
wt.of dry soil+cont(g)	226.95	222.53	224.70	214.44
Moisture content (%)	**3.42**	**4.96**	**7.03**	**9.21**
dry density(g/cc)	**1.95**	**2.05**	**2.19**	**2.069**

Jabil Toria sample:

	1	2	3	4
wt.of wet soil+mould(g)	7635	8099	8215	8135
wt.ofmould(g)	3349	3349	3349	3349
wt.ofwet soil(g)	4286	4750	4866	4786
volume of mould(cc)	2126	2126	2126	2126
wet density(g/cc)	2.020	2.230	2.290	2.250
wt.ofcont(g)	24.19	25.89	23.50	20.08
wt.of wet soil+cont(g)	252.15	247.47	235.94	257.38
wt.of dry soil+cont(g)	243.89	235.79	221.45	237.33
Moisture content(%)	**3.76**	**5.56**	**7.32**	**9.23**
dry density(g/cc)	**1.95**	**2.11**	**2.13**	**2.060**

Appendix (D)

CBR Test Results

Hattab (1) sample:

M.C%		
Mass of wet soil + container =	234.53	238.16
Mass of dry soil +container =	225.47	227.17
Mass of container =	25.21	28.7
M.C% =	4.52	5.54

Densities	
Wt. of Mould + Wet Sample =	8843
Wt. of Mould =	4256
Mould Volume =	2086
Wt. of Wet Sample =	4587
Bulk Density =	2.20
Dry Density =	2.09

Penetration (mm)	**Force** (KN)	**Factor**	**CBR**
2.5	**4.305**	**13.2**	**33**
5.0	**8.84**	**19.96**	**44**

Factor =		0.0210
حطاب (1)		
Penetration (mm)	**Force** (KN)	**P.R.R**
0	0	0
0.25	0.084	4
0.5	0.315	15
0.75	0.609	29
1	0.966	46
1.25	1.344	64
1.5	1.701	81
1.75	2.016	96
2	2.373	113
2.25	2.793	133
2.5	3.255	155
2.75	3.822	182
3	4.305	205
3.25	4.767	227
3.5	5.208	248
3.75	5.565	265
4	6.006	286
4.25	6.405	305
4.5	6.825	325
4.75	7.287	347
5	7.833	373
5.25	8.421	401
5.5	8.841	421
5.75	9.366	446
6	9.744	464
6.25	10.206	486
6.5	10.668	508
6.75	10.983	523
7	11.361	541
7.25	11.676	556
7.5	12.012	572

Hattab (2) sample:

Factor =	0.0210	
Penteration	Force	P.R.R
0	0	0
0.25	0.105	5
0.5	0.273	13
0.75	0.609	29
1	1.932	92
1.25	2.625	125
1.5	3.171	151
1.75	3.591	171
2	3.885	185
2.25	4.263	203
2.5	4.641	221
2.75	5.103	243
3	5.439	259
3.25	5.796	276
3.5	6.09	290
3.75	6.384	304
4	6.573	313
4.25	6.762	322
4.5	6.972	332
4.75	7.203	343
5	7.455	355
5.25	7.665	365

5.5	7.854	374
5.75	8.043	383
6	8.127	387
6.25	8.253	393
6.5	8.316	396
6.75	8.337	397
7	8.442	402
7.25	8.526	406

M.C%

Mass of wet soil + container =	234.53	238.16
Mass of dry soil +container =	225.47	227.17
Mass of container =	25.21	28.7
M.C% =	4.52	5.54
		5.03

Densities

Wt. of Mould + Wet Sample =	8843
Wt. of Mould =	4256
Mould Volume =	2086
Wt. of Wet Sample =	4587
Bulk Density =	2.20
Dry Density =	2.09

Penetration (mm)	**Force** (KN)	**Factor**	**CBR**
2.5	**4.641**	**13.2**	**35**
5.0	**7.46**	**19.96**	**37**

El housh sample:

M.C%		
Mass of wet soil + container =	234.53	238.16
Mass of dry soil +container =	225.47	227.17
Mass of container =	25.21	28.7
M.C% =	4.52	5.54
		5.03

Densities	
Wt. of Mould + Wet Sample =	8843
Wt. of Mould =	4256
Mould Volume =	2086
Wt. of Wet Sample =	4587
Bulk Density =	2.20
Dry Density =	2.09

Penteration (mm)	**Force** (KN)	**Factor**	**CBR**
2.5	**1.806**	**13.2**	**14**
5.0	**2.86**	**19.96**	**14**

Factor =		0.0210
الحوش		
Penetration (mm)	**Force** (KN)	**P.R.R**
0	0	0
0.25	0.042	2
0.5	0.105	5
0.75	0.189	9
1	0.315	15
1.25	0.483	23
1.5	0.735	35
1.75	0.966	46
2	1.155	55
2.25	1.344	64
2.5	1.47	70
2.75	1.596	76
3	1.722	82
3.25	1.806	86
3.5	1.911	91
3.75	1.995	95
4	2.079	99
4.25	2.163	103
4.5	2.289	109
4.75	2.415	115
5	2.52	120
5.25	2.646	126
5.5	2.751	131
5.75	2.856	136
6	2.94	140
6.25	3.045	145
6.5	3.129	149
6.75	3.192	152
7	3.276	156
7.25	3.36	160

Appendix (E)

Plate Load Test Results

Layer (1):

pressure	settlement	ch4	ch3	ch1
(KN/m2)	(mm)	(mm)	(mm)	(KN)
20.06369	0.005	0.005	0.005	**0.63**
26.43312	0.4305	0.028	0.833	**0.83**
39.80892	0.8175	0.236	1.399	**1.25**
70.70064	2.4375	2.078	2.797	**2.22**
99.04459	4.6855	4.337	5.034	**3.11**
148.0892	7.4775	7.073	7.882	**4.65**
147.1338	7.559	7.152	7.966	**4.62**
197.1338	10.6125	10.332	10.893	**6.19**
242.9936	13.289	13.03	13.548	**7.63**
251.9108	14.003	13.75	14.256	**7.91**
280.8917	16.26	16.027	16.493	**8.82**
278.6624	16.267	16.034	16.5	**8.75**

Layer (2):

pressure	settlement	ch4	ch3	ch1
(KN/m2)	(mm)	(mm)	(mm)	**(KN)**
5.732484	0.006	0.005	0.007	**0.18**
49.04459	0.797	1.031	0.563	**1.54**
48.08917	1.008	1.223	0.793	**1.51**
102.8662	2.6405	2.719	2.562	**3.23**
134.0764	3.403	3.502	3.304	**4.21**
154.4586	4.2535	4.414	4.093	**4.85**
197.7707	5.9005	6.202	5.599	**6.21**
236.9427	6.554	7.888	5.22	**7.44**
233.4395	10.3195	7.963	12.676	**7.33**

Layer (3):

pressure	settlement	ch4	ch3	ch1
(KN/m2)	(mm)	(mm)	(mm)	**(KN)**
5.73248	0.012	0.012	0.012	**0.18**
25.1592	0.0955	0.021	0.17	**0.79**
48.4076	0.425	0.068	0.782	**1.52**
98.0891	2.2745	1.903	2.646	**3.08**
97.1337	2.4415	2.065	2.818	**3.05**
143.949	4.9265	4.568	5.285	**4.52**
187.261	7.063	6.724	7.402	**5.88**
193.949	8.5725	8.248	8.897	**6.09**
240.445	12.5505	12.197	12.904	**7.55**
290.127	15.086	14.631	15.541	**9.11**
287.579	17.3475	16.897	17.798	**9.03**

Layer (4):

pressure	settlement	ch4	ch3	ch1
(KN/m2)	(mm)	(mm)	(mm)	**(KN)**
45.22293	0.0615	0.119	0.004	**1.42**
24.20382	0.066	0.128	0.004	**0.76**
70.06369	0.2535	0.456	0.051	**2.2**
102.8662	0.4565	0.82	0.093	**3.23**
120.7006	0.527	0.944	0.11	**3.79**
149.0446	0.8765	1.581	0.172	**4.68**
185.3503	1.4355	2.576	0.295	**5.82**
235.9873	2.1815	3.921	0.442	**7.41**
277.0701	2.968	5.454	0.482	**8.7**
322.6115	5.575	10.189	0.961	**10.13**
422.6115	7.4425	12.744	2.141	**13.27**
424.2038	9.357	14.814	3.9	**13.32**
477.0701	11.6695	17.233	6.106	**14.98**
454.1401	14.261	19.905	8.617	**14.26**

*Area of plate = 0.0314 m^2.

Appendix (F)

Site density Test Results

*Unit of sand = 1.37g/cc.

Layer (1):

data	results
Wt. of wet soil (g)	2606
Initial wt. of sand (g)	5000
Residual wt. of sand (g)	1130
Wt. of sand in cone (g)	1446
Wt. of sand in hole (g)	2424
Volume of hole (cc)	1769
Wet density of soil (cc)	1.47
Wt. of wet soil + container (g)	224.72
Wt. of dry soil + container (g)	216.93
Wt. of container (g)	23.01
Moisture content (%)	4.02
Dry density of soil (g/cc)	1,44
Compaction degree (%)	92

Layer (2):

data	results
Wt. of wet soil (g)	2526
Initial wt. of sand (g)	5000
Residual wt. of sand (g)	1232
Wt. of sand in cone (g)	1446
Wt. of sand in hole (g)	2322
Volume of hole (cc)	1695
Wet density of soil (cc)	1.49
Wt. of wet soil + container (g)	230.52
Wt. of dry soil + container (g)	221.44
Wt. of container (g)	23.15
Moisture content (%)	4.58
Dry density of soil (g/cc)	1,45
Compaction degree (%)	92

Layer (3):

data	results
Wt. of wet soil (g)	2994
Initial wt. of sand (g)	5000
Residual wt. of sand (g)	1050
Wt. of sand in cone (g)	1446
Wt. of sand in hole (g)	2504
Volume of hole (cc)	1828
Wet density of soil (cc)	1.64
Wt. of wet soil + container (g)	228.91
Wt. of dry soil + container (g)	220.56
Wt. of container (g)	24.83
Moisture content (%)	4.27
Dry density of soil (g/cc)	1.59
Compaction degree (%)	102

Layer (4):

data	results
Wt. of wet soil (g)	2903
Initial wt. of sand (g)	5000
Residual wt. of sand (g)	996
Wt. of sand in cone (g)	1469
Wt. of sand in hole (g)	2535
Volume of hole (cc)	1850
Wet density of soil (cc)	1.57
Wt. of wet soil + container (g)	214.23
Wt. of dry soil + container (g)	204.51
Wt. of container (g)	23.66
Moisture content (%)	5.37
Dry density of soil (g/cc)	1.51
Compaction degree (%)	96