CE 6502- FOUNDATION ENGINEERING
UNIT - I

1. What is mean dilataney? [N/D-15]

the phenomenon exhibited by some fluids, sols, and gels in which they become more viscous or solid under pressure.

2. Write the uses of bore hole report. [N/D-15]

a borehole used as a water well is completed by installing a vertical pipe (casing) and well screen to keep the borehole from caving. This also helps prevent surface contaminants from entering the borehole and protects any installed pump from drawing in sand and sediment.

3. Differentiate disturbed and undisturbed samples. [M/J-16]

Disturbed soil sample

Natural structure of soils gets partly or fully modified and destroyed

Undisturbed soil sample

Natural structure and properties remain preserved

4. What are the limitations of Static Cone Penetration test? [M/J-16]

- No sample recovered
- Penetration depth limited to 150 – 200 feet
- Normally cannot push through gravel
- Requires special equipment and skilled operator

5. What are the information obtained in general exploration? [N/D-16]

Preliminary selection of foundation type, depth of water, depth and composition of soil strata, Engineering properties required disturbed or partly disturbed samples, approximate values of strength and compressibility.

6. What are various methods of site exploration? [N/D-16]

Open excavation, borings, geophysical methods, sub-surface soundings.

1. Explain in detail about the geophysical method of site exploration with neat sketch. [N/D-15]

... care and sensitivity.

1.10. BROAD CLASSIFICATION OF SOIL EXPLORATION

Soil exploration methods may be classified as (1) Direct exploration, (2) Semi direct exploration and (3) Indirect exploration. They are distinguished as given in Table 1.1(*a*) and their features are given in Table 1.1 (*b*).

Table 1.1. (a) Methods of Exploration

Direct	Semi Direct	Indirect
Trial pits, Trenches and load tests	Boring and penetration tests Most common methods to reach deep layer, rock, to study the resistance to load and deep layer sample collection	Seismic, Electric resistivity and dynamic methods

Table 1.1. (b) Features of the Methods of Exploration

Direct	Semi Direct	Indirect
(1) Undisturbed (natural) soil profile and water table shall be observed visually. UDS and DS samples shall be collected from vertical, inclined and horizontal directions.	(1) Visual observation of soil profile is not possible.	(1) No sample can be collected.

Direct	Semi Direct	Indirect
(2) Strength test and consistency test shall be made and the results shall be directly predicted.	(2) Hand boring is practicable upto 10m and machine boring even for 100 m deep.	(2) Less cost and rapid test than load test.
(3) Testing upto 4 m (or less if water table is encountered) is practically possible.	(3) Strength of soil is predicted, insitu at any required depth.	
(4) To feel the practical problems of opening the ground and to see whether the sides of excavation, stands, crumbles, collapse, *etc.*		

1.10.1. COMMON METHODS OF SITE EXPLORATION

As per IS code, the common methods of site exploration are given below:

(a) Open trial pits: The method consists of excavating trial pits and thereby exposing the subsoil surface thoroughly, enabling undisturbed samples to be taken from the sides and bottom of the trial pits. This is suitable for all types of formations, but should be used for small depths (upto 3 m). In the case of cuts which cannot stand below water table, proper bracing should be given.

(b) Auger boring: The auger is either power or hand operated with periodic removal of the cuttings.

(c) Shell and auger boring: Both manual and mechanized rig can be used for vertical borings. The tool normally consists of augers for soft to stiff clays, shells for very stiff and hard clays, and shell or sand pumps for sandy strata attached to sectional boring rods.

(d) Wash boring: In wash boring, the soil is loosened and removed from the bore hole by a stream of water or drilling mud is worked up and down or rotated in the

bore hole. The water or mud flow carries the soil up the annular space between the wash pipe and the casing and it overflows at ground level, where the soil in suspension is allowed to settle in a pond or tank and the fluid is recirculated as required. Samples of the settled out soil can be retained for identification purposes but this procedure is often unreliable. However, accurate identification can be obtained if frequent 'dry' sampling is resorted to using undisturbed sample tubes.

(e) ***Sounding/Probing*** including standard penetration test, dynamic and static cone penetration test.

(f) Geophysical method

(g) Percussion boring and rotary boring

(h) Pressure meter test

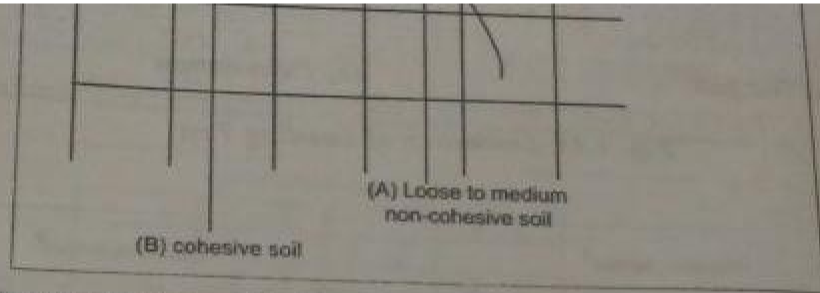


Fig. 1.12. (c) Load Vs. Settlement curves for a non-cohesive and cohesive soil from plate load test

1.16. SEMI DIRECT METHOD OF EXPLORATION

1.16.1. AUGER BORING

The auger is either

(1) Power operated or

(2) Hand operated;

with periodic removal of 3.75 mm diameter of the cutting.

For small jobs upto 10 m deep investigations, (1) hand operated auger may be used.

Hand operated augers are of two types, namely

(1) Post hole auger (Bucket auger) and

(2) Helical auger (Screw auger)(Fig.1.13 (*a*), (*b*), (*c*), (*d*), (*e*), (*f*))

To get undisturbed sample at any depth remove the auger and fit a sampler with hammer and collect the undisturbed sample.

Power driven auger is used to do number of borings at quicker rate.

(Fig.1.13 (*d*))

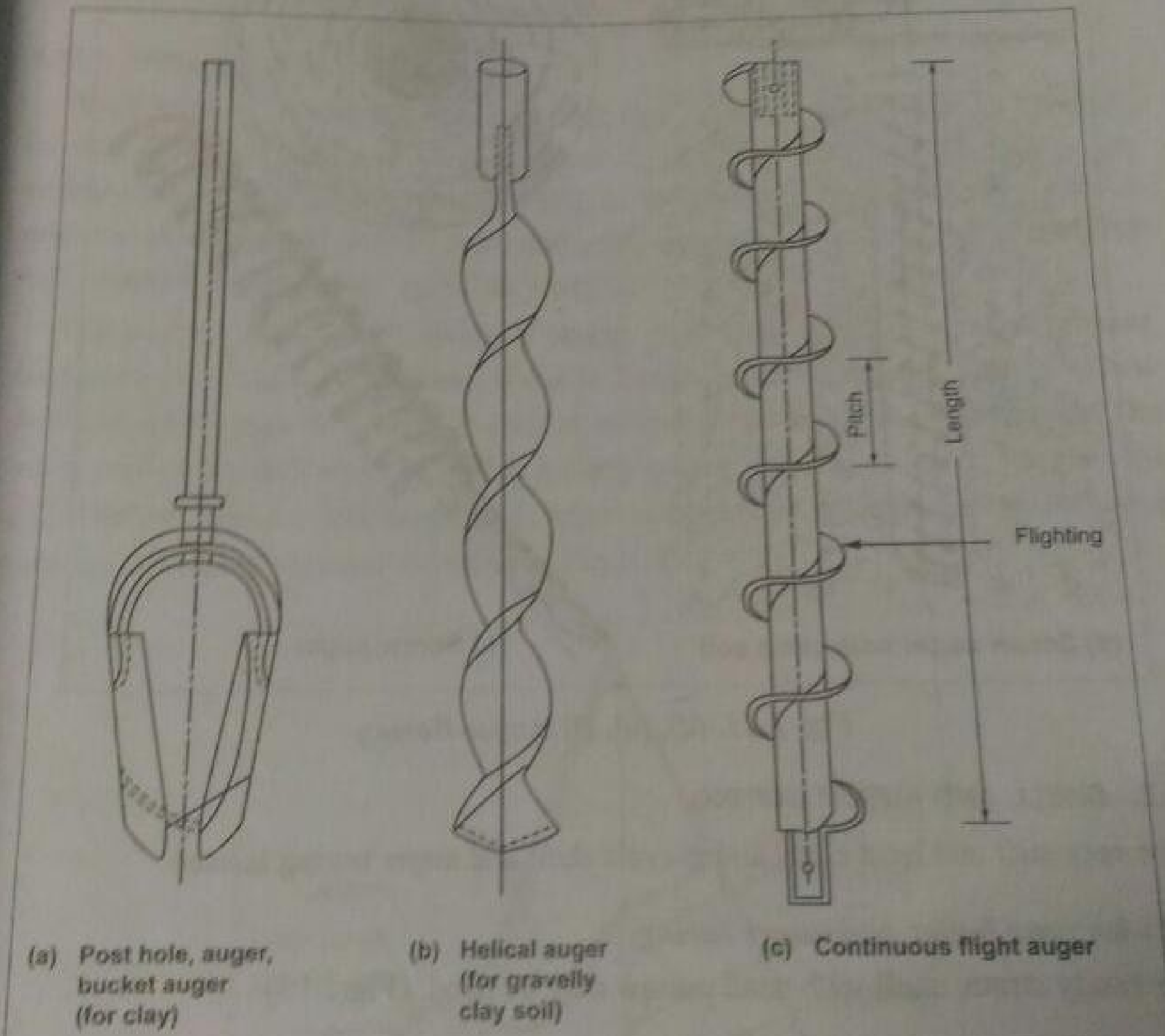


Fig. 1.13. (a), (b), (c) Auger Boring

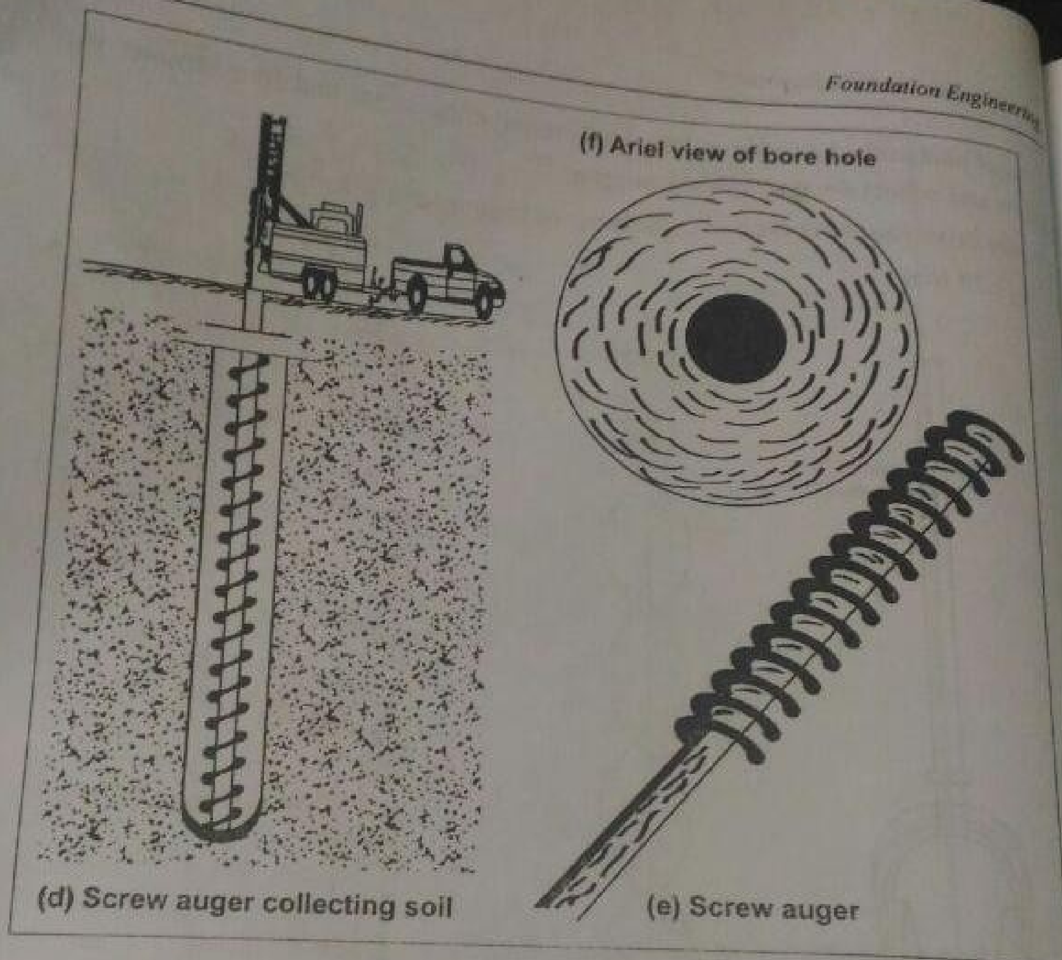


Fig. 1.13. (d), (e), (f) Auger Boring

1.16.2. SHELL AND AUGER BORING

For very stiff and hard clays and gravels shell and auger boring is used.

Shell with sand boiler and auger boring

For sandy strata, shell with sand pumps may be used. (Fig.1.15)

Casing

Casing or shell is preventing soil falling by caving-in bore hole. Sand/stiff clay require casing.

Chiselling

Soft rock or gravel is broken by chisel.

Sand Pump

Sand pumps / sand boiler are used in sandy soil.

Below water boring may be carried by making use of shell and auger boring, with steel casing pipes or by pumping in bentonite clay slurry (drilling mud) for water tightening of bore hole walls. The shell is a long cylindrical tube with a cutting edge and a valve at the bottom which opens only inwards. Rotary drilling with the use of drill bits is the most rapid method of advancing holes in rock.

1.16.3. WASH BORING

Wash boring is a very convenient method for sand, silt or clay below water table. In this method casing pipe is pushed into bore hole to prevent the sides from casing-in. By a stream of water, the soil in the casing is removed. To get insitu undisturbed sample, stop the boring and push a sampler to collect the undisturbed soil. Washed material is not useful for analysis.

The method consists of driving a casing pipe through a heavy drop hammer upported on a tripod and pulley. Water is forced under pressure through a hollow rill rod which may be rotated or moved up and down inside the casing pipe. The wer end of the drill rod, fitted with a sharp cutting edge or chopping bit, cuts the il. The soil thus cut gets mixed with water and floats up through the annular space tween the casing pipe and the drill rod Fig.1.14.

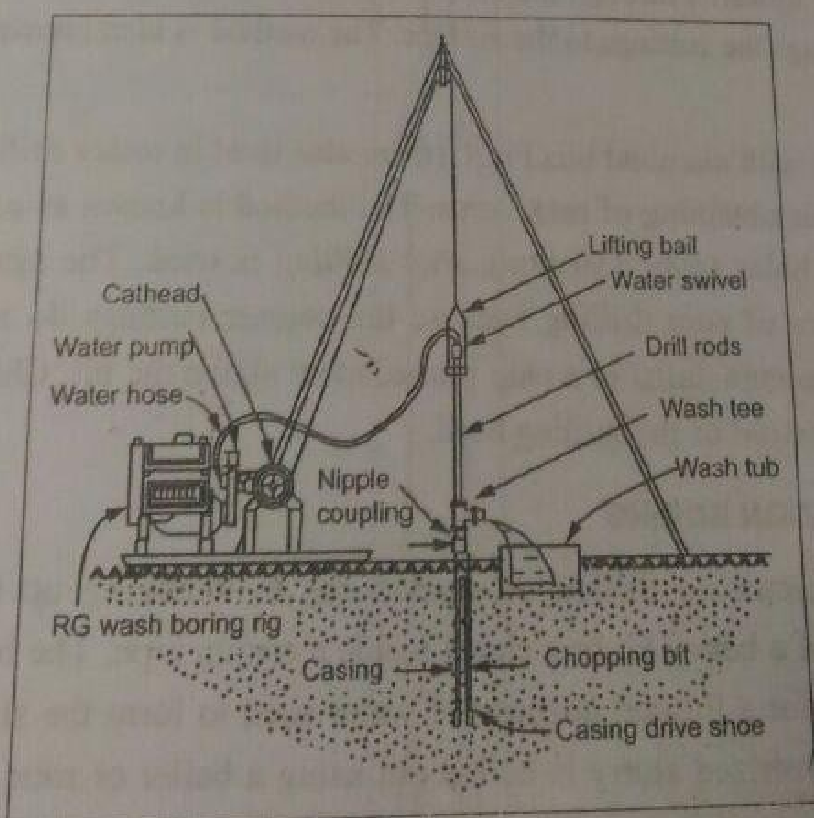


Fig. 1.14. Wash boring

The slurry flowing out provides an indication of the soil type. Replace chopping bit with split spoon to take penetration numbers and undisturbed sample.

In this method, heavier particles of different soil layers remain under suspension in the casing pipe and get mixed up. Because of this, the samples recovered from wash water are of no value. Samples of the soil should be obtained through suitable samplers after the borehole has been cleaned.

Wash boring can be conveniently used even below water table in practically any type of soil except hard soils or rock.

1.16.4. ROTARY DRILLING

Rotary boring or rotary drilling is useful if the soil is highly resistant to augering or wash boring. The method can also be used in case of sands and clay. In this method, boring is effected by the cutting action of a rotating bit which is kept in firm contact with the bottom of the hole. The bit is attached to the lower end of a hollow drill rod which is rotated by a suitable chuck. The bit is attached to the lower end of a hollow drill rod which is rotated by a suitable chuck. Drilling mud (usually bentonite slurry with some admixtures) is continuously forced below down the hollow drill rods. The mud returning upwards through the annular space between the drill rods and the sides of the hole brings the cuttings to the surface. The method is also known as *mud rotary drilling*.

Core barrels with diamond bits Fig.1.16 are also used in rotary drilling and enable the simultaneous obtaining of rock cores. The method is known as *core drilling*. For large diameter holes (over 150 mm), *shot drilling* is used. The system is different from other types of core drilling because the coarser cuttings do not return to the surface but are accumulated in a chip immediately above the bit. Chilled shot is used as an abrasive instead of the drilling head.

1.16.5. PERCUSSION BORING

Boring by percussion drilling is carried out by breaking up the formation by repeated blows of a heavy bit or a chisel inside a casing pipe. The borehole is usually kept dry, except for a limited quantity of water used to form the slurry of pulverized material. The pulverized slurry is bailed out using a bailer or sand pump. Unless the sides of bore hole are likely to cave in, a casing pipe may not be necessary. M

time, in percussion drilling, drill rods are replaced by cables. This is suitable for drilling boreholes in bouldery and gravelly strata.

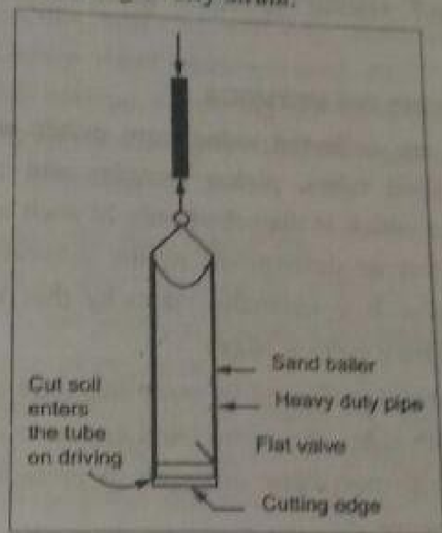


Fig. 1.15. Shell sand bailer

Fig. 1.16. Diamond cutter for Rock

2. Write short notes on: [N/D-15]

i) Selection of Foundation based on sell condition

short notes on selection of foundation based on soil conditions. (8)

Selection of foundation based on soil conditions:

Foundation type	Applicable soil condition
(1) Spread footing, wall footings.	Any conditions where bearing capacity is adequate for applied load.
(2) Mat foundation.	Bearing Capacity of soil is less; over one-half area of building covered by individual footings.
(3) Pile foundations. – floating.	Poor surface and near surface soils; Soils of High BC is 20 – 50 m below basement, but by distributing load along pile shaft soil strength is adequate.
– End bearing	Poor surface and near surface soils; Soils of High BC is 8 – 50 m below ground surface, by distributing load at the end.
(4) Caissons	–do– for pile foundation, Eliminates pile cap by using caissons as column extension.
(5) Retaining walls, bridge abutments	Any type of soil, but a specified zone in back of wall usually of controlled back fills.
(6) Sheet-pile structures	Any soil: Water front structures may require special alloy or corrosion protection. Coffer dams require control of fill material.

(ii) Disturbed and Undisturbed soil sample

Disturbed soil sample

Natural structure of soils gets partly or fully modified and destroyed Undisturbed soil sample

Natural structure and properties remain preserved

(iii) Uses of soil Exploration.

...arise due to ground conditions.

1.5. PURPOSE OF SOIL EXPLORATION

The purposes for soil exploration are to obtain information as bases for:

New structures

1. The selection of type and depth of foundation.
2. The determination of bearing capacity of the selected foundation.
3. The prediction of settlement of the selected foundation.
4. The establishing of the ground water level.
5. The evaluation of the earth pressure against walls, abutments.
6. The provisions against constructional difficulties.
7. The suitability of soil and the degree of compaction of fill (under slabs and pavements and against retaining walls).

Existing structures

1. The investigation of the safety of the structure.
2. The prediction of settlement.
3. The determination of remedial measures if the structure is unsafe or will suffer detrimental settlement.

Highways and Airfields

1. The location of the road (and runways) both vertically and horizontally.
2. The location and selection of borrow material for fills and subgrade treatment.
3. The design and location of ditches, culverts and drains.
4. The design of roadway sections.
5. The need and type of subgrade treatment.
6. The location of local source of construction materials for bases and wearing surfaces.

3. (a) (i) Why SPT values recorded in sand at different depths are corrected for overburden and submergence 7 How these corrections are applied?

11. (b) **Discuss standard penetration test. What are the various corrections? What is the importance of the test in geotechnical engineering?**

SPT is generally done in all types of soils. In granular soils like sand and gravel, it is difficult to get reliable undisturbed samples. SPT test gives insitu soil strength immediately. This method involves driving a standard split spoon sampler (Fig.1.18 (b)) with a standard amount of driving energy (Fig.1.18 (a)). The resistance to penetration obtained is an index of the consistency of relative density of the soil.

1. Drive the casing to the required depth without disturbing the test soil.
2. Clean the hole.
3. Lower the split spoon sampler of 3.5cm diameter and 45 cm length to the bottom of the hole.
4. Use hammer of 63.5 Kg weight and 760 mm height of fall to seat 15 cm depth of sampler.
5. **The total blows required for the second and third 15 cm of penetration is termed as the penetration resistance N.**

The test are made at 1.5 m intervals under normal conditions. A plot of depth Vs penetration resistance shall be plotted. The N value correlated shall be with the properties of cohesionless and cohesive soils are given in Table 1.11 and Table 1.4. In loose coarse gravel deposit, the split spoon tends to slide into the large voids and give low penetration resistance. When the spoon is blocked by a large piece of gravel, more resistance is shown. Therefore the correlation shall be made carefully. At shallow depth, N value is usually low. At a greater depth, the same soil with same relative density would give higher penetration resistance, due to the influence of the weight of

soil above (over burden pressure). The effect of overburden may be obtained form the Fig. 1.19 In saturated silty or fine sand IS 2131 - 1981.

Various corrections for N - values in SPT.

1. Terzaghi and Peck recommended a correction for dilatancy of N as

$$N_e = 15 + (N - 15)/2.$$

2. Gibbs and Holtz recommended correction for overburden pressure not greater than 0.28 N/mm^2 by the equation.

$$N_e = N\left(\frac{350}{P+70}\right)$$

N_e = Corrected value for over burden effect

N = Actual value of blows

P = Effective overburden pressure kN/m^2

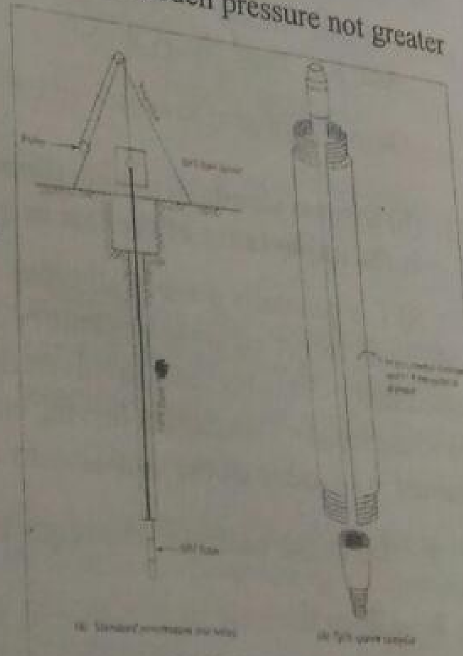


Fig. 1.18. Standard Penetration Test

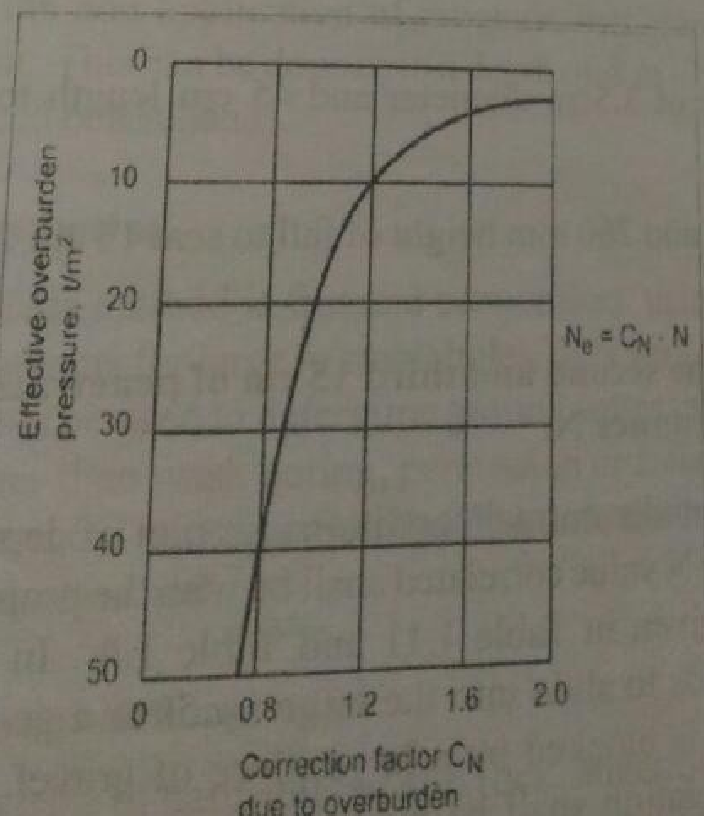


Fig. 1.19. Effect of overburden

Importance of the SPT test in geotechnical engineering:

1. In geotechnical engineering, SPT is the most extensively used penetrometer test.
2. It takes the effect of over burden pressure and the effect of dilatancy.
3. SPT value is easily related to relative density of sandy soil and consistency of clay soil.
4. Out of DCPT (Dynamic cone penetration test), CPT (Static cone penetration test), SPT (Standard penetration test) is more popular in geotechnical engineering.
5. Many Geotechnical engineers validated the SPT for field practices, since 1958 ASTM & IS periodic revision and it is applied currently with latest revision.
6. This test simple and relatively economical.
7. It is the only test that provides representative soil samplers both for vertical inspection in the field and for nature moisture content and classification tests in the laboratory.
8. Because of its wide usage of SPT test, a number of time tested correlations between N-Value and soil parameters are available, mainly for cohesionless soils. Even design charts for shallow foundation resting on cohesionless soil have been developed on the basic of N-Value.

(ii) Explain wash boring method of advancing bore hole. [M/J-16]

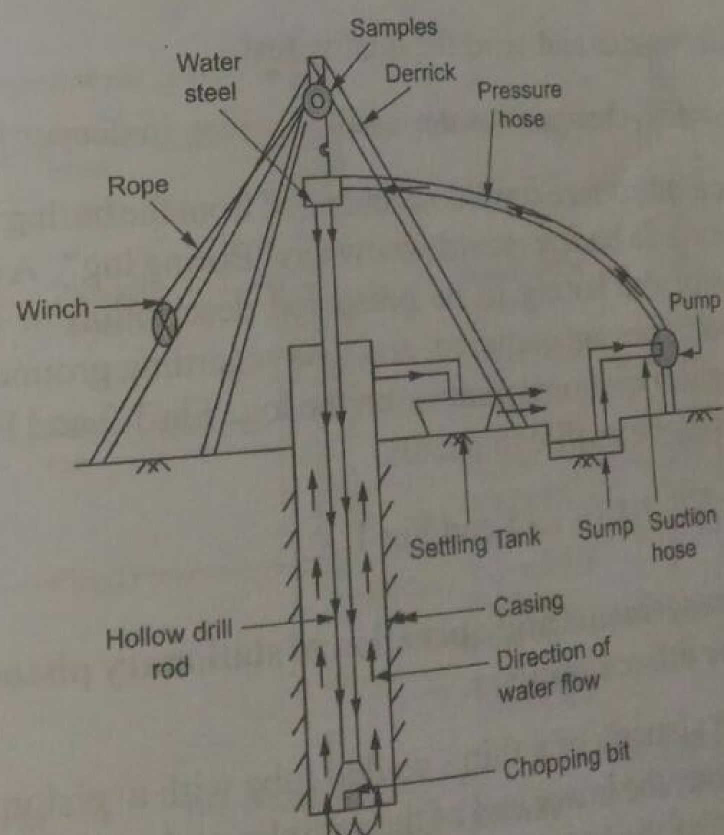
(ii) **Explain wash boring method of advancing bore hole.** S.53

It is a popular method due to the use of limited equipments. The advantage of this is the use of inexpensive and easily portable handling and drilling equipments. Here first an open hole is formed on the ground so that the soil sampling or rock drilling operation can be done below the hole.

The hole is advanced by chopping and twisting action of the light bit. Cutting is done by forced water and water jet under pressure through the rods operated inside the hole. (Fig A.1).

In India the "**Dheki**" operation is used. i.e., a pipe of 50 mm diameter is held vertically and filled with water using horizontal lever arrangement and by the process of suction and application of pressure, soil slurry comes out of the tube and pipe goes down. This can be done upto a depth of 8 m - 10 m (excluding the depth of hole already formed beforehand).

Just by nothing the change of colour of soil coming out with the change of soil character can be identified by any experienced person. It gives completely disturbed sample and is not suitable for very soft soil, fine to medium grained cohesionless soil and in cemented soil.



4. (b) (i)Explain the arrangements and operation of stationary piston sampler. State its advantages over other samplers. [M/J-16]

(ii) Explain the arrangements and operation of stationary piston sampler. State its advantages over other samplers.

A piston sampler consists of a thin - walled tube with a piston inside Fig. 1.21(a), (b), (c). The piston keeps the lower end of the sampling tube closed when the sampler is lowered to the bottom of the hole. After the sampler has been lowered to the desired depth, the piston is prevented from moving downward by a suitable arrangement, which differs in different types of piston samplers. The thin tube sampler is pushed past the pistons to obtain the sample. The piston remains close contact with the top of the sample.

The presence of the piston prevents rapid squeezing of the soft soils into the tube and reduces the disturbance of the sample. A vacuum is created on the top of the sample, which helps in retaining the sample. During the withdrawal of the sampler, the piston provides protection against the water pressure which otherwise would have occurred on the top of the sample.

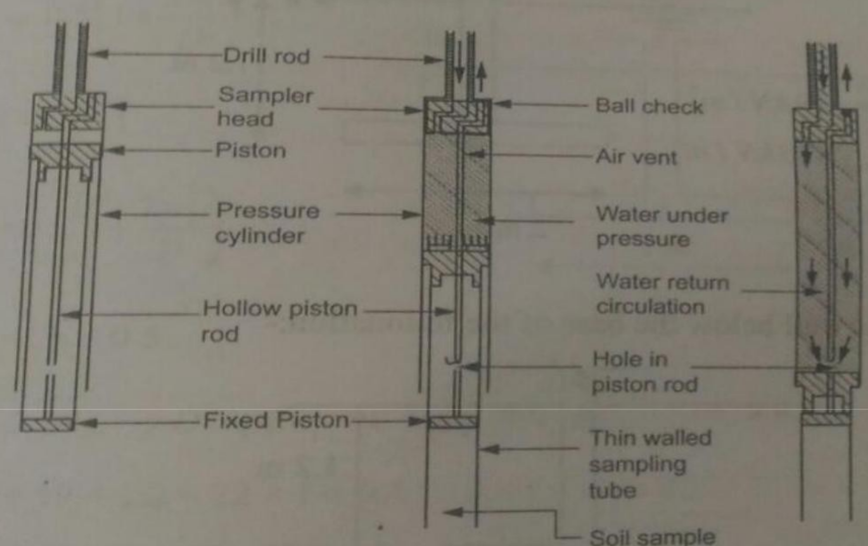
Piston samplers are used for getting undisturbed soil samples from soft and sensitive clays.

For consolidation tests samples of 75 mm or larger diameters are taken. Some soil tends to drop out from the sampler while being withdrawn from the bore hole. In such cases, piston samplers may be used. The principle of this type is illustrated in Fig. 1.21.

The major advantage of the piston sampler is

(1) it is capable of securing samples whereas the open sampler fails to do so.

(2) the sample is less disturbed.



(a) Sampler set in drilled hole *(b) Sampling tube is propelled hydraulically into soil* *(c) Pressure is released through hole in piston rod*

(ii) Explain in detail the salient features of bore log report.

1.11. BORE LOG REPORT

From Fig.1.1 and 1.2 one can observe the following points.

1. Boring, sampling, testing
2. Minimum one sample for 1 m deep and every change of layer.
3. Sample is visually examined and preserved with all details and descriptions for laboratory test and conclusion.
4. Recorded the depth of change of layers.
5. UDS in every layer for medium to stiff clay, for unconfined compression test taken.
6. Rock cores received.
7. Recorded water level, *etc.*
8. Carry out insitu strength test at every 1 m depth and in every distinct layers and record N values of penetration as shown in Fig.1.3.
9. Collect water and send for quality test.
10. Recording change of water quality is seen in deeper level.

Information on subsurface conditions obtained from the boring operation is typically presented in the form of a boring record, known as ***"Boring log"***. A

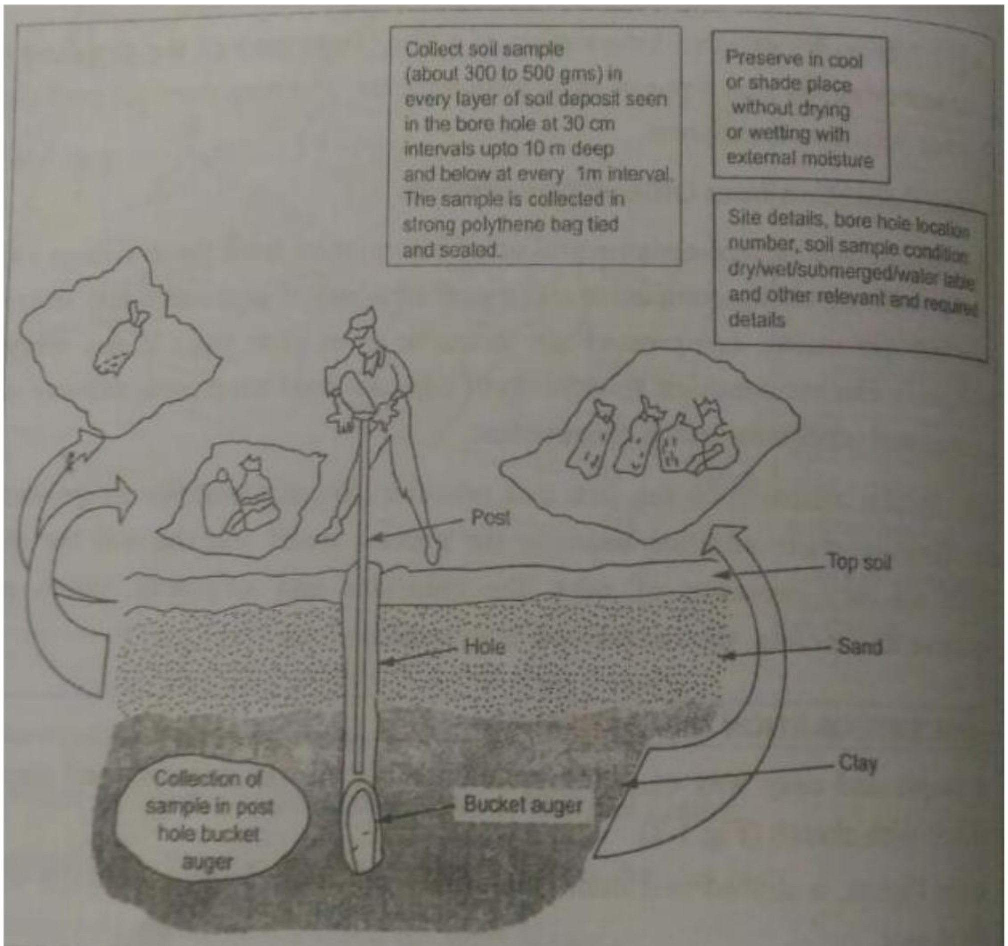


Fig. 1.2. Soil exploration and sampling (Hand auger boring)

(b) Land Survey

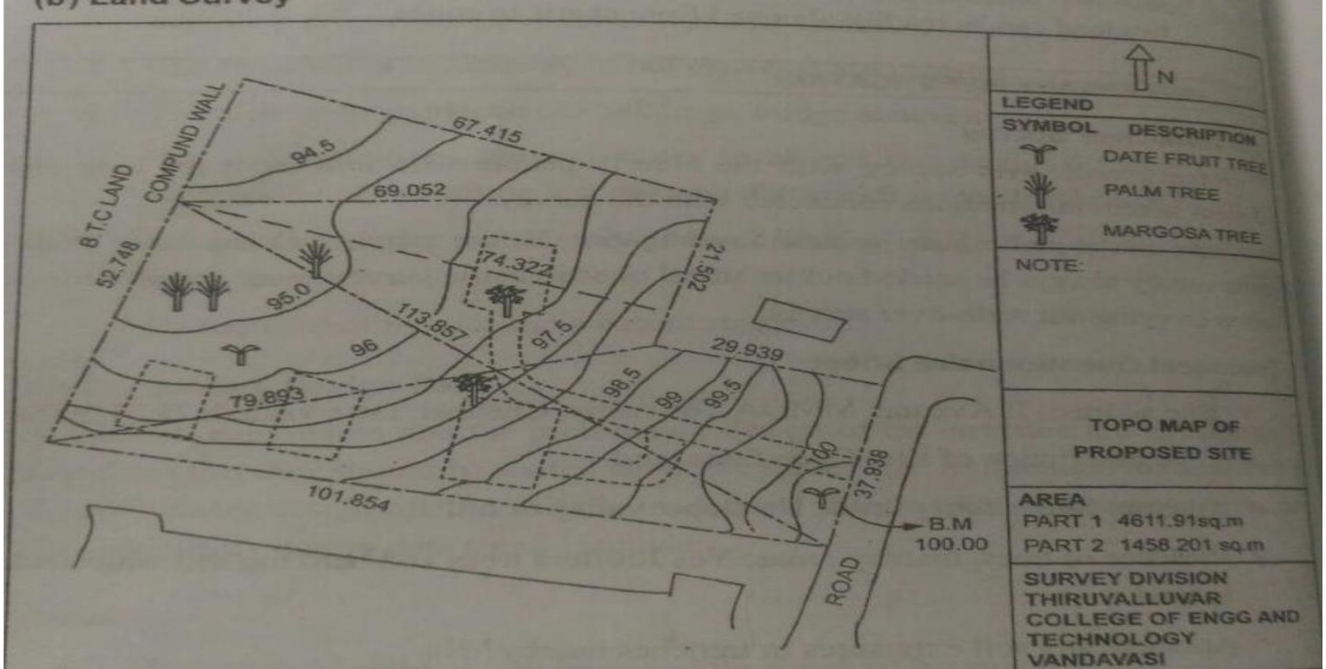


Fig. 1.1. Land Map Survey

continuous record of the various strata identified at various depths of the boring to be presented description or classification of the various soil and rock types encountered, and data regarding ground water level have to be necessarily given in a pictorial manner on the log. Fig.1.3 and Fig.1.4 provides bore log report and sub surface profile in details.

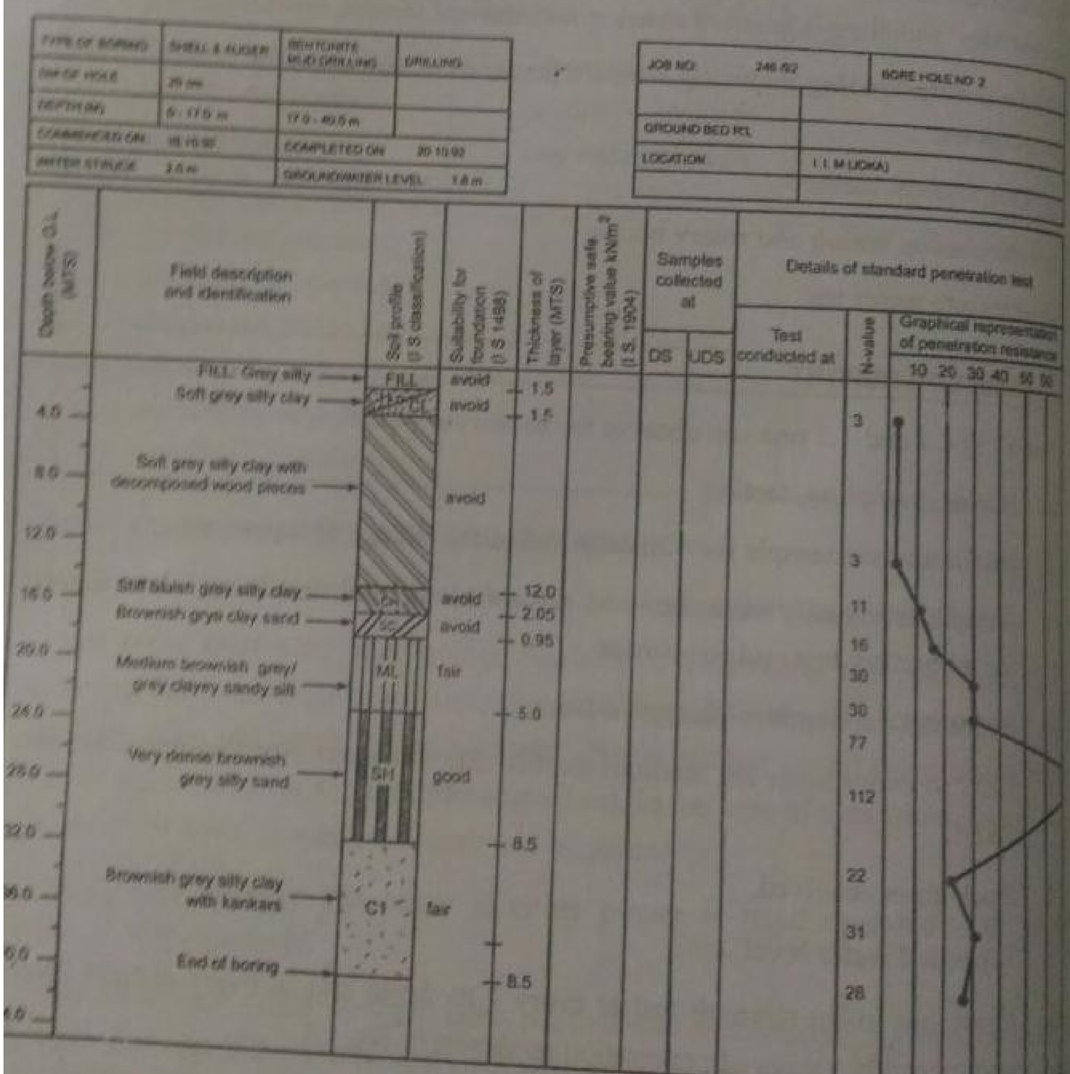


Fig. 1.3. Bore Log Data

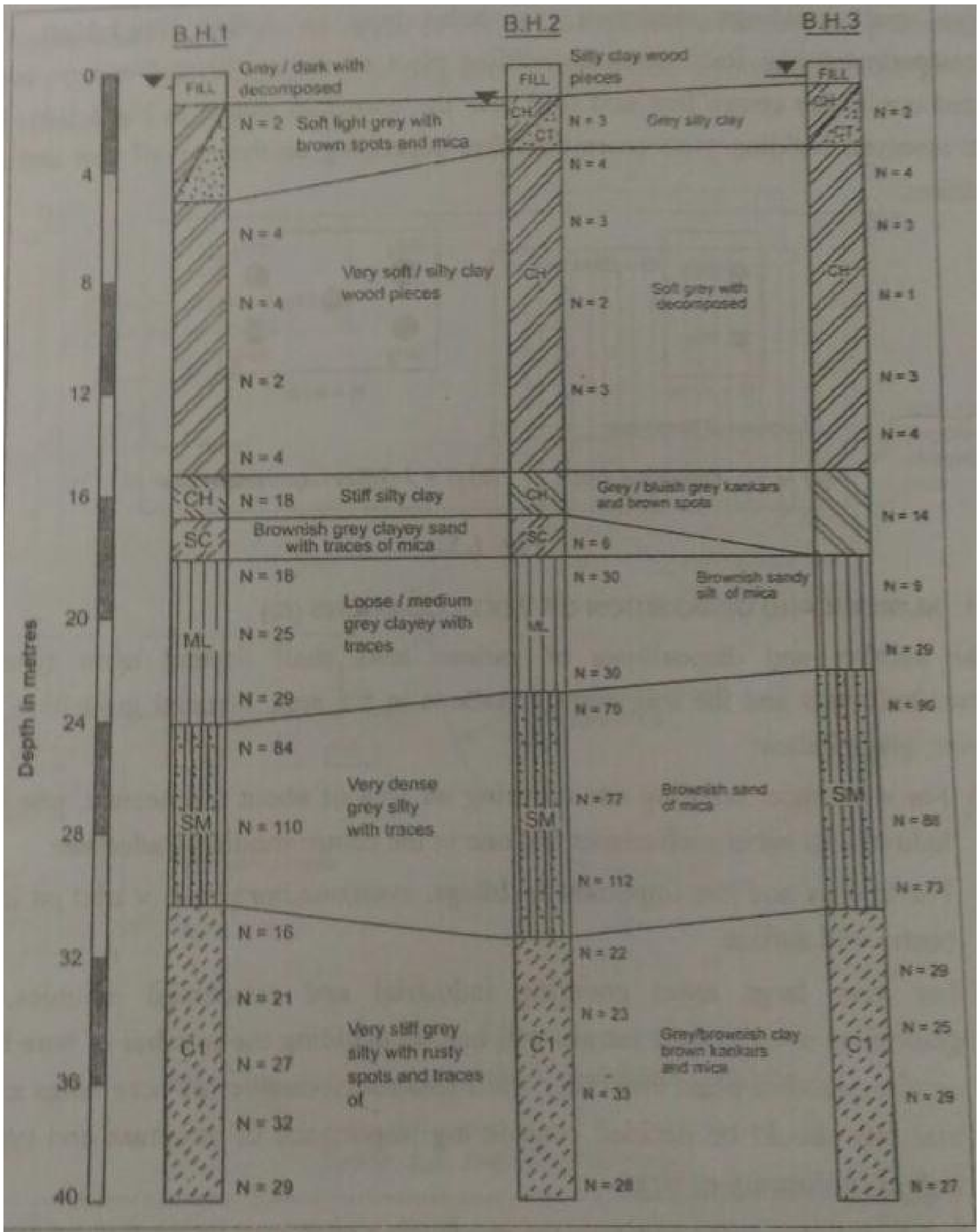


Fig. 1.4. Sub Surface Profile Sections

5. (a) Describe the salient features of a good sub-soil investigation report. [N/D-16]

(a) General

In areas which have already been developed information should be obtained regarding the existing local knowledge; records of trial pits bore holes *etc.* in the vicinity and the behaviour of the existing structures, particularly those of a similar nature to those proposed. This information may be made use of for design of foundation of lightly loaded structures of not more than two storeys and also for deciding scope of further investigation for other structures.

If the existing information is not sufficient or is inconclusive, the site should be explored in detail as per good practice so as to obtain knowledge of the type, uniformity, consistency, thickness, sequence and dip of the strata, hydrology of the area and also the engineering properties. In the case of lightly loaded structures of not more than two storeys, the tests required to obtain the above information are optional,

mainly depending on site conditions. Geological maps of the area give valuable information of the site conditions. The general topography will often give some indications of the soil conditions and their variations. In certain cases the earlier uses of the site may have a very important bearing on the proposed new structures.

(b) Planning an Exploration Program

An engineer planning a soil exploration program for a specific job must (1) have a clear idea of what he is trying to accomplish by the exploration, (2) be well acquainted with current methods and procedures for soil boring, sampling, and testing and (3) keep in mind the relative costs of soil exploration versus the cost of the foundation construction.

The planning of a soil exploration should always start by obtaining preliminary information. For buildings and similar projects, the following information should be obtained first.

- Available information
- Reconnaissance
- Building code for requirements
- Preliminary design data

After this preliminary information is obtained and digested, a tentative exploration program is worked out. The first two or three borings should be scattered around the entire site to disclose the general characteristics of the subsoils. As the boring operation progresses, the balance of the boring program should be constantly revised so that the number and type or types of borings will furnish sufficient data concerning the arrangement of the successive soil strata, and that sufficient number of soil samples are taken for laboratory tests.

There is no hard and fast procedure for planning a boring program. Each condition must be weighed with common sense, good judgement, and relative economy. For example, if the job is small, it may bé more economical to make the foundation design or conservative values rather than making elaborate borings and tests.

(c) Available Information

For large and important projects, the engineer should get the published geological and topographical information before starting the soil exploration.

1. Geologic Map

Rock units are distinguished.

Status Index Maps: A series of maps showing the status of various phases of mapping. Each map is accompanied by a text which gives a detailed explanation.

(*a*) ***Aerial Mosaics of India***, show areas in India for which photomaps have been prepared from aerial photographs and agencies from which copies may be obtained.

(*b*) ***Geologic Mapping of India*** shows by color patterns the areas covered by published geologic maps.

(*c*) ***Topographic Mapping of India*** provides an index to topographic mapping in each state. On a base map the available quadrangles of topographs are shown.

(*d*) ***State Geological Index Maps*** are available for almost all of the states. Each published geologic map is outlined on a state base map an explanatory key gives the source of publication.

2. State Geologic Survey: Most of the states have a geological survey or similar agencies that can supply information on availability of geologic maps and other references.

3. Soil Survey Section of the Bureau of Plant Industry, Department of Agriculture. The Agriculture Year Book has an abundance of useful data. Areas which are not covered by these maps have often been mapped by individual farm maps. These maps indicate the soil type and series which can be invaluable aid for furnishing ground information. The regional soil scientists usually can furnish with soil profile descriptions, soil keys, nomenclature, and the type of parent material associated with the various soil series mapped in his region. The Highway Research Board has published several bulletins concerning the available information.

4. Hydrological Data: Army Engineers map of areas and waterways; information regarding river and tidal levels; stream flow data and maximum flood levels.

5. Soils Manual: Several state highway departments have published such manuals.

6. The Origin, Distribution and Airphoto Identification of soils from soil maps of India.

(d) Reconnaissance

The engineer should always inspect the site to obtain the following data before actual exploration starts:

1. The general topographical characteristics-site on top of a hill on a bluff, in a valley, on an abandoned lot with debris, *etc.*
2. The type of construction and conditions of the existing structures in the adjoining properties. Look for settlements cracks on exterior walls.
3. The soil profiles in highway or rail road cuts and quarries.
4. The high water marks on old buildings, bridge abutments, *etc.*
5. The soil conditions, ground water level and the depth of rocks. General information of this nature can often be obtained from the local people.
6. The depth of scour and history of flood levels (bridge foundations) from the local people.
7. Photographs of the site and adjacent structures.

(e) Preliminary Design Data

The soil exploration and the preliminary design of the structure are so intimately associated that they should be started about the same time. Exploration made ahead of the preliminary design often results in inadequate information or unnecessary waste. The preliminary design data should include:

Buildings

1. The size and height of buildings and the depth of basement.
2. The approximate arrangement of columns and bearing walls.
3. The approximate range of column and wall loads.
4. The type of framing-simple span structures, continuous or rigid frame structures, arches, shell structures, foundations for precision machinery *etc.*
5. The type of exterior walls-brick and glass are sensitive to settlement where metal panels and sidings are more flexible.

Bridges

1. The type and length of bridge span.
2. The approximate vertical and horizontal loads on the piers and abutments. For shorter span bridges the dead load can be estimated from typical design drawings (e.g. Standard plans for highway Bridge superstructures by Bureau of Public Roads, and Publications by some state highways departments). The live load can be readily obtained from charts in codes.

1.8. PRELIMINARY INVESTIGATION

(a) Walk-Over Survey

Have a walk over survey with the elderly people who live there for long and collect salient informations connected with the survey.

...d investigation before purchasing the land. Walk-

6. (b) Explain any two methods of site exploration in detail. [N/D-16]

SAME AS QUESTION 1

UNIT II

Part A (2Marks)

1. **What is shallow foundation?**

If the depth of the foundation is less than its breadth, such foundation is known as shallow foundation.

2. **What are the factors to be considered while designing the foundation?**

Stability of the foundation against shear failure
Settlement within tolerable limit
Location and depth of foundation

3. **Define Bearing capacity and Ultimate bearing capacity. (M/J 08)**

The supporting power of the soil or rock is referred as its bearing capacity.
Gross pressure at the base of the foundation at which the soil fails in shear is called ultimate bearing capacity.

4. **Define Net ultimate bearing capacity and Net safe bearing capacity.**

Net ultimate bearing capacity is the minimum net pressure intensity causing shear failure of soil.

$$q_{nd} = q_d - \gamma D$$

The net maximum pressure which the soil can carry safely without the risk of shear failure is net safe bearing capacity.

$$q_{ns} = \frac{q_{nd}}{FOS}$$

5. **Define Safe bearing capacity and Allowable bearing pressure.**

Allowable bearing pressure is the net loading intensity at which neither the soil fails in shear nor there is excessive settlement detrimental to the structure.
The maximum pressure which the soil can carry safely without the risk of shear failure is safe bearing capacity.

6. **What are the zones used in the Terzaghi's bearing capacity analysis for dividing the failure envelope of the soil? (M/J 08)**

Elastic equilibrium zone
Radial Stress zone
plastic zone

7. **State the different modes of shear failure.**

General shear failure, local shear failure, punching shear failure

8. **In what way the local shear failure differs from General shear failure. (M/J 08)**

General shear failure	Local shear failure
Well defined failure surface reaching up to ground	Failure surface do not reach the ground

Failure is sudden	Failure is not sudden and characterized by large settlements
Tilting	No tilting
Ultimate bearing capacity is well defined	Ultimate bearing capacity is not well defined

9. How the effective dimensions can be calculated in an eccentrically loaded footing?

$$L' = L - 2e_y$$

$$B' = B - 2e_x$$

Where e_x and e_y are the eccentricities in the direction of axes

10. What are the Assumptions made in Terzaghi's Analysis? (N/D-15)

- The base of the footing is rough
- The load on footing is vertical and uniformly distributed
- The base of the footing is laid at shallow depth
- The shearing resistance of soil between ground surface and depth of footing is neglected
- The failure is based on general shear failure condition

11. State the Limitations of Terzaghi's Analysis. (M/J 08)

It does not take in to account the inclination and eccentricity of load on the bearing capacity.

12. State the factors affecting Bearing capacity. (M/J 08)

- Width of footing
- Depth of footing
- Unit weight of soil
- Location of ground water table

  Angle of shearing resistance

13. What is the allowable maximum settlement of commercial, Industrial and ware house building? [N/D-15]

Type of soil	Permissible Settlement		Permissible Differential Settlement	
	Isolated footing	Raft footing	Isolated footing	Raft footing
Sandy Clays	4.0 cm 6.5 cm	4 to 6.5 cm 6.5 to 10 cm	2.5 cm 4.0 cm	2.5 cm 4.0 cm

14. What is the ultimate bearing capacity of a circular footing of 1.5 m diameter resting on the surface of a saturated clay of unconfined compressive strength of 100 kN/m^2. Take N_c= 5.7, N_q=1, N_r=0, c=r, D=O. [N/D-15]

Strip Footing

$$q_n = cN_c + \gamma D(N_q - 1) + 0.5\gamma BN_\gamma$$

q_n = 2148.33 kPa

Square Footing

$$q_n = 1.3cN_c + \gamma D(N_q - 1) + 0.4\gamma BN_\gamma$$

q_n = 1994.43 kPa

15. What are the modes of failure of shallow foundations? [M/J-16]

1. General shear failure
2. Local shear failure
3. Punching shear failure

16. List various methods of minimizing total and differential settlement [M/J-16]

1. Correction for the effect of three dimensional consolidation
2. Correction for the rigidity of foundation.
3. Correction for the depth of embedment.

17. Define net pressure intensity. [N/D-16]

Net ultimate bearing capacity (qnf)

It is the minimum net pressure intensity causing shear failure of soil.

qnf = qf – γ. Df

Net safe bearing capacity (qns)

The Net safe bearing capacity is the net ultimate bearing capacity divided by the factor of safety.

qns = (qf – γ. Df)/ F

18. List out the methods of computing elastic settlements. [N/D-16]

The term settlement is used to describe the vertical displacement of the base of a structure. The effects of settlement depend only on its magnitude but also on its degree of uniformity and on the nature of the engineering works affected.

There are three components of settlements. They are:

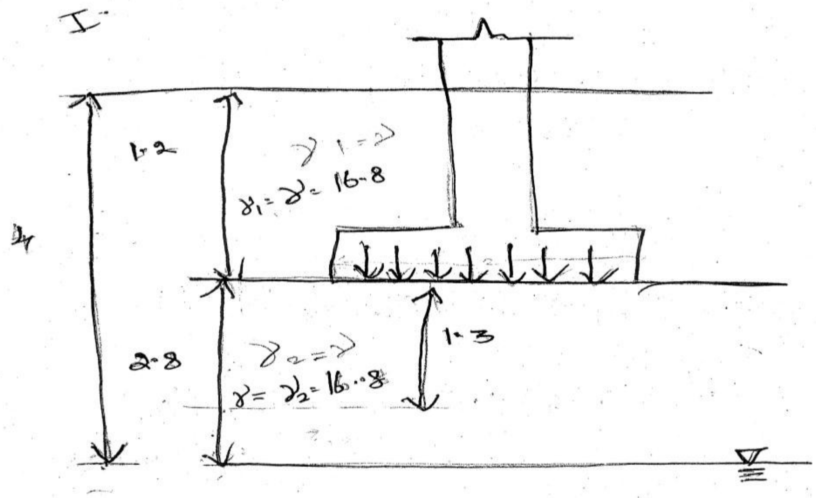
1. Immediate settlement, Si
2. Primary consolidation settlement, Sc
3. Secondary compression settlement, Ss

The total settlement is the sum of these three which may be written as S = Si + Sc + Ss

1. (a) A strip footing 2 n wide carries a load intensity of 560 kN/m2 at a depth of 1.2 m in sand. The saturated unit weight of sand is 18 kN/m^3 and unit weight have a water table is 16.8 kN/m^3.The shear strength parameters are C = 0 and Ø= 35^0. Determine the factor safety with respect to shear failure for the following cares of e location of water table.

i) Water table is 3 m below ground level
(ii) Water table is at G. L itself level
(iii) Water table is 4 m below ground level
iv) Water table is 0.5 m below level [N/D-15]

I.



$R_{w_1} = 1$

$$q_f = \overset{0}{\cancel{C N_c}} + \gamma D N_q + 0.5 \gamma B N_\gamma$$

$$q_f = \gamma_1 D N_q R_{w_1} + 0.5 \gamma_2 B N_\gamma R_{w_2}.$$

$$F = \frac{q_{nf}}{q_{ns}} \quad (\text{or}) \quad \frac{q_f}{q_c}$$

q_c - load intensity

$$q_f = 16.8(1.2)(41.4)(1) + 0.5(16.8)(2)(42.4)$$

$$= 834.624(1) + 712.32$$

$$= 1546.94 \text{ kN/m}^2$$

$$F = \frac{1546.94}{400} = 3.86$$

II : Water table is at the base of footing.

$q_f = \gamma_1 D N_q \cdot R_{w1} + 0.5 \gamma_{sat} B \cdot N_\gamma R_{W2}$

$= 16.8 (1.2)(41.4)(1) + 0.5 (19.5)(2)(42.4)(0.5)$

$q_f = 1248.024$

$F = \frac{1248.024}{400} = 3.12$.

III Water table is 0.5 m below G.L.

$q_f = \gamma_1 D N_q \; R_{w1} + 0.5 (\gamma_{sat}) B N_\gamma R_{w2}$

$= 16.8 (1.2)(41.4)(1) + 0.5 (\overset{17.745}{\cancel{19.5}})(2)(42.4)(0.825)$

$R_{w1} = 1$ & $R_{w2} = 0.5\left(1 + \frac{1.3}{2}\right)$

$0.5\left(1+\frac{1.3}{1.2}\right)$

$= 0.825$

$q_f = \cancel{1516.734}$ 1455.849

$F = \dfrac{q_f}{q_c} = 3.64$

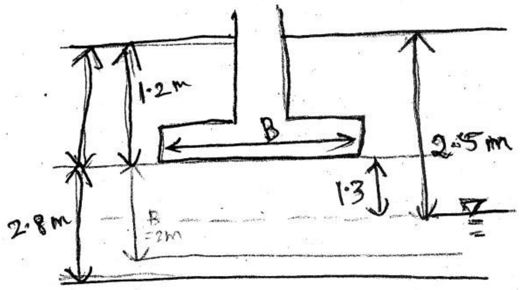
$\gamma_{av} = \dfrac{1.3 \times 16.8 + 0.7 \times 19.5}{1.3 + 0.7}$

$= 17.745$

(iv) Water table is 2.5 m below g.L.

$q_f = \gamma D N_q R_{w_1} + 0.5$

$\frac{0.7}{\frac{0.5}{2}}$



$\gamma_{av} = \dfrac{16.8 \times 0.5 + 19.5 \times 0.7}{0.5 + 0.7}$

$\gamma_{av} = 18.375$

$R_{w_1} = 0.5\left(1 + \frac{0.5}{1.2}\right)$

$= 0.708$

$R_{w_2} = 0.5$

$q_f = \gamma_{av} \cdot D \cdot N_q \cdot R_{w_1} + 0.5\, \gamma_{sat}\, B \cdot N_\gamma \cdot R_{w_2}$

$= 18.375 (1.2)(41.4)(0.708)$

$+ 0.5 (19.5)(2)(42.4)(0.5)$

$q_f = 1059.7$

$\therefore F = 2.649 \simeq 2.65$

(V) Water table @ GL.

$q_f = \gamma_{sat} D N_q R_{w_1} + 0.5 \gamma_{sat} B N_\gamma R_{w_2}$

$= 19.5(1.2)(41.4)(0.5) + 0.5(19.5)(2)(42.4)(0.5)$

$= 897.78$

$F = q_f / q_c = 2.24$

% Reduction $\Rightarrow \dfrac{1546.9 - 1248.024}{1546.9} \times 100$

$= 19.32\%$

2. Explain in detail about IS code method for computing the bearing capacity of soil with various types of failure and sha`pe factor. [N/D-15]

8.2 Extract from IS 1904 -1986 : General Requirements for Design & Construction of Foundation

IS 1904-1986 presents Table 1 which gives details about the permissible settlement in steel structures, reinforced concrete structures, multi-storeyed buildings and water towers and silos in two different types of soils, namely (1) Sand and hard clay and (2) Plastic clay. The settlements considered are maximum settlement, differential settlement and angular distortion or tilt. The details in this table can be followed in the absence of more precise settlement suggested by the user. In case of multi storeyed buildings both RC frames and load bearing wall structures are considered. Load bearing structures with L/H 2 and 7 are dealt with. Two types of foundations considered are isolated footing and raft foundation. Table 8.2 gives the extract of IS code and Table 8.3 presents the same table in different form for steel and RC structures. A maximum settlement of 75 mm, differential settlement of 0.0015L and angular distortion of 1 in 666 is permitted for isolated footings.

Table 3.2 : Permissible uniform and differential settlement and tilt for shallow foundations

TABLE 1 PERMISSIBLE DIFFERENTIAL SETTLEMENTS AND TILT (ANGULAR DISTORTION) FOR SHALLOW FOUNDATION IN SOILS

(*Clause* 16.3.4)

Sl No.	Type of Structure	Isolated Foundations: Sand and Hard Clay, Maximum settlement	Isolated Foundations: Sand and Hard Clay, Differential settlement	Isolated Foundations: Sand and Hard Clay, Angular distortion	Isolated Foundations: Plastic Clay, Maximum settlement	Isolated Foundations: Plastic Clay, Differential settlement	Isolated Foundations: Plastic Clay, Angular distortion	Raft Foundations: Sand and Hard Clay, Maximum settlement	Raft Foundations: Sand and Hard Clay, Differential settlement	Raft Foundations: Sand and Hard Clay, Angular distortion	Raft Foundations: Plastic Clay, Maximum settlement	Raft Foundations: Plastic Clay, Differential settlement	Raft Foundations: Plastic Clay, Angular distortion
		mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
i)	For steel structure	50	·0033L	1/300	50	·0033L	1/300	75	·0033L	1/300	100	·0033L	1/300
ii)	For reinforced concrete structures	50	·0015L	1/666	75	·0015L	1/666	75	·0021L	1/500	100	·002L	1/500
iii)	For multistoreyed buildings												
	a) RC or steel framed buildings with panel walls	60	·002L	1/500	75	·002L	1/500	75	0·0025L	1/400	125	0·0033L	1/300
	b) For load bearing walls												
	1) L/H = 2*	60	·0002L	1/5000	60	·0002L	1/5000	Not likely to be encountered					
	2) L/H = 7*	60	·0004L	1/2500	60	·0004L	1/2500						
iv)	For water towers and silos	50	·0015L	1/666	75	·0015L	1/666	100	·0025L	1/400	125	·0025L	1/400

NOTE — The values given in the table may be taken only as a guide and the permissible total settlement/different settlement and tilt (angular distortion) in each case should be decided as per requirements of the designer.

L denotes the length of deflected part of wall/raft or centre-to-centre distance between columns.

H denotes the height of wall from foundation footing.

*For intermediate ratios of L/H, the values can be interpolated.

Table 3.3 : Permissible uniform and differential settlement and tilt for shallow foundations

	Sand & Hard Clay			Plastic Clay		
	Max. Settlement	Diff. Settlement	Angular distortion	Max. Settlement	Diff. Settlement	Angular distortion
Isolated foundation **i) Steel** **ii) RCC**	50mm 50mm	0.0033L 0.0015L	1/300 1/666	50mm 75mm	0.0033L 0.0015L	1/300 1/666
Raft foundation **i) Steel** **ii) RCC**	75mm 75mm	0.0033L 0.002L	1/300 1/500	100mm 100mm	0.0033L 0.002L	1/300 1/500

Table 5.4 : Limiting Values of movement for Geotechnical Structures

Design Application	Parameter	Typical Movement
Shallow Foundation	Allowable Bearing Pressure	25 mm for buildings
Deep Foundation	Skin Friction	10 mm for skin friction to mobilize
Retaining Wall	Active & Passive earth pressure	0.1% H for Ka & 1% H for Kp to mobilize in dense sand
Reinforced Earth wall	Friction & Dilatncy to load transfer in soil & reinforcement	25 to 50 mm for geogrid 50 to 100 mm for geotextile
Pavement	Rut depth based on strain due to no. of repetitions	20 mm rut depth in major roads & 100 mm rut depth in minor roads
Embankment	Self weight settlement	0.1% height of embankment
Drainage	Total settlement	100 to 500 mm

Total Settlement

Total foundation settlement can be divided into three different components, namely Immediate or elastic settlement, consolidation settlement and secondary or creep settlement as given below.

$$S = S_I + S_C + S_S$$

Here, S = Total Settlement
S_I = Immediate / Elastic Settlement
S_C = Consolidation Settlement
S_S = Secondary Settlement

Immediate Settlement of Cohesive Soils

- Immediate settlement is also called elastic settlement.
- It is determined from elastic theory.
- It occurs in all types of soil due to elastic compression.
- It occurs immediately after the application of load
- It depends on the elastic properties of foundation soil, rigidity, size and shape of foundation.
-

Immediate settlement is calculated by the equation mentioned below.

$$S_I = \left(\frac{1-\mu^2}{E}\right) qBI_\rho$$

Here,

S_I = Immediate settlement
μ = Poisson's Ratio of foundation soil
E = Young's modulus of Foundation Soil
q = Contact pressure at the base of foundation
B = Width of foundation
I_ρ = Influence Factor

Table 8.5 presents the typical values of Poisson's ratio in different soils. Table 8.6 represents the ranges of soil modulus in clayey soil of different consistencies in undrained state. In the absence of more accurate data, the values in tables can be used. The influence factor I_ρ depends on the shape and flexibility of footing. Further, in flexible footing I_ρ is not constant. Table ?.7 presents the different values of I_ρ.

Table ?.5 : Typical Range of Poisson's Ratio for different soils

Type of Soil	Poisson's Ratio
Saturated Clay	0.5
Sandy Clay	0.3 – 0.4
Unsaturated Clay	0.35 – 0.4
Loess	0.44
Silt	0.3 – 0.35
Sand	0.15 – 0.3

Rock	0.1 – 0.4

Table 3.6 : Typical Range of Soil Modulus in undrained state

Soil Type	Soil Modulus (kPa)
Very Soft Clay	400 – 3000
Soft Clay	1500 – 4000
Medium Clay	3000 – 8500
Hard Clay	7000 – 17000
Sandy Clay	28000 – 42000

Table 3.7 : Typical Values of Influence Factors I_ρ

Shape of Footing	Flexible			Rigid
	Center	Corner	Mean	
Circle	1.00	0.64	0.85	0.80
Rectangle L/B = 1	1.12	0.56	0.95	0.90
Rectangle L/B = 1.5	1.36	0.68	1.20	1.09
Rectangle L/B = 2	1.52	0.77	1.31	1.22
Rectangle L/B = 5	2.10	1.05	1.83	1.68
Rectangle L/B = 10	2.52	1.26	2.25	2.02
Rectangle L/B = 100	3.38	1.69	2.96	2.70

3. Determine the ultimate bearing capacity of a strip footing, 1.5 m wide, with its base at a d stratum.

Take Υ= 17 kN/m^3; φ= 38°; Use IS code method. For φ = 38°, N_q= 48.9 and N_γ =56.2.
The following data was obtained from a plate load test carried out on a 60 cm square test plate at a depth of 2 m below ground surface on a sandy soil which extends upto a large depth. Determine the settlement of a foundation 3.0 m x 3.0 m carrying a load of 1100 kN and located at a depth of 2 m below ground surface.

Load intensity,	50	100	150	200	250	300	350	400
Settlement, mm	2.0	4.0	7.5	11.0	16.3	23.5	34.0	45.0

[M/J-16]

$$C_N = 0.77 \log \frac{2000}{\sigma'}$$

$$\sigma' = \gamma D \Rightarrow 17 \times 1.5 = 25.5$$

$$C_N = 0.77 \log \frac{2000}{25.5}$$

$$= 1.45$$

$$N = C_N N'$$

$$N = 1.45 \times 20$$

$$N = 29.17$$

$$R_{w2} = 0.5\left(1 + \frac{3.5}{1.5}\right)$$

$$\boxed{R_{w2} = 1}$$

$$R_d = 1 + 0.2\,\frac{1.5}{2}$$

$$\boxed{R_d = 1.15}$$

$$= 35(29.17 - 3)\left(\frac{2+0.3}{2 \times 2}\right)^2 \times 1 \times 1.15$$

$$q_{na} = 348.26 \text{ kN/m}^2$$

SCPT Value:

For strip (or) square footing of $B \leq 1.2m$

$$\boxed{q_a = 3.6 q}$$

For strip (or) square footing of $B > 1.2m$

$$\boxed{q_a = 2.1 q_c (1 + 1/B)^2}$$

$q_c \rightarrow kg/cm^2$

$q_a \rightarrow$ allowable bearing capacity

$q_c \rightarrow$ cone penetration resistance

4. (i)A strip footing of 1.5 m wide, resting on a sand stratum with its base at a depth of 1 m. Th kN/m^3, Υ_{sat} = 20 kN/m^3, φ= 38° and c' = 0. Determine the ultimate bearing capacity of the fo ground water table is located at a depth of 0.5m below the base of the footing. For φ= 38°, assum

$N\Upsilon$= 75.

(ii) Find the net allowable load on a square footing of 2.5 m x 2.5 m. The depth of foundation is 2 in and the tolerable settlement is 40 mm. The soil is sandy with Standard Penetration Number of 12. Take a factor of safety of 3. The water table is very deep.

Using Terzaghi's equation, take $N_q = 41.4$ and $N_\gamma = 42.4$. (Feb 2005)

Data

C = 0

$\phi = 35°$

B = 2 m

D = 1.2 m

$\gamma_b = 19.5$ kN/m^3 (bottom)

$\gamma_t = 16.8$ kN/m^3 (top)

$N_c = 0$

$N_q = 41.4$

$N_\gamma = 42.4$

Safe load intensity = 400 kPa

$$q_s = 400 = [cN_c + \gamma D(N_q - 1)R_{W1} + 0.5\gamma BN_\gamma R_{W2}]\frac{1}{F} + \gamma D$$

a. Water table is 4 m below Ground Level

$R_{W1} = R_{W2} = 1$

$\gamma = 16.8$ kN/m^3

F = 4.02

b. Water table is 1.2 m below Ground Level

$R_{W1} = 1$, $R_{W2} = 0.5$

$$400 = [16.8X1.2X40.4X1 + 0.5X19.5X2X42.4X0.5]\frac{1}{F} + 16.8X1.2$$

F = 3.227

c. Water table is 2.5 m below Ground Level

RW2 = 0.5(1+1.3/2) = 0.825

$$\gamma_{eff} = \frac{16.8X1.3 + 19.5X0.7}{2} = 17.745 \text{ kN/m}^3$$

$$400 = [16.8X1.2X40.4X1 + 0.5X17.745X2X42.4X0.825]\frac{1}{F} + 16.8X1.2$$

F = 3.779

d. Water table is at Ground Level

$R_{W1} = R_{W2} = 0.5$

$\gamma = 19.5\ kN/m^3$

$$400 = [19.5X1.2X40.4X0.5 + 0.5X19.5X2X42.4X0.5]\frac{1}{F} + 19.5X1.2$$

F = 2.353

5. Explain Terzaghi's analysis of bearing capacity of soil in general shear failure with assumptions[N/D-16]

TERZAGHI'S BEARING CAPACITY THEORY

Terzaghi (1943) was the first to propose a comprehensive theory for evaluating the safe bearing capacity of shallow foundation with rough base.

Assumptions

1. Soil is homogeneous and Isotropic. 2. The shear strength of soil is represented by Mohr Coulombs Criteria.

3. The footing is of strip footing type with rough base. It is essentially a two dimensional plane strain problem.

4. Elastic zone has straight boundaries inclined at an angle equal to φ to the horizontal.

5. Failure zone is not extended above, beyond the base of the footing. Shear resistance of soil above the base of footing is neglected. 6. Method of superposition is valid.

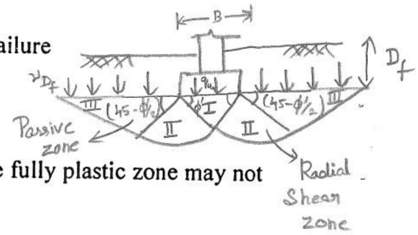
7. Passive pressure force has three components (cohesion, surcharge and weight of shear zone).

8. Effect of water table is neglected. 9. Footing carries concentric and vertical loads.

10. Footing and ground are horizontal.

11. Limit equilibrium is reached simultaneously at all points. Complete shear failure is mobilized at all points at the same time.

12. The properties of foundation soil do not change during the shear failure



Limitations

1. The theory is applicable to shallow foundations

2. As the soil compresses, φ increases which is not considered. Hence fully plastic zone may not develop at the assumed φ.

3. All points need not experience limit equilibrium condition at different loads.

4. Method of superstition is not acceptable in plastic conditions as the ground is near failure zone.

A strip footing of width B gradually compresses the foundation soil underneath due to the vertical load from superstructure. Let q_f be the final load at which the foundation soil experiences failure due to the mobilization of plastic equilibrium. The foundation soil fails along the composite failure surface and the region is divided in to five zones, Zone 1 which is elastic, two numbers of Zone 2 which are the zones of radial shear and two zones of Zone 3 which are the zones of linear shear, Considering the horizontal force equillibrium and incorporating empirical relation, the equation for ultimate bearing capacity is obtained.

6. Explain different types of shear failures of soil with neat sketch.[N/D-16

1. General Shear Failure

This type of failure is seen in dense and stiff soil. The following are some characteristics of general shear failure.

1. Continuous, well defined and distinct failure surface develops between the edge of footing and ground surface.

2. Dense or stiff soil that undergoes low compressibility experiences this failure.

3. Continuous bulging of shear mass adjacent to footing is visible.

4. Failure is accompanied by tilting of footing. 5. Failure is sudden and catastrophic

6. The length of disturbance beyond the edge of footing is large.

7. State of plastic equilibrium is reached initially at the footing edge and spreads gradually downwards and outwards.

8. General shear failure is accompanied by low strain (<5%) in a soil and large N (N > 30) having high relative density (I_D > 70%).

2. Local Shear Failure

This type of failure is seen in relatively loose and soft soil. The following are some characteristics of general shear failure.

1. A significant compression of soil below the footing and partial development of plastic equilibrium is observed. 2. Failure is not sudden and there is no tilting of footing.

3. Failure surface does not reach the ground surface and slight bulging of soil around the footing is observed.

4. Failure surface is not well defined. 5. Failure is characterized by considerable settlement.

6. Well defined peak is absent

7. Local shear failure is accompanied by large strain (> 10 to 20%) in a soil with considerably low φ (φ <28) and low N (N < 5) having low relative density (I_D > 20%).

3. Punching Shear Failure

This type of failure is seen in loose and soft soil and at deeper elevations. The following are some characteristics of general shear failure.

1. This type of failure occurs in a soil of very high compressibility.

2. Failure pattern is not observed.3. Bulging of soil around the footing is absent.

4. Failure is characterized by very large settlement.

]

UNIT III

1. **State the types of shallow foundations**

   Spread footing or pad footings, strap footings, combined footings, raft or mat foundation.

2. **Define spread or Isolated footing**

   It is a type of shallow foundation used to transmit the load of isolated column, or that of wall to sub soil. The base of footing is enlarged and spread to provide individual support for load.

3. **Define combined footing and Raft footing. (M/J 08)**

   Combined footing is a long footing supporting two or more columns in one row.
   Raft foundation is a large footing supporting several columns in two or more rows

4. **Define Strap (or) Cantilever footing.**

   Cantilever or strap footing comprises two footings connected by a beam called strap. It is a special case of combined footing.

5. **Define Raft or mat foundation(M/J 08)**

   It is a combined footing that covers the entire area beneath a structure and supports all the walls and columns.

6. **Define Eccentric loading.**

   When the resultant of loads on a footing does not pass through center of footing, it is called eccentric loading.

7. **What are the circumstances necessitating combined footing (N/D 14)**

   When sub-soil contains compressible pockets, unequally loaded columns and boundary columns, a combined footing is used to control settlement.
   When area covered by isolated footings tends to cover more than half of building area.

8. **Under what circumstances a rectangular and trapezoidal combined footings are adopted(M/J 08)**

   It is preferred when the columns are close to each other and if the soil is of expansive in nature.

9. **Under what circumstances a strap footing is adopted**

   When the distance between the two columns is so great, so that trapezoidal footing is very narrow and so it is uneconomical. It transfers the heavy load of one column to other column.

10. **Where the Raft or Mat Foundation would be used? (N/D 16)**

    It is used when the area of isolated footing is more than fifty percentage of whole area or the soil bearing capacity is very poor.

11. **What is mean by proportioning of footing?**
    Footings are proportional such that the applied load including the self weight of the footing including soil ,the action are not exceeding the safe bearing capacity of the soil.

12. What are the two methods of design of raft foundation as per IS

Rigid method

Elastic plate method

13. List out the types of footing. [N/D-15]

Shallow foundation & Deep foundation

14. Write the components of total settlement? [N/D-15]

There are three components of settlements. They are:

1. Immediate settlement, Si
2. Primary consolidation settlement, Sc
3. Secondary compression settlement, Ss

The total settlement is the sum of these three which may be written as S = Si + Sc + Ss

15. What are the modes of failure of shallow foundations? [M/J-16]

1. General shear failure
2. Local shear failure
3. Punching shear failure

16. List various methods of minimizing total and differential settlement [M/J-16]

- Use raft foundation with a thick slab
- Use deeper underground space to reduce pressure on soil.
- Transfer load to the stronger and not easily compressible soil with the foundation applied such as piles, caisson, etc.
- Method of repair of soil - increase in soil strength

17. Define net pressure intensity. [N/D-16]

Net safe bearing capacity (qns) is the maximum net pressure intensity to which the soil at the base of foundation can be subjected without risk of shear failure.

18. List out the methods of computing elastic settlements. [N/D-16]

- Due elastic deformation of soil grains without any change in moisture content
- It is usually small and occurs directly after the application of a load.
- The magnitude of the contact settlement will depend on the flexibility of the foundation and the type of material on which it is resting, these distributions are true if E is constant with depth.

1. Discuss in detail about the design producer for Rectangular combine footing and Trapezoidal combine footing with suitable sketch.[N/D-15]

Step 1:

Add 10% of column load to get the total load acting on the foundation.

step 2:

Calculate the area of foundation

$$\text{Area of foundation} = \frac{\text{Total load acting on column}}{\text{safe bearing capacity } (q_s)}$$

step 3:

calculate the length of the foundation by using following relation

$$\frac{1}{2}\left[\frac{\text{Area of foundation}}{L} - \text{shorter dimension of the column}\right] = \frac{1}{2}\left[L - \text{longer dimension of the column}\right]$$

step 4:

Calculate the width of the foundation

$$\text{width of foundation} = \frac{\text{Area of foundation}}{\text{length of the foundation}}$$

step 5:

calculate the projected length

$$\text{Projected length} = \frac{\text{width of the foundation} - \text{longer dimension of the column}}{2}$$

step 6:

calculate the Net upward pressure intensity

$$p = \frac{\text{Actual load acting on the column}}{\text{size of the foundation}}$$

step 7:

Calculate the BM along x and y direction

$$M_x = \frac{P_l\,[\text{projected length})^2}{2}$$

$$M_y = \frac{P_B\,[\text{projected length})^2}{2}$$

step 8:

Calculate the depth of the foundation by using the following relation

$$M_x = Q\, b_{long}\, d^2$$

$$M_y = Q\, b_{short}\, d^2$$

Greater value of d is taken on the effective depth

step 9:

Calculate the overall depth

$D = \text{Eff. d} + \text{cover}$ $\boxed{\text{Cover} = 50\,mm}$

step 10:

calculating of area of steel reinforcement

$$A_{st} = \frac{M_x}{\sigma_{st}\, j\, d}$$

$$A_{st} = \frac{M_y}{\sigma_{st}\, j\, d}$$

σ_{st} → Tensile strength of steel depends on the grade of steel

Fe 250 → value of σ_{st} is $140\,N/mm^2$

Fe 415 → σ_{st} $230\,N/mm^2$

j → Lever arm distance (depends on grade of steel)

Fe 250 → 0.87

Fe 415 → 0.9

Step - 11

Check for shear

$$q_v = \frac{S_V}{b'd'}$$

$S_V = P_l$ (projected length - eff. depth)

$$b' = \left[\text{Longer dimension of the column}\right] + \left[\frac{L - \text{longer dimension of coln}}{\text{projected length}} \times d\right]$$

$$d' = D - \left(\frac{D - D_{edge}}{\text{projected length}} \times d\right)$$

$$q_c = 0.16\sqrt{f_{ck}}$$

$q_v < q_c$ (Hence safe)

Design procedure for combined Trapezoidal footing:

step 1:

Calculate the total load acting on the column $Q = Q_1 + Q_2$

step 2:

calculate the area of foundation

$$A = \frac{\text{Total load}}{SBC}$$

step 3:

calculate the centroided distance from any one column

$$\bar{x} = \frac{Q_2 x_2}{Q}$$

step 4:

Calculate the length of foundation

step 5:

Calculate the width of the foundation by using of following relation

$$B_1 = \frac{2A}{L} - B_2$$

$$B_2 = \frac{2A}{L}\left[\frac{3x'}{L} - 1\right]$$

$$x' = \bar{x} + \frac{b_1}{2}$$

step 6:

Calculate the Net upward pressure Intensity

$$P = \frac{T \cdot L}{A}$$

step 7:

Convert Net upward pressure intensity into udl

$$W_1 = B_2 \times P$$

$$W_2 = B_1 \times P$$

Step 8 :

Calculate the load Intensity at different points using following relation

$$w_1 + \left(\frac{w_2 - w_1}{l} \right) x$$

step 9 :

Draw SFD and BMD and obtain the max BM

step 10 :

calculate the eff. depth of the foundation by using

$$M_{max} = Q B d^2$$

$$B = \frac{B_1 + B_2}{2}$$

Step 11

Calculate the area of the steel reinforcement under column A and column B by using the relation

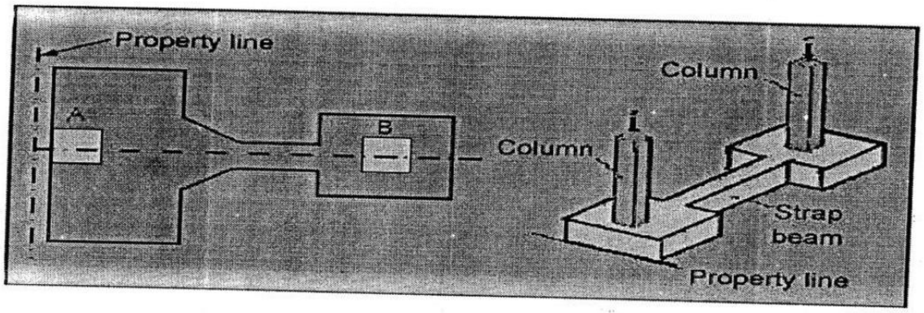
$$A_{st} = \frac{M_A}{\sigma_{st} j d}$$

$$A_{st} = \frac{M_B}{\sigma_{st} j d}$$

2. Write brief notes on:
(1) Mat Foundation
(ii) Floating Foundation
(iii) Seismic force consideration in footing design. [N/D-15]

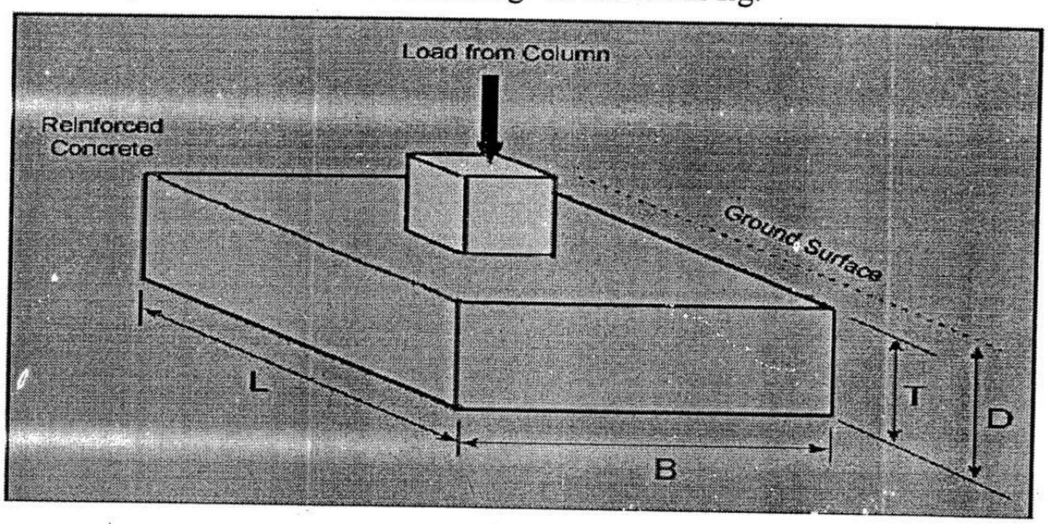
2. Strap Footing:

It consists of two isolated footings connected with a structural strap or a lever, as shown in fig. The strap connects the footing such that they behave as one unit. The strap simply acts as a connecting beam. A strap footing is more economical than a combined footing when the allowable soil pressure is relatively high and distance between the columns is large.

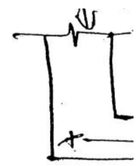


1.Spread Footing:

It is circular, square or rectangular slab of uniform thickness. Sometimes, it is stepped or haunched to spread the load over a larger area. When spread footing is provided to support an individual column, it is called "Isolated footing" as shown in fig.



3. (a) A trapezoidal footing is to be produced to support two square columns of 30 cm and 50 cm sides re the safe bearing capacity of the soil is 400 kN/m^2. The bigger column carries a load of 5000 kN and the suitable size of the footing so that if does not extend beyond the face of the columns
(b)Explain with neat sketch different types of shallow foundations.
[M/J-16]



soln:

(i) Total load Q

$Q = Q_1 + Q_2$

$= 2000 + 1500$

$= 3500$ kN

ii) $A = \frac{T \cdot L}{q_s}$

$A = \frac{3500}{200} = 17.5 m^2$

$= \cancel{1.75 m^2}$

iii) $\bar{x} = \frac{Q_2 x_2}{Q}$

$= \frac{1500 \times 6}{3500} = 2.57 m$

iv) $L = 6 + 0.25 + 0.25$

$= 6.5 m$

v) $B_1 = \frac{2A}{L} - B_2 \quad \neq B_2$

$B_2 = \frac{2A}{L}\left[\frac{3x'}{L} - 1\right]$

$x' = \bar{x} + b'/2$

$x' = 2.57 + \frac{0.5}{2}$

$x' = 2.82 m$

$B_2 = \frac{2 \times 17.5}{6.5}\left[\frac{3 \times 2.82}{6.5} - 1\right]$

$= 1.62$

$B_1 = \frac{2 \times 17.5}{6.5} - 1.62$

$= 3.76 m$

xi) Net upward pressure

$$P = \frac{T \cdot L}{Area}$$

$$= \frac{3500}{17.5} = 200\, kN/m^2$$

$W_1 = B_2 \times P = 1.62 \times 200 = 324\, kN/m$

$W_2 = B_1 \times P = 3.76 \times 200 = 752\, kN/m$

Different of point of load

$$W_1 + \left(\frac{W_2 - W_1}{l}\right)x$$

$$= 324 + \left(\frac{752 - 324}{6.5}\right)0.25$$

$$= 341\, kN/m$$

$$= W_1 + \left(\frac{W_2 - W_1}{l}\right)x$$

$$= 324 + \left(\frac{752 - 324}{6.5}\right)6.25$$

$$= 736\, kN/m$$

∴ SFD & BMD

Area of trapezoidal

$\frac{1}{2}(752 + 736) \times 0.25 = 186\, m^2$

Trapezoidal at x distance

$\frac{1}{2}[324 + 324 + 65.85x]\,x - 1500 = 0$

$324 + 32.92x = 1500$

$324x + 32.92x^2 = 1500$

$x = 3.43\, m$

B.M.D

$$A \times \left(\frac{b+2a}{b+a}\right)\frac{h}{3}$$

$$M_A = 186\left(\frac{752+2\times736}{752+736}\right)\times\frac{0.25}{3}$$

$$= 23.17\ kN\cdot m$$

$$M_B = \tfrac{1}{2}(324+341)0.25\left(\frac{341+2(324)}{341+324}\right)\frac{0.25}{3}$$

$$= 10.30\ kN\cdot m$$

$$M_{max} = \tfrac{1}{2}(550+324)\,3.43\left[\frac{550+2(324)}{550+324}\right]\frac{3.43}{3} - 1500\times3.18$$

$$= -2421\ kN\cdot m$$

$$B = \frac{B_1+B_2}{2} = \frac{1.62+3.76}{2} = 2.69m$$

$$M = Qbd^2 = 2421\times10^6 = 0.652\times2.69\times1000\times d^2$$

$$d = 1174mm$$

$$A_{st} = \frac{M_{max}}{\sigma_{st}\,j\,d} = \frac{2421\times10^6}{230\times0.9\times1174}$$

$$= \cancel{95.34mm^2}\ 9962mm^2$$

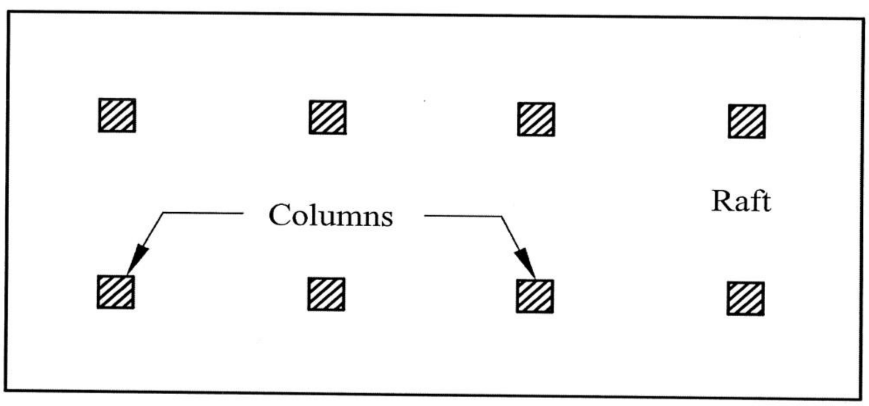
4. (b) (i)Explain the conventional method of proportioning of raft' foundation
(ii)Proportion a rectangular combined footing for two columns 5 m apart. The exterior column of size 0.3 m x 0.3 m carries a load of 600 kN and interior column of size 0.4 m x 0.4 m carries a load of 900 kN. The allowable soil pressure is 100 kN/m^2
[M/J-16]

Raft Foundations

Raft foundations are also called mat foundations. These are combined foundations supporting several columns arranged in one or more rows and columns. These may be necessary in the following situations

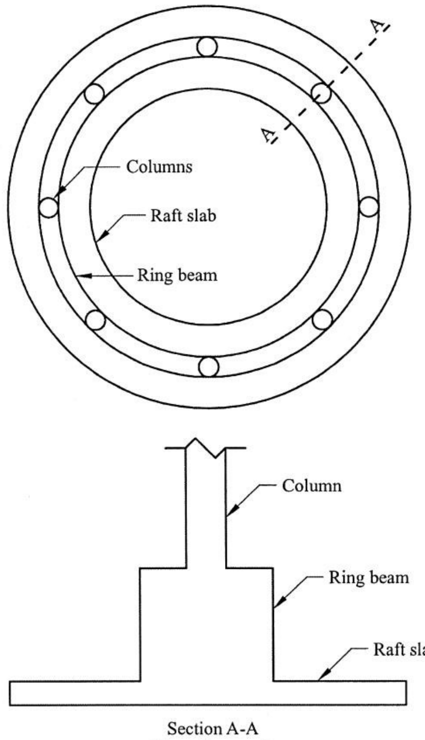
1. Soil strength is very poor.
2. Area of individual footings if provided, exceeds half the total area.
3. To minimize excessive differential settlements.
4. The soil is not uniform.
5. In cases when floating foundations are to be designed.
6. To support machine foundations where differential settlements need to be reduced such as in power generators, bar mills, large tanks and so on.
7. When large hydrostatic pressure is encountered at site, a mat foundation is preferable because of its structural strength and the feasibility of making it watertight.
8. To reduce the settlements and differential settlements of structures to be constructed on soils with high compressibility.

Since rafts are constructed at some depth below ground level, a large volume of excavation may be required. If weight of the excavated soil is equal to the weight of structure and that of the raft, and the centers of gravity of excavation and structure coincide, settlement should be negligible. Such foundations are called floating foundations. Where complete compensation is not feasible, a shallower raft may be acceptable if the net increase in loads is small enough to lead to tolerable settlement. A raft foundation may be rectangular or circular or annular as shown in Figure.

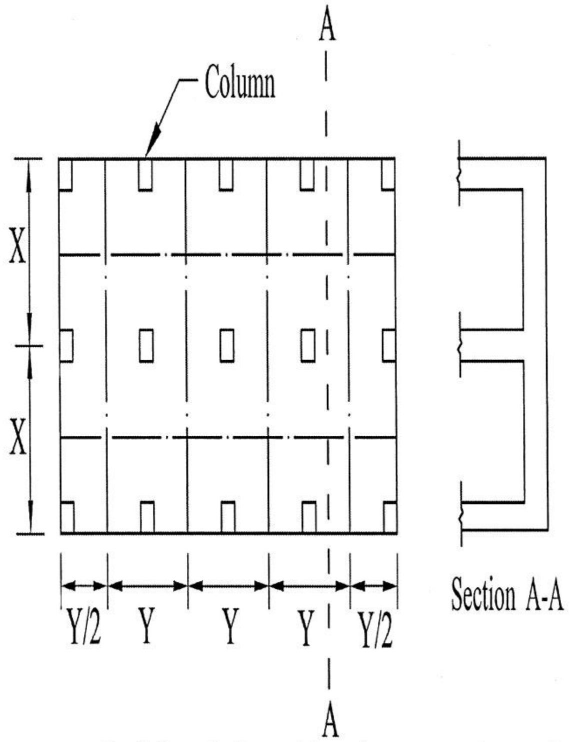


If the columns are equally spaced and loads are not very heavy a raft may be designed as having uniform thickness. The conventional design of such a raft consists of establishing its dimensions using loads and design soil pressure. Then, the soil pressure at various points beneath the slab may be computed. The raft is divided into a series of continuous strips centered on the appropriate column rows in both directions as shown in Figure. The shear and bending moment diagrams may be drawn treating it as a continuous beam. The depth is provided to satisfy bending moment and shear force requirements. The steel requirements will vary from strip to strip. This method generally gives a conservative design since the interaction of adjacent strips is neglected. If the columns are equally spaced and their loads are equal, the pressure on the soil will be uniform, otherwise, moments of the loads may be taken about the center of the

contact area and pressure distribution determined using Equations (1) and (2). These equations are derived assuming the raft to be rigid. However, since a raft in general is not a rigid member, the pressures and resulting internal stresses may be seriously in error if the eccentricity is very large. The weight of the raft is not considered in the structural design because it is assumed to be carried directly by subsoil.



Circular raft

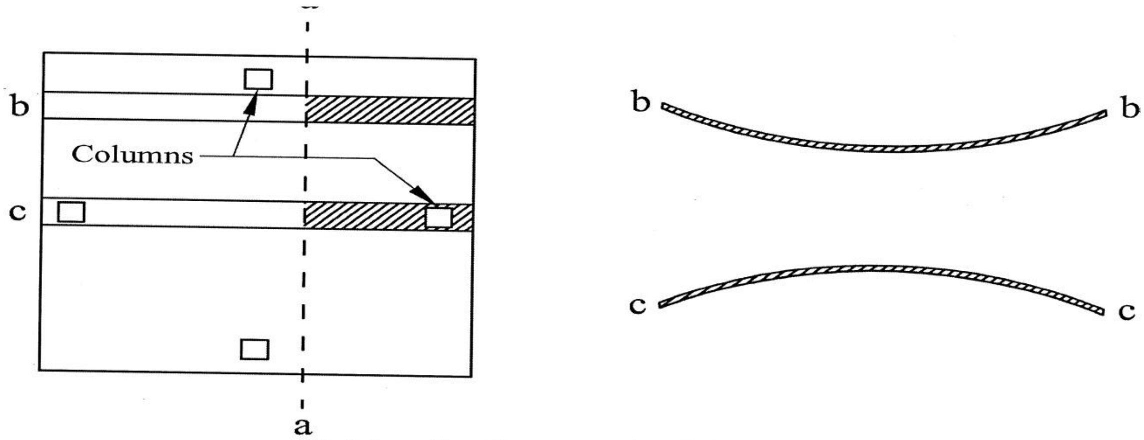


Raft foundation with strips centered on columns.

Conventional Design of Rafts

The mat is analyzed as a whole in each of two perpendicular directions. The total shear force acting on any section of the mat is equal to the sum of all forces and reactions (due to bearing pressure) to the left or right of the section. The total bending moment acting on such section is equal to the sum of all moments to the left, or right, of this section, as per the classical bending theory of beams. Although the total shear forces and moments can be determined by the principles of simple structures, the stress distribution along this section is a problem of highly indeterminate nature.

If the column loads and spacing are almost equal, an approximate idea as to how the moment and shear are distributed along each section may be arrived at. In most cases, however, the variation of moment and shear is often far different from the average value. This point can be illustrated by a simple example shown in Figure . The total bending moment on section a-a is equal to the difference between the positive moment (tension on the bottom of the slab) due to the soil reaction and the negative moment due to the column load. Let us say the net total moment is positive. Then the average bending moment on section a-a is equal to this net moment divided by the length of section a-a. But it is obvious in this case that the strip b is subjected to a positive moment and the strip c is subjected to a negative moment.



Variation of bending moment in raft foundation.

Thus, the average moment is not indicative of the sign and the magnitude of the bending moments in the individual strips. In order to obtain some idea of the upper limit of stresses, each strip bounded by center lines of column bays may be analyzed as independent, continuous, or combined footings. Full column loads are used and the soil reaction under each strip is determined without reference to the planar distribution determined with the mat as a whole. This method undoubtedly gives very high stresses because it ignores the two way action of the mat. Therefore certain arbitrary reductions in stresses (e.g., 15.0, 25.0, or sometimes >33.33%) are used. In the design by conventional method, the column loads, wall loads and allowable bearing first determined. Then a trial size for the mat is assumed. The eccentricity of the centroid of all loads from the columns and walls with reference to the C.G of the mat or raft can now be obtained as ex and ey along the x and y directions. Then, the soil pressure under the mat is determined by the general equation (Equation (12.12))

$$q = \frac{\sum P}{BL}\left(1 \pm 6\frac{e_x}{L} \pm 6\frac{e_y}{B}\right)$$

where

$\sum P$ = sum of vertical loads from all columns and walls.

L = length of the mat (along the x direction)

B = width of the mat (along the y direction)

e_x, e_y = eccentricities of the C.G of loads with respect to the C.G of the contact area of the mat along the x and y directions.

With the soil pressure determined, the mat is analyzed as individual bands along column center lines. In this analysis, a moment coefficient of 1/10 is used.

5. (a)A trapezoidal footing is to be produced to support two square columns of 30 cm and 50 cm sides respectively. Columns are 6 meters apart and the safe bearing capacity of the soil is 400 kNim^2. The bigger column carries a load of 5000 kN and the smaller carries a load of 3000 kN. Design a suitable size of the footing so that if does not extend beyond the face of the columns. [N/D-16]

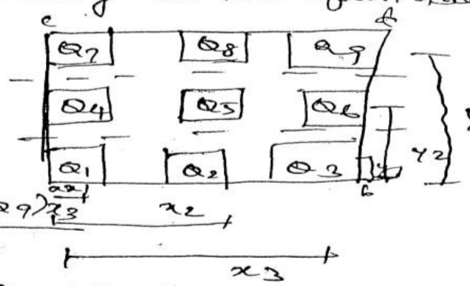
SAME AS QUESTION NO 3.

6. Write the IS codal provisions for design of raft foundation .[N/D-16]

Design procedure for mat (or) Raft foundation

(i) calculate the total load acting on the foundation

$Q = Q_1 + Q_2 + \cdots Q_n$



ii) calculate the centroidal distance $\bar{x}$, $\bar{y}$

$$\bar{x} = \frac{(Q_1 + Q_4 + Q_7)x_1 + (Q_2 + Q_5 + Q_8)x_2 + (Q_3 + Q_6 + Q_9)x_3}{Q}$$

$$\bar{y} = \frac{(Q_1 + Q_2 + Q_3)\,y_1 + (Q_4 + Q_5 + Q_6)y_2 + (Q_7 + Q_8 + Q_9)y_3}{Q}$$

iii) calculate eccentricity along x and y direction

$$e_x = \frac{B}{2} - \bar{x}$$

$$e_y = \frac{L}{2} - \bar{y}$$

v) calculate the Net upward pressure Intensity by considering two cases

Case 1:

If the resultant load acting on the axis of the column $[e = 0]$

$$P = Q/A$$

Case 2:

If the column load subjected to eccentricity along x and y direction

$$P = \frac{Q}{A} - \frac{Q\,e_x\,x}{I_{yy}} - \frac{Q\,e_y\,y}{I_{xx}}$$

$$I_{xx} = \frac{BL^3}{12}, \quad I_{yy} = \frac{LB^3}{12}$$

vi) calculate the Net upward pressure Intensity at different points

P_a, P_b, P_c, P_d

(vi) Divide the section into different strips
"Assume" that Individual section is acting as a seperate beam

(vii) calculate the average pressure intensity along each strip

$$P_{avg} = \frac{P_a + P_b}{2}$$

(viii) Assume width of the strip for considering cases:

case 1:
If the space b/w the column is less than are equal to $\leq 5m$

$$B_1 = 50\% \ (L)$$

case 2:
If the spacing b/w the column is more than 5m

$$B_1 = 50\% \ (L/2)$$

ix) calculate the average load acting on the strip

$$Q_{avg} = \frac{1}{2} \ (\text{downward load} + \text{upward load})$$

$$\text{upward load} = P_{av} \times B_1 \times B$$

x) calculate the modified soil pressure Intensity

$$P_{av} = \frac{Q_{av}}{B_1 B}$$

xi) Calculate the column load modification factor

$$F = \frac{Q_{avg}}{Q_1 + Q_2 + Q_3}$$

ii) calculate the modified column load

$$FQ_1, FQ_2, FQ_3$$

iii) Draw shear force and bending moment diagram for the modified column load

(xiv) convert the Net upward pressure Intensity (modified soil intensity into udl

$$udl = P_{avg} \times B_1$$

(xv) calculate the eff. depth of foundation by equating

$$M_{max} = Q B_1 d^2$$

(xvi) calculate the area of steel reinforcement

$$A_{st} = \frac{M_x}{\sigma_{st} j d}$$

UNIT IV

1. What are the General forms of deep foundation?

Pile foundation

Caisson or well foundation

2. What are the different types of piles according to Material of construction? (M/J 08)

Timber pile, concrete pile, steel pile, composite pile.

3. What are the different types of piles according to its function?

Where the topsoil is soft or too weak to support the superstructure, piles are used to transmit the load to the underlying bedrock- end bearing piles.

If the bedrock is not at a reasonable depth below the ground surface, the load is transferred through friction along the pile shaft, such piles are called friction piles.

Transmission rowers, off--shore platforms are subjected to uplift forces, piles are used to resist it is called uplift or tension pile.

In order to resist horizontal and inclined forces in water and earth retaining structures, batter pile is used.

4. What are the different types of piles according to its method of Installation? (M/J 08)

Based on installation - driven piles and cast-in-situ piles. Driven piles are of concrete, steel or timber. Concrete piles are classified as driven precast concrete piles, driven cast in-situ concrete piles and bored cast in-situ concrete piles.

5. What is the use of batter pile?

The batter piles are used to resist large horizontal forces or inclined forces.

6. What is the need for pressure piles?

These piles are especially suitable for those congested sites where heavy vibrations and noise are not permitted and also where heavy pile driving machinery cannot move in.

7. State any two functions of pile foundation.

- It transfers the load from soft strata to hard soil strata.
- It will resist uplift and inclined forces, hydrostatic pressure.

8. Define pile cap.

Pile cap is a thick concrete mat that rests on concrete or timber piles that has been driven in to soft or unstable ground to provide a suitable stable foundation.

9. When a pile foundation is preferable? (M/J 08)

- When the strata at or just below the ground surface is highly compressible and very weak to support the load transmitted by the structure.
- To transmit the loads of the structure through deep water to a firm stratum.

10. What factors are to be considered in selection of a pile?

Size, weight and type of structure to be supported

Soil profile and properties of soil

Location of Water table and its fluctuations

Type of structure in the vicinity

Depth to which the pile has to be taken

11. What are the methods for estimating the load –carrying capacity of a single pile? (N/D 16)

- dynamic formulae
- static formula
- pile load test
- penetration tests

12. Define Pile load test(M/J 08)

Pile load test is a direct method of determining allowable load on piles. Pile load tests are very reliable for cohesion-less soil but in cohesive soil, data should be used with caution.

13. What are the methods available to determine Load caring capacity of pile? [N/D-15]

* End bearing piles
* Friction pile
* Compaction pile
* Tension pile
* Anchor pile
* Fender pile and dolphins
* Batter pile
* Sheet pile

14. What is under reamed pile? When is it preferred? [M/J-16]

* Under-reamed pile is a special type of bored pile having an increased diameter or bulb at some point in its length, to anchor the foundation in expansive soil subjected to alternate expansion and contraction

15. What are methods to determine the load carrying capacity of a pile? [N/D-16]

* Dynamic formulae
* Static formulae
* Pile load test
* Penetration test

16. What is meant by group settlement ratio? [N/D-16]

The ratio of resisting capacity of a pile group to the sum of individual capacities of piles in the group is called group efficiency

1. (a)Explain in details about the various types of pile foundation with neat sketch and write their functions. [N/D-15]

CLASSIFICATION OF PILES

1. Based on material ▪Timber piles ▪Steel piles ▪Concrete piles ▪Composite piles (steel + concrete)

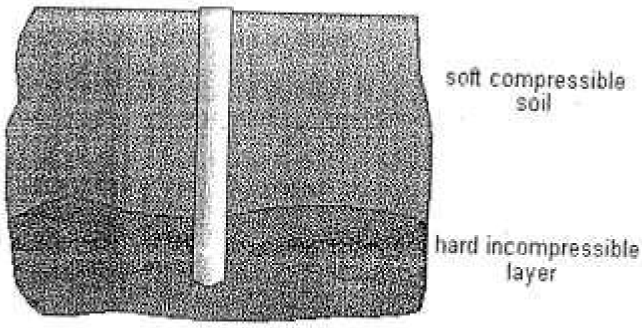
2. Based on method of installation ▪Driven piles ----(i) precast (ii) cast-in-situ. ▪Bored piles. 3. Based on the degree of disturbance ▪Large displacement piles (occurs for driven piles) ▪Small displacement piles (occurs for bored piles)

3. Based on functionality ▪Friction piles. ▪End bearing piles. ▪Compaction piles.(Used for ground movement, not for load bearing) ▪Tension piles/Anchored piles.(To resist upliftment) ▪ Batter piles (Inclined) --- +ve and –ve.

Piles are often used because adequate bearing capacity can not be found at shallow enough depths to support the structural loads. It is important to understand that piles get support from both **end bearing** and **skin friction**. The proportion of carrying capacity generated by either end

bearing or skin friction depends on the soil conditions. Piles can be used to support various different types of structural loads.

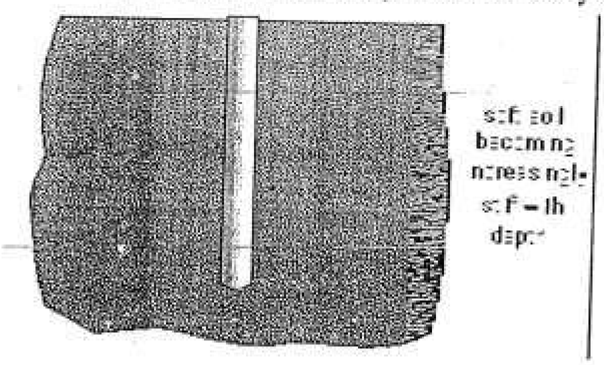
End bearing piles



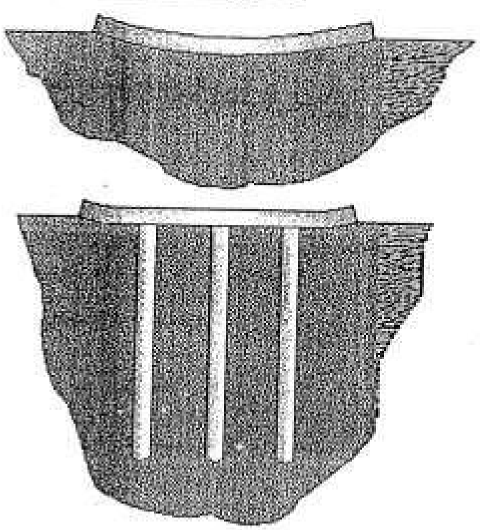
End bearing piles are those which terminate in hard, relatively impenetrable material such as rock or very dense sand and gravel. They derive most of their carrying capacity from the resistance of the stratum at the toe of the pile.

Friction piles

Friction piles obtain a greater part of their carrying capacity by skin friction or adhesion. This tends to occur when piles do not reach an impenetrable stratum but are driven for some distance into a penetrable soil. Their carrying capacity is derived partly from end bearing and partly from skin friction between the embedded surface of the soil and the surrounding soil.



Settlement reducing piles



Settlement reducing piles are usually incorporated beneath the central part of a raft foundation in order to reduce differential settlement to an acceptable level. Such piles act to reinforce the soil beneath the raft and help to prevent dishing of the raft in the centre.

Tension piles

Structures such as tall chimneys, transmission towers and jetties can be subject to large overturning moments and so piles are often used to resist the resulting uplift forces at the foundations. In such cases the resulting forces are transmitted to the soil along the embedded length of the pile. The resisting force can be increased in the case of bored piles by under-reaming. In the design of tension piles the effect of radial contraction of the pile must be taken into account as this can cause about a 10% - 20% reduction in shaft resistance.

Laterally loaded piles

Almost all piled foundations are subjected to at least some degree of horizontal loading. The magnitude of the loads in relation to the applied vertical axial loading will generally be small and no additional design calculations will normally be necessary. However, in the case of wharves and jetties carrying the impact forces of berthing ships, piled foundations to bridge piers, trestles to overhead cranes, tall chimneys and retaining walls, the horizontal component is relatively large and may prove critical in design. Traditionally piles have been installed at an angle to the vertical in such cases, providing sufficient horizontal resistance by virtue of the component of axial capacity of the pile which acts horizontally. However the capacity of a vertical pile to resist loads applied normally to the axis, although significantly smaller than the axial capacity of that pile, may be sufficient to avoid the need for such 'raking' or 'battered' piles which are more expensive to install. When designing piles to take lateral forces it is therefore important to take this into account.

2. Write short notes on:
(i) Negative skin friction
(ii) Under reamed piles
(iii) Piles Cap
(iv) Settlement of pile group in clay [N/D-15]

* Negative skin friction is downward drag acting on a pile due to the downward movement of the surrounding compressible soil relative to the pile

* Under-reamed pile is a special type of bored pile having an increased diameter or bulb at some point in its length, to anchor the foundation in expansive soil subjected to alternate expansion and contraction

* A pile cap is provided over piles to act as a plat form for the superstructure and to transfer the column load to the pile heads

The ratio of resisting capacity of a pile group to the sum of individual capacities of piles in the group is called group efficiency

3. (i)Classify the pile foundation based on (1) method of installation, (2) load transfer mechanism. [M/J-16]

SAME AS QUESTION 1

4. (ii) It is proposed to provide pile foundation for a heavy column; the pile group consisting of 4 piles placed at 2 m center to center, forming a square pattern. The underground soil is clay, having Cu at surface as 60kN/m^2 and at depth 10 in, as 100kN/m^2. Compute the allowable column load on the pile cap, if the piles r having diameters 0.5 m each and length as 10 m. [M/J-16]

Given data:

25 no. of pile

$m = 5;\ n = 5,\ \phi = 40cm \simeq 0.4m$

spacing $= 1.5m$

$\theta = \tan^{-1}(d/s) = 14.93$

Soln:

(i) converse Labarine:

$$\eta_g = \left[1 - \frac{14.93}{90}\left[\frac{(5-1)5 + (5-1)5}{25}\right]\right] \times 100$$

$$\eta_g = 73.45\%$$

ii) Seiler Keeney

$$\eta_g = \left[1 - 0.479\left(\frac{1.5}{1.5^2 - 0.093}\right)\left(\frac{5+5-2}{5+5-1}\right)\right] + \left(\frac{0.3}{0.575}\right) \times 100$$

$$\eta_g = 0.734 \times 100$$

$$= 73.4\%$$

5. (ii) It is proposed to provide pile foundation for a heavy column; the pile group consisting of 4 piles placed at 2 m center to center, forming a square pattern. The underground soil is clay, having Cu at surface as 60kN/m^2 and at depth 10 in, as 100kN/m^2. Compute the allowable column load on the pile cap, if the piles r having diameters 0.5 m each and length as 10 m.[N/D-16]

Given :

$L = 10.5 m$

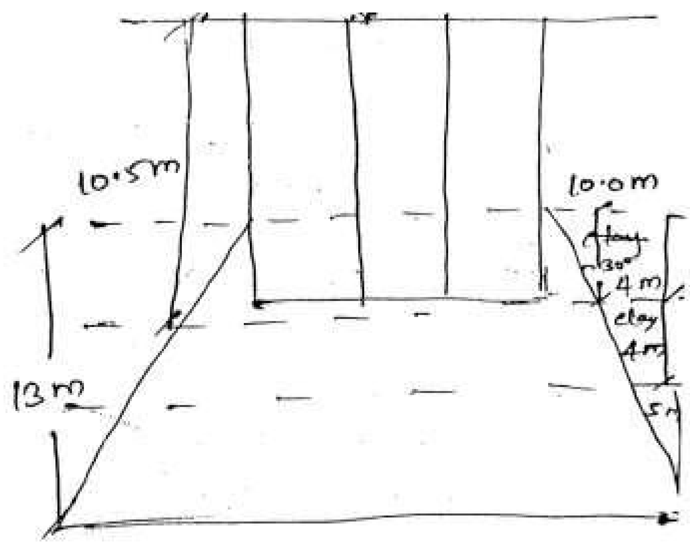
load $= 3000 kN$

depth $= 20m$

$W_L = 60\%$

$e_o = 1.$

$\gamma = 16 kN/m^3$



soln :

$2/3 \quad L$

$2/3 \times 10.5 = 7m$

$C_c = 0.009 (W_L - 10)$

$= 0.009 (60 - 10)$

$= 0.45$

$e_o = 1$

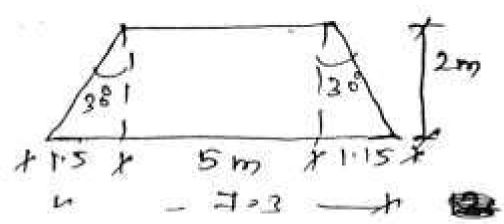
$H = 4m$

Centre of the each layer

eff. stress

$\sigma_o = \gamma z$

$= 16 \times (4/2 + 7)$



Additional stress

$$\Delta\sigma = \frac{Q}{B^2} = \frac{3000}{7.3 \times 7.3} = 56.29$$

$$S_I = \frac{C_c}{1+e_0} 1 + \log\left(\frac{\sigma_0 + \Delta\sigma}{\sigma_0}\right)$$

$$= \frac{0.45}{1+1} \times 4 \times \log\left(\frac{144+56.29}{144}\right)$$

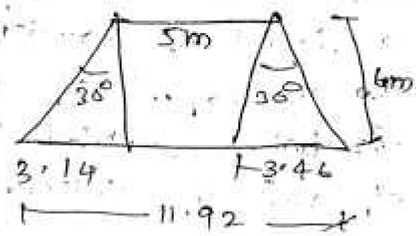
$$= 0.128\,m$$

$$\Delta\sigma = \frac{3000}{11.92 \times 11.92}$$

$$\Delta\sigma = 2681\ KN/m^2$$

$$\sigma_0 = 16 \times (2+4+7)$$

$$= 208\,KN/m^2$$



$$S_{II} = \frac{0.45}{1+1} \times 4 \times \log\left(\frac{208+21.11}{208}\right)$$

$$S_{II} = 0.038$$

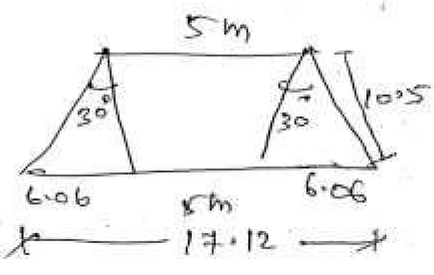
$$\Delta\sigma = \frac{3000}{17.12 \times 17.12}$$

$$= 10.23\,KN/m^2$$

$$\sigma_0 = \gamma z$$

$$= 16\left(\tfrac{5}{2} + 4 + 4 + 7\right)$$

$$= 280\,KN/m^2$$



$$S_{III} = \frac{0.45}{2} \times 5 \times \log\left(\frac{280+10.23}{280}\right)$$

$$= 0.017\,m$$

$$S = S_I + S_{II} + S_{III}$$

$$S = 0.183\,m$$

6.(ii) Explain the method of determining the load carrying capacity of a pile. [N/D-16]

Load Carrying Capacity of pile:

(i) The ultimate bearing resistance of pile divided by suitable factor of safety.

(ii) The permissible settlement

(iii) Overall stability of pile foundation.

Methods of determining load carrying capacity:

(i) Dynamic formulae.

(ii) Static formulae.

(iii) pile load test.

(iv) penetration test.

Dynamic formula: The energy is calculated by weight of hammer & drop of blocks.

Static formula: Depend on type of soil.

$$R = R_f + R_p$$

$R_f \rightarrow$ Frictional resistance

$R_p \rightarrow$ point resistance.

Pile load Test :

- It is similar to plate load test
- It is field test.

DYNAMIC FORMULA :

- The total driving energy is equal to the {Weight of hammer × Height of drop}.
- It is assumed that the soil resistance of dynamic penetration of the pile is the same as to penetration of pile in the static load.

UNIT –V

1. What is alligator crack? [N/D-15]

Fatigue cracking, also called crocodile cracking or alligator cracking, is a common type of distress in asphalt pavement. Fatigue cracking is characterized by interconnecting or interlaced cracking in the asphalt layer resembling the hide of a crocodile

2. What is FWD and state its use. [N/D-15]

A falling weight deflectometer (FWD) is a testing device used by civil engineers to evaluate the physical properties of pavement. FWD data is primarily used to estimate pavement structural capacity for 1) overlay design and 2) to determine if a pavement is being overloaded.

3. What are the parameters that should be observed for evaluating rigid pavements? [A/M- 17]

Sub-grade support
Pavement composition
Thickness of pavement
Traffic loading
Environmental conditions

4. What are the causes of cracks? [A/M- 17, M/J-12]

- Defects in quality of materials
- Defects in construction method and quality control during construction
- Inadequate surface or sub-surface drainage
- Increase in the magnitude of wheel loads and the number of load repetitions due to increase in traffic volume.

5. Give examples for surface defects in pavements? [M/J- 13]

- Alligator (map) cracking
- Consolidation of pavement layers
- Shear failure
- Longitudinal cracking
- Reflection cracking
- Transverse cracking
- Shoving or Formation of waves and corrugations
- Pot holes
- Bleeding

6. What is pavement evaluation? [M/J- 13]

Pavement performance is a function of its relative ability to serve traffic over a period of time (Highway Research Board, 1962). Originally, a pavement's relative ability to serve traffic was determined quite subjectively by visual inspection and experience.

7. Differentiate delamination and depression. [M/J- 16]

Localized pavement surface areas with slightly lower elevations than the surrounding pavement. Depressions are very noticeable after a rain when they fill with water.

Problem : Roughness, depressions filled with substantial water can cause vehicle hydroplaning

Possible Causes: Frost heave or subgrade settlement resulting from inadequate compaction during construction.

8. What are the causes of cracks? [M/J- 16]

Refer answer 4

9. Define Bleeding. [N/D- 16, M/J-12]

Bleeding or flushing is shiny, black surface film of asphalt on the road surface caused by upward movement of asphalt in the pavement surface. Bleeding is a safety concern since it results in a very smooth surface, without the texture required to prevent hydroplaning.

10. Differentiate Pumping and Ravelling. [N/D- 16]

Seeping or ejection of water and fines from beneath the pavement through cracks is called pumping.

The common reasons for this defect are

- Infiltration of water through the joints, cracks or edge of the pavement forms soil slurry. Movement of heavy vehicles on pavement forces this soil slurry to come out causing mud pumping.
- When there is void space between slab and the underlying base of sub-grade layer
- Poor joint sealer allowing infiltration of water
- Repeated wheel loading causing erosion of underlying material

Raveling is the progressive disintegration of a hot mix asphalt layer from the surface downward as a result of the dislodgement of aggregate particles. Asphalt raveling results in loose debris on the pavement, roughness, water collecting in the raveled locations which can result in vehicle hydroplaning and loss of skid resistance. The cause of this type of pavement damage can be numerous and include the following:

- Loss of bond between aggregate particles and the asphalt binder as a result of:
  - A dust coating on the aggregate particles that forces the asphalt binder to bond with the dust rather than the aggregate.
  - Aggregate Segregation. If fine particles are missing from the aggregate matrix, then the asphalt binder is only able to bind the remaining coarse particles at their relatively few contact points.
  - Inadequate compaction during construction. High density is required to develop sufficient cohesion within the asphalt mix.
- Mechanical dislodging by certain types of traffic (studded tires, snowplow blades or tracked vehicles).

1. List any eight cracks and defects in flexible pavements and describe their respective symptoms, possible causes and the treatment/ repair for each defect. [N/D-15, M/J-12, N/D-14]

1) Alligator (map) cracking
2) Consolidation of pavement layers
3) Shear failure
4) Longitudinal cracking
5) Reflection cracking
6)Transverse cracking
7) Shoving or Formation of waves and corrugations
8) Pot holes

Alligator (map) cracking

- This is the most common type of failure that occurs due to relative movement of pavement layer materials.
- When cracking is characterized by interconnected cracks, the cracking pattern resembles that of an alligator's skin or map. Therefore, it is referred to as alligator cracking or map cracking.
- This may be caused by repeated application of heavy wheel loads resulting in fatigue failure.
- Localized weakness in underlying base course would also cause a cracking of the surface course in this pattern.

Consolidation of pavement layers-Rutting

- Formation of ruts is mainly attributed to the consolidation of one or more layers of pavement.
- The repeated application of loads along the same wheel path cause cumulative deformation resulting in longitudinal ruts.
- There are two basic types of rutting
- Mix rutting occurs when the sub-grade does the pavement surface exhibits wheel path depressions as a result of compaction/mix design problems.
- Sub-grade rutting occurs when the sub-grade exhibits wheel path depressions due to loading. In this case, the pavement settles into the sub-grade ruts causing surface depressions in the wheelpath.
- Specific causes of rutting can be due to insufficient compaction of pavement layers during construction .If it is not compacted enough initially, pavement may continue to consolidate under traffic loads.

Joint Reflection Cracking

Possible Causes: Movement of the rigid pavement slab beneath the HMA surface because of thermal and moisture changes. Generally not load initiated, however loading can hasten deterioration. Repair: Strategies depend upon the severity and extent of the cracking:

- Low severity cracks (< 1/2 inch wide and infrequent cracks). Crack seal to prevent (1) entry of moisture into the subgrade through the cracks and (2) further raveling of the crack edges. In general, rigid pavement joints will eventually reflect through an HMA overlay without proper surface preparation.
- High severity cracks (> 1/2 inch wide and numerous cracks). Remove and replace the cracked pavement layer with an overlay after proper preparation of the underlying rigid pavement.

Longitudinal Cracking

Possible Causes:

- Poor joint construction or location. Joints are generally the least dense areas of a pavement. Therefore, they should be constructed outside of the wheelpath so that they are only infrequently loaded. Joints in the wheelpath will general fail prematurely.
- A reflective crack from an underlying layer (not including joint reflection cracking)
- HMA fatigue (indicates the onset of future fatigue cracking)
- Top-down cracking

Repair: Strategies depend upon the severity and extent of the cracking:

- Low severity cracks (< 1/2 inch wide and infrequent cracks). Crack seal to prevent (1) entry of moisture into the subgrade through the cracks and (2) further raveling of the crack edges. HMA can provide years of satisfactory service after developing small cracks if they are kept sealed (Roberts et. al., 1996[1]).
- High severity cracks (> 1/2 inch wide and numerous cracks). Remove and replace the cracked pavement layer with an overlay.

2. Define Overlay and the procedure for design and construction of overlays . [N/D-15]

Types of Overlays

- Asphalt overlay over asphalt pavements
- Asphalt overlays on CC pavements
- CC overlays on asphalt pavements
- CC overlays on CC pavements

Steps in Design of Overlays

Measurement and estimation of the strength of the existing pavement

Design life of overlaid pavement

Estimation of the traffic to be carried by the overlaid pavement

Determination of the thickness and the type of overlay

Effective Thickness Method

Basic concept

Thickness of overlay is the difference between the thickness required for a new pavement and the effective thickness of the existing pavement

- $h_{OL} = h_n - h_e$
- Where,
- h_{OL} = thickness of overlay
- h_n = thickness of new pavement
- h_e = *effective* thickness of existing pavement

All thicknesses of new and existing materials must be converted into an equivalent thickness of AC

$$h_e = \sum_{i=1}^{n} h_i C_i$$

h_i = thickness of layer *i*

C_i = conversion factor for layer *i*

3. Explain in detail the possible causes and remedial measures for joint failure. [A/M- 17, M/J-12]

The two prime factors responsible for rigid pavement failure are

- Use of poor quality material
- Inadequate stability of the pavement structure

Poor quality of material consist of following items

- Using soft aggregate
- Poor quality of sub-grade soil
- Poor joint filler R sealer materials

Inadequate stability of the pavement structure can be due to following reason

Inadequate pavement thickness

Lack of sub-grade support

Improper compaction of sub-grade

Improper spacing of joints

General surface maintenance:

For maintenance of gravel roads blading and occasional resurfacing is required.

For surface treatments of low type bituminous surface in maintenance of roads; Patching, seal coating or possible loosening oiling, re mixing and relaying are involved.

For high type bituminous concrete and Portland cement concrete, the Removal and replacement of failure areas and resurfacing are approximate treatment methods for highway maintenance.

Use same material and methods for road surface maintenance as fas as possible.

Highway Maintenance must be planned for rapid performance and to cause least possible disruption or hazard to traffic

4. Explain the methods employed for evaluation of pavement and explain the evaluation of pavements by Benkelman Beam method and deflection measurements. [A/M – 17]

The performance of flexible pavements is closely related to the elastic deflection of pavement under the wheel loads. The deformation or elastic deflection under a given load depends upon surgrade soil type, its moisture content and compaction, the thickness and quality of the pavement courses, drainage conditions, pavement surface temperature etc.

The Benkelman Beam Deflection Method is thus widely used for Evaluation of Structural Capacity of Existing Flexible Pavements and also for Estimation and Design of Overlays for Strengthening of any weak pavement for Highways.

This test procedure covers the determination of the **rebound deflection** of a pavement under a standard wheel load and tyre pressure.

Benkelman beam:

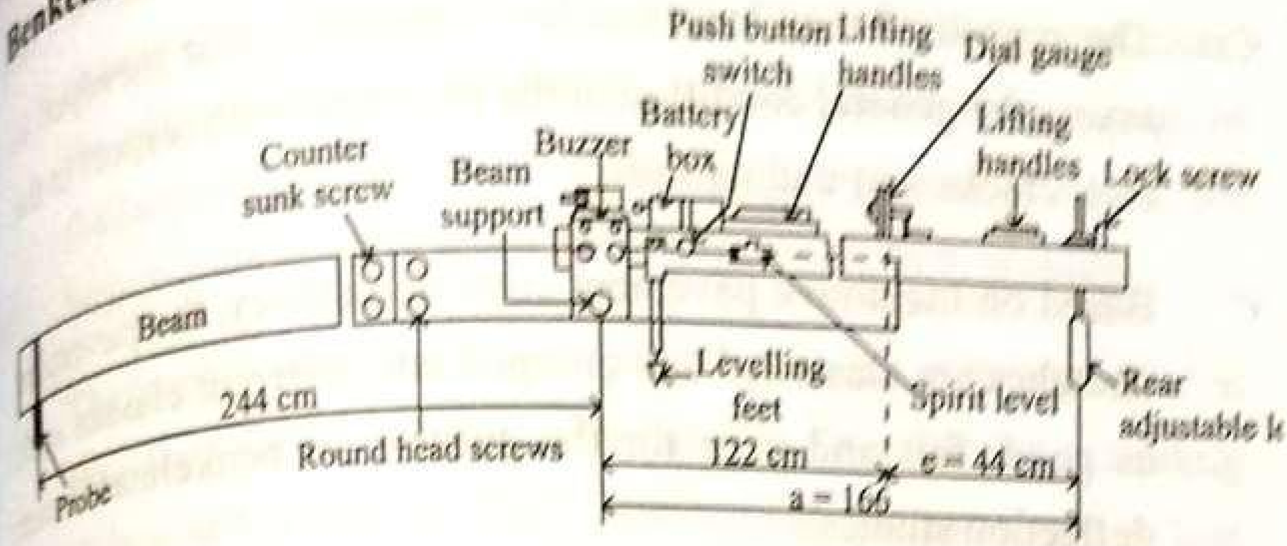


Figure 5.20 Benkelman Beam

- Benkelman beam is a device which can be used to measure the rebound deflection of the pavement due to dual wheel assembly or the design wheel load.
- The equipment consists of a slender beam which is pivoted to a datum frame at a probe end.
- The probe end of the beam is inserted between the dual rear wheels of truck and rests on the pavement surface at the centre of the loaded area of dual wheel load assembly.
- A dial gauge is fixed on the datum frame with its spindle in contact with the other end of the beam in such a way that the distance between the probe end and the fulcrum of the beam is twice the distance between the fulcrum and the dial gauge spindle.
- The rebound deflection reading measured at the dial gauge is to be multiplied by two to get the actual movement of the probe end due to the rebound deflection of the pavement surface when the dual wheel load is moved forward.

4. What are the possible causes for longitudinal cracking? [N/D- 16]

Problem

Allows moisture infiltration, roughness, and it may indicate the possible onset of alligator cracking and structural failure.

Possible Causes

Poor joint construction or location.

Joints are generally the least dense areas of a pavement. Therefore, they should be constructed outside of the wheel path so that they are only infrequently loaded. Joints in the wheel path like those shown in third through fifth figures above, will general fail prematurely. A reflective crack from an underlying layer (not including joint reflection cracking) HMA fatigue (indicates the onset of future alligator cracking) top-down cracking

Repair

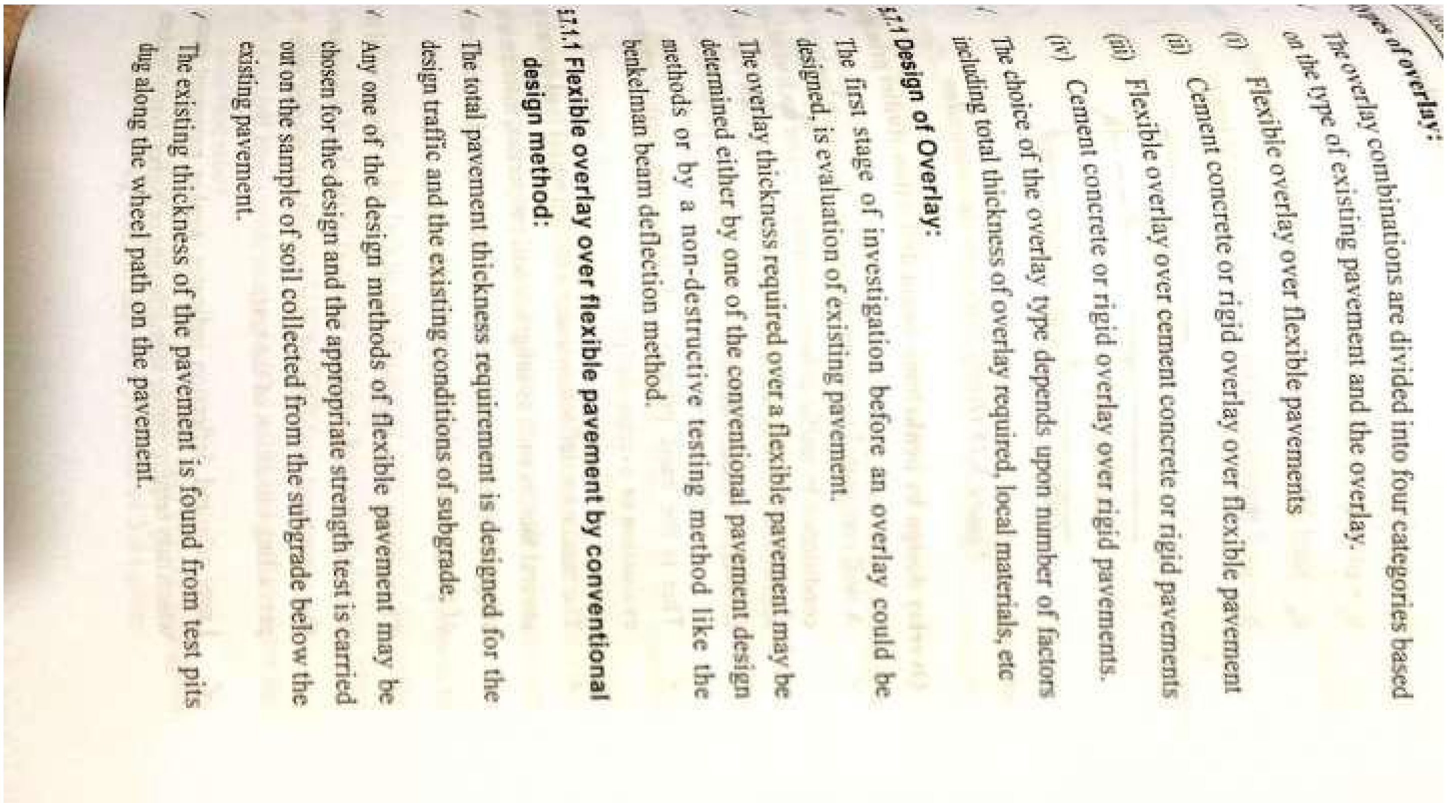
Strategies depend upon the severity and extent of the cracking:

- **Low severity cracks (< 1/2 inch wide and infrequent cracks)**. Crack seal to prevent (1) entry of moisture into the subgrade through the cracks and (2) further raveling of the crack edges. HMA can provide years of satisfactory service after developing small cracks if they are kept sealed (Roberts et. al., 1996[1]).
- **High severity cracks (> 1/2 inch wide and numerous cracks)**. Remove and replace the cracked pavement layer with an overlay.

5. Explain in detail about any four methods of strengthening of pavements. [N/D- 16]

5.7 STRENGTHENING OF EXISTING PAVEMENTS:

- ✓ For the successful maintenance of pavements, it is essential that they have adequate stability to withstand the design traffic under prevailing climatic and subgrade conditions.
- ✓ It the pavement have to support increased wheel loads and load repetitions, the pavements rapidly undergo the distress.
- ✓ Due to unexpected economic developments in the given region, the loading conditions may become severe and the alternative would be to strengthen the existing pavements.
- ✓ Strengthening may be done by providing additional thickness of the pavement of adequate thickness in one or more layers over the existing pavement, which is called overlay.
- ✓ If the existing pavements have completely deteriorated, an overlay would not serve the purpose and the solution would be to remove the existing damage pavement structure and rebuild the same.
- ✓ In partially damaged pavement sections, patch repair works are carried out before constructing the overlay.



Types of overlay:

- The overlay combinations are divided into four categories based on the type of existing pavement and the overlay.
  - (i) Flexible overlay over flexible pavements
  - (ii) Cement concrete or rigid overlay over flexible pavement
  - (iii) Flexible overlay over cement concrete or rigid pavements
  - (iv) Cement concrete or rigid overlay over rigid pavements.
- The choice of the overlay type depends upon number of factors including total thickness of overlay required, local materials, etc

5.7.1 Design of Overlay:

- The first stage of investigation before an overlay could be designed, is evaluation of existing pavement.
- The overlay thickness required over a flexible pavement may be determined either by one of the conventional pavement design methods or by a non-destructive testing method like the benkelman beam deflection method.

5.7.1.1 Flexible overlay over flexible pavement by conventional design method:

- The total pavement thickness requirement is designed for the design traffic and the existing conditions of subgrade.
- Any one of the design methods of flexible pavement may be chosen for the design and the appropriate strength test is carried out on the sample of soil collected from the subgrade below the existing pavement.
- The existing thickness of the pavement is found from test pits dug along the wheel path on the pavement.

6.Explain in detail the possible causes and remedial measures for joint failure. [M/J – 16]

Refer answer 3

7.Explain in detail the possible causes and remedial measures for joint spalling. [M/J – 16, N/D-14]

Joint spalling is the breakdown of the slab near edge of the joint. Normally it occurs within 0.5 m of the joints. The common reasons for this defect are

- Faulty alignment of incompressible material below concrete slab
- Insufficient strength of concrete slab near joints
- Freeze-thaw cycle
- Excessive stress at joint due to wheel load

Repair

Spalling less than 75 mm (3 inches) from the crack face can generally be repaired with a partial-depth patch.
Spalling greater than about 75 mm (3 inches) from the crack face may indicated possible spalling at the joint bottom and should be repaired with a full-depth patch.