CHAPTER 2

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SUBSOIL EXPLORATION

2.1 SOIL EXPLORATION

All office, laboratory and field works are done in order to *explore the subsurface of soil or rock conditions* at any given site to *obtain the necessary information required in design and construction*. Subsoil exploration is the first step in the design of a foundation system. Soil exploration consists essentially of boring, sampling and testing.

Mainly, planning of subsoil exploration involves three phases; reconnaissance phase, preliminary site investigation phase, and detailed site investigation phase.

2.1.1 RECONNAISSANCE PHASE

This phase consists of:

(a) Collection of all available information, and

(b) Reconnaissance of the site.

So that, it will indicate any settlement limitations and help to estimate foundation loads.

2.1.2 A PRELIMINARY SITE INVESTIGATION PHASE

This phase consists of:

(a) Preliminary design data that satisfy building code requirements, and

(b) Number and depth of boreholes.

So, it involves knowing of the distribution of structural loads which is required in the design of foundations. Also, a few borings or tests pits are to be opened to establish the stratification types of soil and location of water table. In addition to, one or more borings should be taken to rock when the initial boreholes indicate that the upper soil is loose or highly compressible.

2.1.3 A DETAILED SITE INVESTIGATION PHASE

In this phase, additional boreholes, samples will be required for zones of poor soil at smaller spacing and locations which can influence the design and construction of the foundation.

2.2 DRILLING OR BORING

- **Definition:** It is a procedure of advancing a hole into ground.
- **Drilling Methods:**

(1) Test Pits

(2) Auger Drilling

(a) Hand-auger drilling.
(b) Power-auger drilling.

(3) Wash Boring

(a) Jetting.
(b) Sludging (reverse drilling).

(4) Rotary Drilling

(a) Rotary drilling with flush.
(b) Rotary-percussion drilling.

(5) Percussion Drilling

Each of these methods has its merits and its drawbacks. However, Table (2.1) gives a guide for selecting the most appropriate drilling method.

Table (2.1): Drilling method selection.

Type of soil		Hand auger drilling	Wash boring		Rotary drilling		Percussion drilling
			Jetting	Sludging	Rotary drilling with flush	Rotary percussion drilling	
Gravel	Unconsolidated formations	X	X	X	X	✓ ?	✓ ?
Sand		✓	✓	✓	✓	✓ ?	✓ ?
Silt		✓	✓	✓	✓	✓ ?	✓ ?
Clay		✓	?	✓	✓	✓ slow	✓ slow
Sand with pebbles or boulders		X	X	X	X	✓ ?	✓ ?
shale	Low to medium strength formations	X	X	X	✓	✓ slow	✓
Sandstone		X	X	X	✓	✓	✓
Limestone	Medium to high strength formations	X	X	X	✓ slow	✓	✓ slow
Igneous (granite, basalt)		X	X	X	X	✓	✓ slow
Metamorphic (slate, gneiss)		X	X	X	X	✓	✓ V slow
Rock with fractures or voids		X	X	X	✓	✓	✓ !
Above water-table		✓	?	X	✓	✓	✓
Below water-table		?	✓	✓	✓	✓	✓

✓ = Suitable drilling method ✓ ? = Danger of hole collapsing ✓ ! = Flush must be maintained to continue drilling

? = Possible problems **x** = Inappropriate method of drilling

2.2.1 TEST PITS

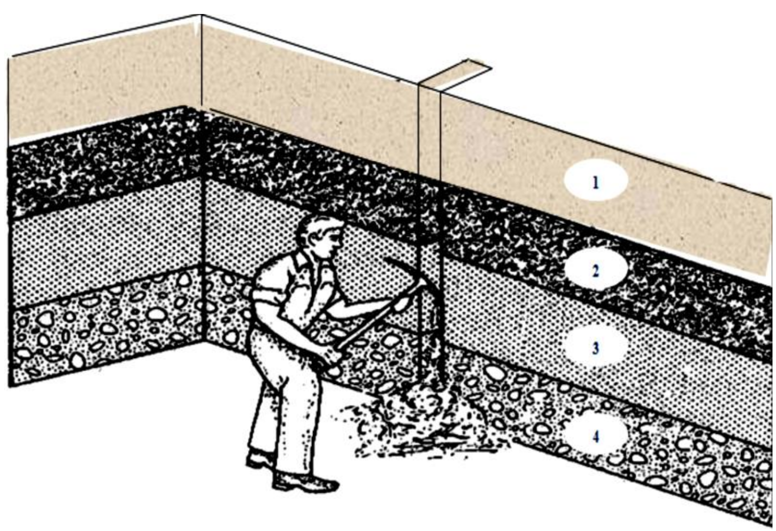
A pit is dug either by hand or by a backhoe. Probably in a test pit, the engineer can examine in detail the subsoil strata and take disturbed or undisturbed samples at the desired location (see Fig. 2.1):

Advantages:

- Inexpensive.
- Provide detailed information of stratigraphy.
- Large quantities of disturbed soils can be obtained for testing.
- Large blocks of undisturbed samples can be carved out from the pits.
- Field tests can be conducted at the bottom of the pit.

Disadvantages:

- Depth limited to about 6m.
- Deep pits uneconomical such as in case of investigation that involves basement construction.
- Excavation below groundwater (high water table) and into rock difficult and costly.
- Too many pits may scar site and require backfill soils.
- When the soil is unstable and has a tendency to collapse, this prevents the engineer from entering the pit and accompanied by certain risks.
- Unsuitable in granular soils belowwater levelor when the standard penetration resistancetest (N-value) is required.



Walls of test pit indicate four layers
(1) Clayey silt (2) Sandy silt (3) Clean sand (4) Sandy gravel

Fig. (2.1): Test pits.

2.2.2 AUGER DRILLING

(a) Hand-Augers

The auger of (10-20) cm in diameter is rotated by turning and pushing down on the handlebar. Then withdrawing and emptying the soil-laden auger to remove the excavated soil. Several new auger sections are added up to the required depth is reached. These augers can be available in different types such as (see Fig. 2.2):

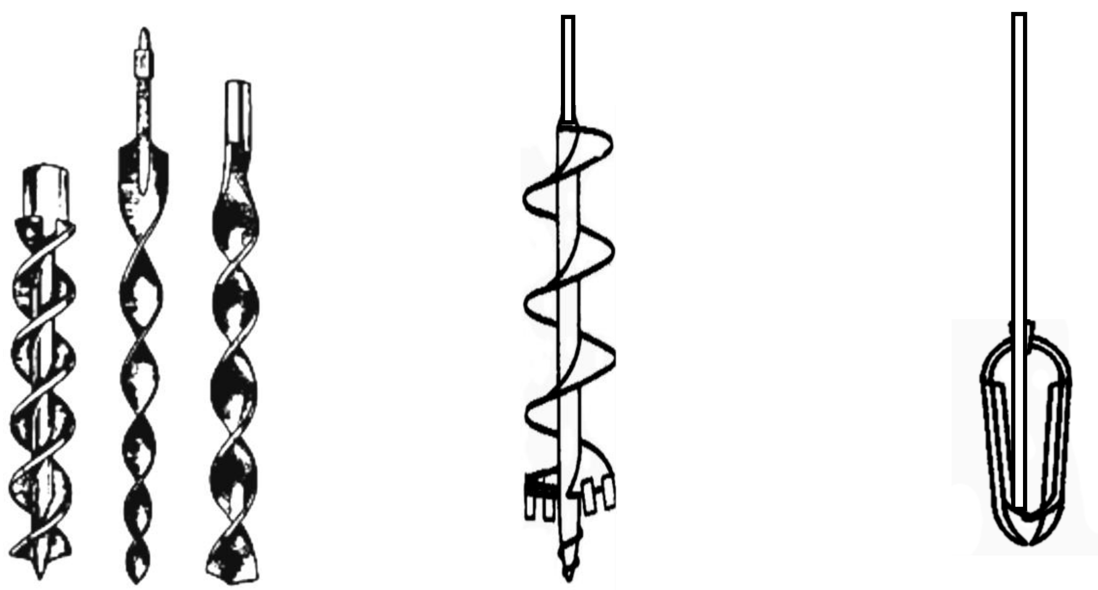
a. Helical Auger.
b. Short flight Auger, and
c. Iwan Auger.

Advantages:

- Inexpensive.
- Simple to operate and maintain.
- Not dependent on terrain.
- Portable.
- Used in uncased holes, and
- Groundwater location can easily be identified and measured.

Disadvantages:

- Slow compared with other methods.
- Depth limited to about 6m.
- Labor intensive.
- Undisturbed samples can be taken only for soft clay deposit, and
- Cannot be used in rock, stiff clays, dry sand, or caliches soils.



a. Helical (worm types) Augers **b. Short flight Auger** **c. Iwan (posthole) Auger**

Fig. (2.2): Hand-augers.

(b) Power-Augers

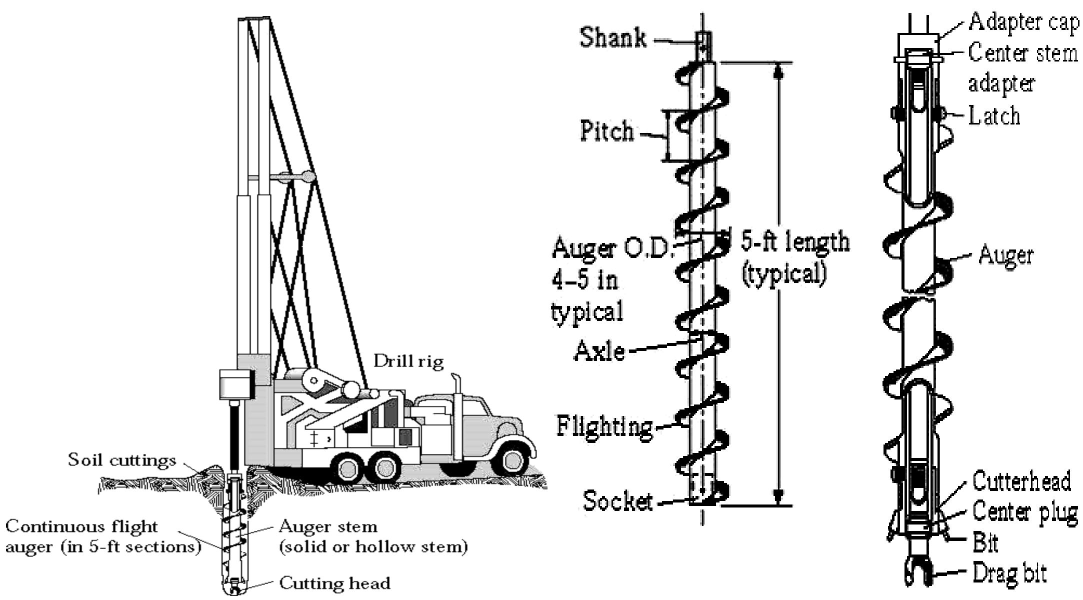
Truck or tractor mounted type rig and equipped with continuous flight augers that bore a hole of 100 to 250 mm in diameter. These augers can have a solid or hollow stem of (20-75) cm in diameter (see Fig.2.3).

Advantages:

- Used in clay or sand or silt soils.
- Quick.
- Used in uncased holes, therefore no need for using drilling mud.
- Undisturbed samples can be obtained quite easily, and
- Groundwater location can easily be identified and measured.

Disadvantages:

- Depth limited to about 15m.At greater depth, drilling becomes expansive, and
- Site must be accessible to motorized vehicle.



a. Continuous flight augers. **b. Solid-stem auger** **c. Hollow-steam auger**

Fig. (2.3): Power or mechanical-augers.

2.2.3 WASH BORING

Water is pumped to bottom of borehole and soil washings are returned to surface. A drill bit is rotated and dropped to produce a chopping action (see Fig. 2.4).

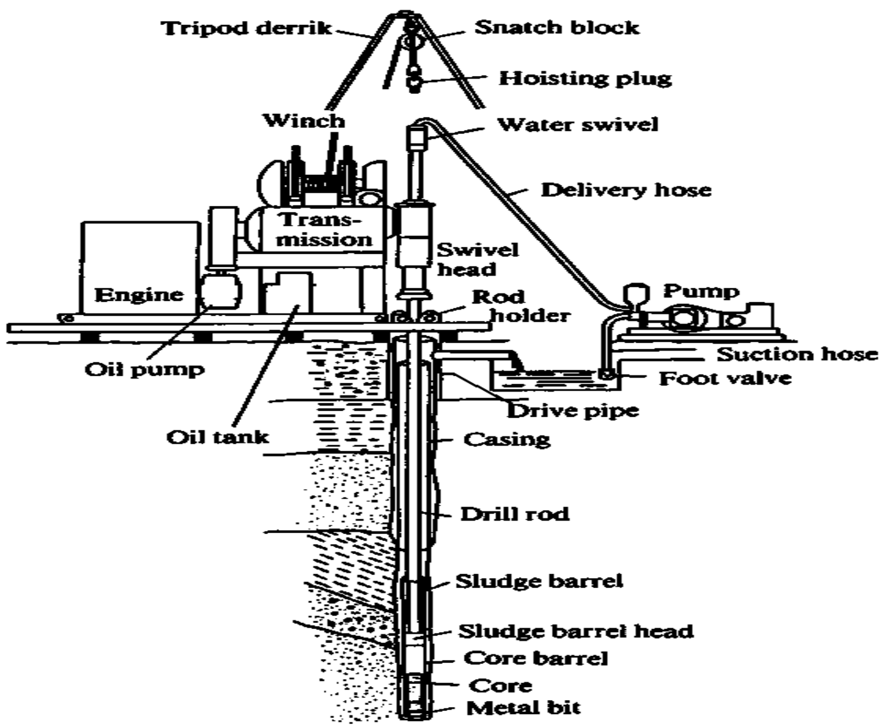


Fig. (2.4): Wash boring rig.

(a)Jetting Method

Method: Water is pumped down the center of the drill-rods, emerging as a jet. It then returns up the borehole or drill-pipe bringing with it cuttings and debris. The washing and cutting of the formation is helped by rotation, and by the up-and-down motion of the drill-string. A foot-powered treadle pump or a small internal-combustion pump is equally suitable.

(b) Slugging (Reverse Jetting)

Method: A hollow pipe of steel is moved up and down in the borehole while a one-way valve can be used to improvise successfully – provides a pumping action. Water flows down the borehole annulus (ring) and back up the drill pipe, bringing debris with it. A small reservoir is needed at the top of the borehole for recirculation. Simple teeth at the bottom of the drill-pipe, preferably made of metal, help cutting efficiency.

Advantages:

- The equipment can be made from local, low-cost materials, and it is simple to use.
- Possible above and below the water-table.
- Suitable for clay to silt clay, silt soils and unconsolidated rocks, and
- Used in uncased holes.

Disadvantages:

- Slow drilling through stiff clays and gravels.
- Undisturbed soil samples cannot be obtained.
- Water is required for pumping.
- Difficulty in obtaining accurate location of groundwater level.
- Boulders can prevent further drilling, and
- Depth is limited to about 30m.

2.2.4ROTARY DRILLING

(a) Rotary Drilling with Flush

Method: A drill-pipe and bit are rotated to cut the rock. Air, water, or drilling mud is pumped down the drill-pipe to flush out the debris. The velocity of the flush in the borehole annulus must be sufficient to lift the cuttings (**see Fig. 2.5**).

Advantages:

- Quick.
- Can drill any type of soil or rock.
- Possible to drill to depths of over 40 meters.
- Operation is possible above and below the water-table.
- Undisturbed soil samples or rock cores can easily be recovered.
- Water and mud supports unstable formations, and
- Possible to use compressed air flush.

Disadvantages:

- Expensive equipment.
- Terrain must be accessible to motorized vehicle.
- Water is required for pumping.
- Difficulty in obtaining accurate location of groundwater level.
- There can be problems with boulders, and
- Rig requires careful operation and maintenance (additional time required for setup and cleanup).

(b) Rotary-Percussion Drilling

Method: In very hard rocks, such as granite, the only way to drill a hole is to pulverize the rock, using a rapid-action pneumatic hammer, often known as a 'down-the-hole hammer' (DTH). Compressed air is needed to drive this tool. The air also flushes the cuttings and dust from the borehole. Rotation of 10-30 rpm ensures that the borehole is straight, and circular in cross-section (**see Fig. 2.5**).

Advantages:

- Drills hard rocks.
- Possible to penetrate gravel.
- Fast, and
- Operation is possible above and below the water-table.

Disadvantages:

- Higher tool cost than other tools illustrated here.
- Air compressor required, and
- Requires experience to operate and maintain.

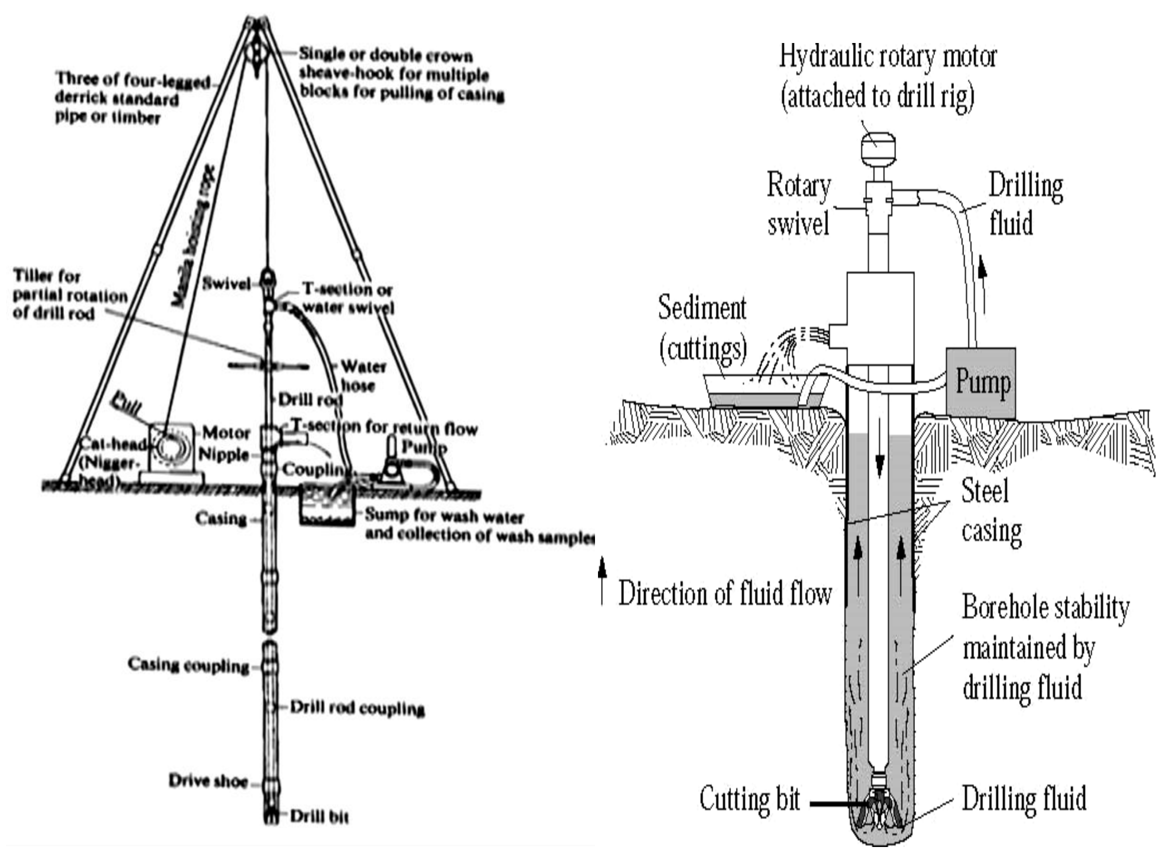


Fig. (2.5): Rotary drilling.

2.2.5 PERCUSSION DRILLING

Method: The lifting and dropping of a heavy (+50kg) cutting tool will chip and excavate material from a hole. The tool may be fixed to rigid drill-rods or to a rope or cable. With a mechanical winch, depths of hundreds of meters can be reached.

Advantages:

- Simple to operate and maintain.
- Suitable for a wide variety of rocks.
- Operation is possible above and below the water-table.
- It is possible to drill to considerable depths, and
- Can be used for boring observation wells.

Disadvantages:

- Slow, compared with other methods.
- Equipment can be heavy.
- Problems can occur with unstable rock formations.
- Water is needed for dry holes to help remove cuttings, and
- Due to high disturbance of soil, the obtained samples can not be used for testing.

2.3 UNDER GROUND WATER IN THE TEST HOLE

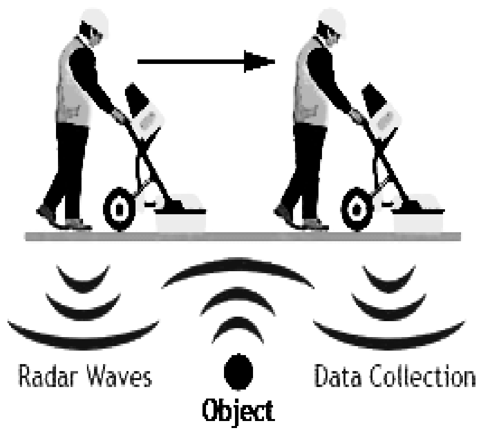
The depth of the water table (W.T.) as measured during drilling and sampling should be carefully evaluated. It is always necessary to wait for at least 24 hours to check on the stabilized water table for the final measurement. The technician should plug the top of the drill holes and flag them for identification. Care is required to ensure that the water level in the drill hole is always maintained. Any sudden drop or rise of the water table or a sudden change in the penetration resistance should be carefully recorded in the field logs of borings.

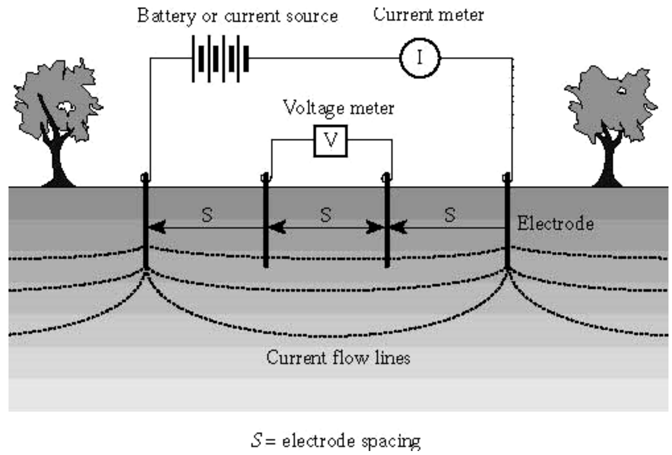
2.4 GEOPHYSICAL METHODS

These methods represent indirect methods of subsoil exploration and mainly consist of:

(1) Ground Penetration Radar (GPR). **(2) Electrical Resistivity Method (ERM)**
(2) Electromagnetic Method (EM), and **(4) Seismic Methods.**

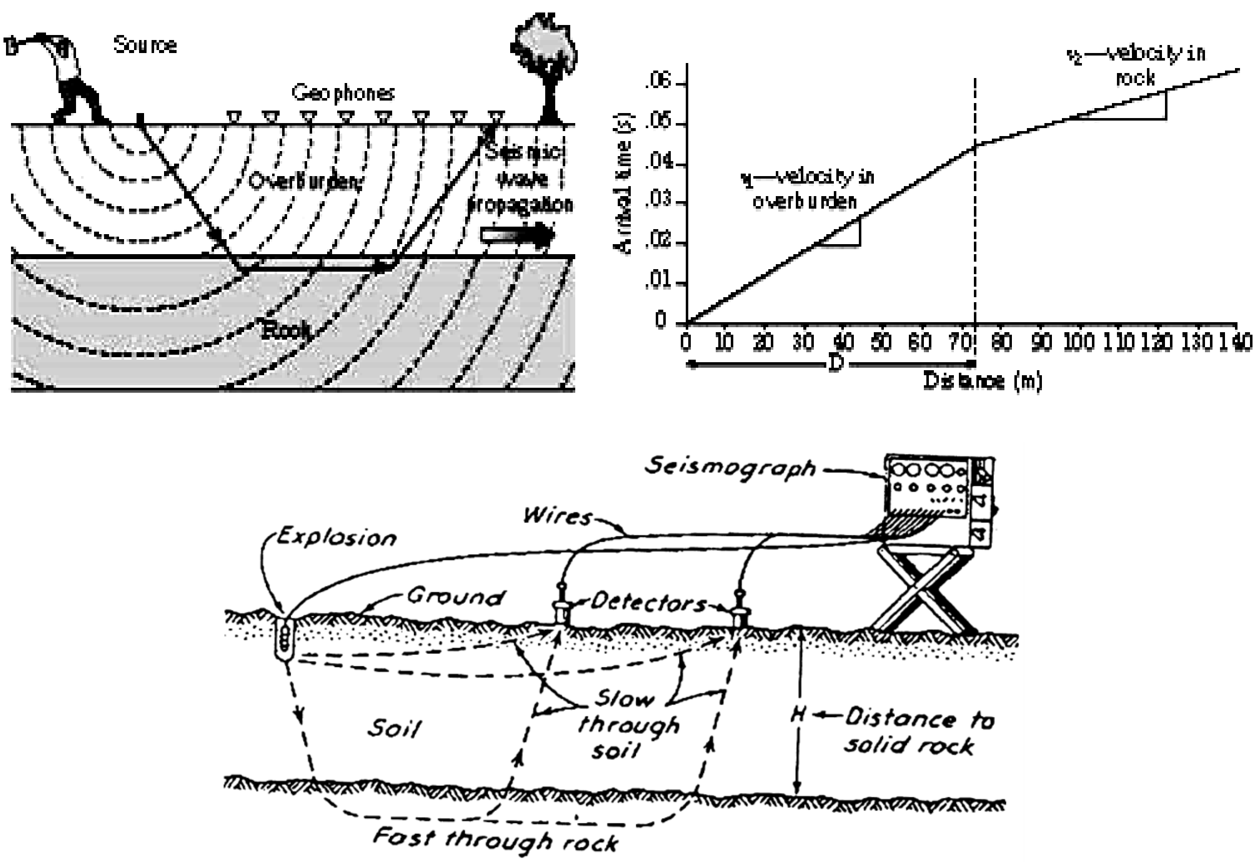
In subsoil investigation, the seismic methods are most frequently used. These methods are based on the variation of the wave velocity in different earth materials. They involve in generating a sound wave in the rock or soil, using a sledge-hammer, a falling weight, or a small explosive charge, and then recording its reception at a series of geophones located at various distances from the shot point, as shown in **Fig.(2.6)**. The time of the refracted sound arrival at each geophone is noted from a continuous reader. Typical seismic velocities of earth materials in (m/sec) are shown in **Table (2.1)**.





(b) Ground penetration radar.

(a) Electrical resistivity method.



(c) Seismic survey method.

Fig. (2.6): Geophysical methods.

Table (2.1): Typical seismic velocities ofdifferent earth materials

(after Peck, Hanson, and Thornburn,1974).

Type of soil	Seismic Velocity (m/sec)
Dry silt, silt, loose gravel. loose rocks, talus, and moist fine-grained soil	150 –180
Compacted till, indurated clays, gravel below water table, compacted clayey gravel, cemented sand, and sandy clay	750 – 2250
Rock, weathered, fractured, or partly decomposed	600 – 3000
Sandstone, sound	1500 – 4200
Limestone, chalk, sound	1800 – 6000
Igneous rock, sound	360 – 6000
Metamorphic rock, sound	300 – 4800

Requirements of seismic exploration:

- Equipment to produce an elastic wave, such as a sledgehammer used to strike a plate on the surface.
- A series of detectors, or geophones, spaced at intervals along a line from wave origin point, and
- A time-recording mechanism to record the time of origin of the wave and the time of its arrival at each detector.

Advantages of seismic exploration:

1. Permits a rapid coverage of large areas at a relatively small cost.
2. Not hampered by boulders and cobbles which obstruct borings, and
3. Used in regions not accessible to boring equipment, such as the middle of a rapid river.

Disadvantages of seismic exploration:

1. Lack of unique interpretation.
2. It is particularly serious when the strata are not uniform in thickness nor horizontal,
3. Irregular contacts often are not identified, and
4. The strata of similar geophysical properties sometimes have greatly different properties.

Note: *Whenever possible, seismic data should be verified by one or two borings before definite conclusions can be reached.*

2.5 SAMPLING

During the boring, three types of representative soil samples should be collected which are valuable to geotechnical engineers; these are as follows:

(a) The disturbed samples (D): which were collected from auger cuttings at specified depths?

(b) The undisturbed samples (U): which were obtained using a thin Shelby tubes of 100mm in diameter and (400-450)mm in length, and

(c) The (SS) samples: which were taken from standard split spoon sampler used in a standard penetration test (S.P.T.) that performed at different intervals depending on soil stratification.

All these samples then sealed tightly in plastic bags to retain its in situ moisture content, labeled and transported to the soil mechanics laboratory, to perform the required tests.

Fig.(2.7) shows some details of standard split-spoon and thin-wall tube samplers that commonly used in in-situ testing and sample recovery equipment. A modification in the design of the split spoon sampler allows the insertion of brass thin-wall liners into the barrel. Four sections of brass liners (each 4 inch long) can be used. Such a device allows the sampling and penetration test at the same time. This method was initiated in California and known as the “California” sampler.

Samples of rock are generally obtained by rotary core drilling. Diamond core drilling is primarily used in medium-hard to hard rocks. Special diamond core barrels up to 8 inch in diameter are occasionally used and larger ones can be used. Such large samples enable the geologist to study the formation and texture of the foundation rock in detail.

A summary of different sampler types which can be used to obtain disturbed or undisturbed samples of each type of soil are listed in ***Table (2.2).***

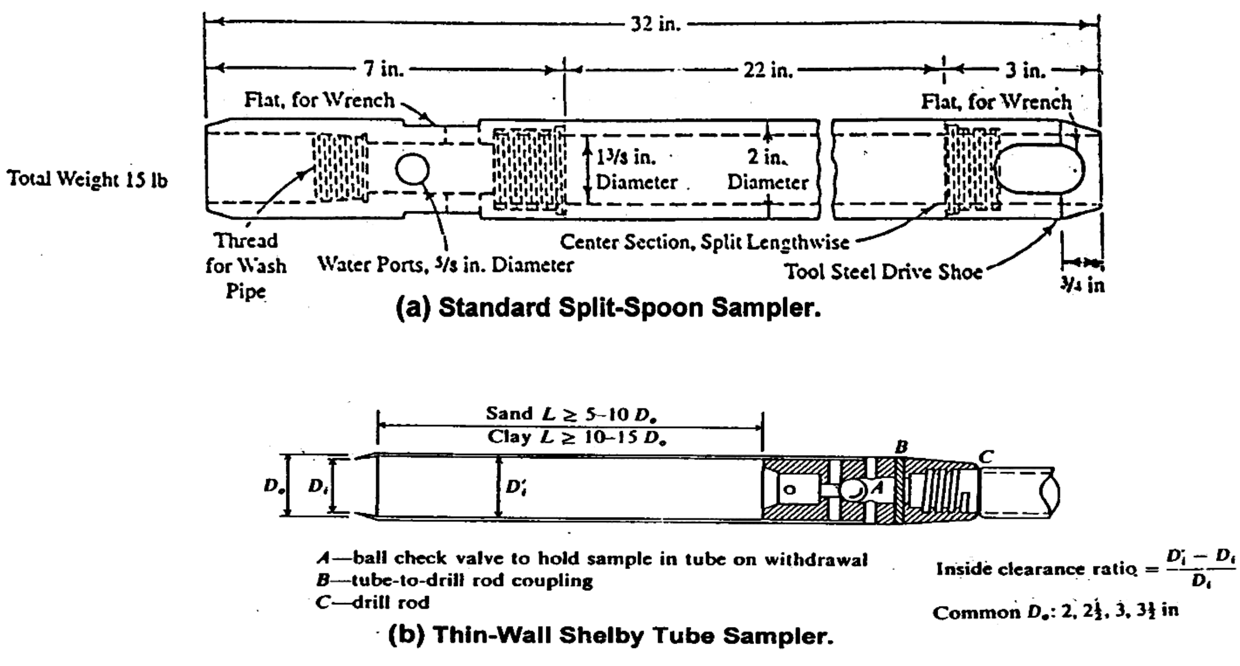


Fig.(2.7) : Details of commonly used samplers for in-situ testing (*after Moore, 1980*).

Table (2.2): Types of samplers used for taking soil and rock samples from test holes.

Type of sampler	Procedure	Type of soil and Remarks
1. Highly disturbed sampler	Auger boring, wash boring, and percussion drilling.	• All types of soils, • Due to high disturbance it is unsuitable for foundation exploration.
2. Split spoon sampler	Standard Penetration Test.	• Cohesive, cohesionless soils and soft rocks, • For taking disturbed samples which are required for physical and geotechnical analysis of soil as well as chemical tests. • In cohesionless soils, the penetration number (N) is used for making both strength and settlement estimates.
3. Thin wall Shelby tube	16gauge seamless steel tube (7.5-15) cm dia.; preferably pushed by static force instead of driven by hammer.	• For taking undisturbed samples from cohesive soil, • Unsuitable for granular soils and hard materials.
4. Core barrel sampler: (a) Single tube, and **(b)**Double tube core barrel.	Rotary drilling	• For taking undisturbed continuous rock samples.
5. Piston samplers	Rotary drilling	• For taking undisturbed samples in soft and slightly stiff cohesive soils.
6. Hard carved samples: **(a)** Spring core catcher, and **(b)**Scraper bucket.	Cut by hand from side of test pit.	• For taking disturbed samples in cohesive or cohesionless soils.
7. Hand-cut samples	Cut by hand from side of test pit.	• For taking disturbed samples in cohesionless soil or disturbed and undisturbed block samples in cohesive soil.

2.6 SAMPLE DISTURBANCE

Certain amounts of disturbance during sampling must be regarded as inevitable:-

1. **Effect of stress relief:**

   Due to boring, the stress state in soil will be changed as a result of a stress relief.

2. **Effect of area ratio (Ar %):**

   It is the ratio of the volume of soil displacement to the volume of the collected sample.

$$A_r = \frac{D_o^2 - D_i^2}{D_i^2} x100 \quad \text{...} \mathbf{(2.1)}$$

   For stiff clay < 20%, for soft clay $\leq 10\%$ and samples with $A_r > 20\%$ considered as disturbed samples.

3. **Effect of friction and adhesion:**

   If the length of sampler is large with respect to diameter, a bearing capacity failure may occur due to disturbance of sample.

$$C_i = \frac{D_o - D_i}{D_i} x100 \quad \text{...} \mathbf{(2.2)}$$

Where, $C_i =$ inside clearance = (0.3-0.4) % and not more than 1%.

4. **Effect of the way in which the force is applied to the spoon:** that means by pushing or driving or by constant rate of penetration.

2.7 TESTING

The tests performed on each type of the three different soil samples are as follows:
As a rule, undisturbed samples (U) can be tested for strength and compressibility to determine the stress-strain characteristics of the material, in addition to classification and chemical tests. Whereas, disturbed (D) or (SS) samples as available were mainly used for physical and geotechnical analysis of soil as well as chemical tests.

2.7.1 LABORATORY TESTS:

The obtained samples should be tested according to the procedure of the American Society for Testing and Materials (ASTM) or the British Standards (BS) whichever is appropriate. The test program of the samples includes the followings:

1. Classification Tests:
Sieve and hydrometer analysis, natural water content, Atterberg's limits, specific gravity, and wet and dry unit weights.

2. Compaction Test:
Modified Procter compaction test must be carried out on some soil samples to obtain the maximum dry density $(\gamma_d^{max.})$ and the relevant optimum moisture content (OMC).

3. Shear Strength and Compressibility Tests:
Unconfined or Triaxial compressive strength test and one-dimensional consolidation test.

4. Chemical Tests:
Sulphate Content (SO_3^{-2})%,Total Soluble Salts(T.S.S.), Organic Matter Content (ORG.)%,PH- value, Carbonate Content (CO_3^{-2}), and Chlorides Content (Cl^{-1})%.

2.7.2 FIELD TESTS

During the subsoil exploration, several field tests as given in **Table (2.3)**, can be performed depending on the available testing equipment, required parameters for design of foundations, and the economic point of view.

Table (2.3): Types of field tests.

Purpose of test	Type of test

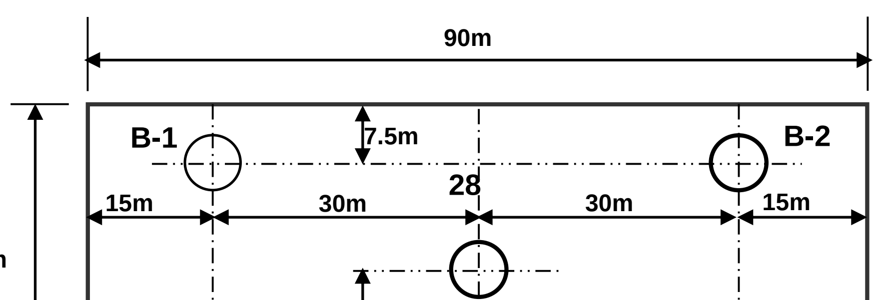
1.SPT N-value (for granular soil)	• Standard or Dynamic Penetration Test (SPT).
2.Undrained shear strength (for cohesive soil)	• Static Penetration Test (CPT) • Vane shear test (for soft to medium fine grained soil, clay and silt clay; up to Cu =1.0 kg/cm^2), • Tor vane shear test (for soft soil; up to Cu =5.0 kg/cm^2), • Pocket penetrometer, • Pressure meter test; it is of three types: **a-** Menard (to obtain; $R_D, \phi, S_u, E_s, G', m_v, C_c$), **b-** Self-boring (to obtain; $R_D, \phi, S_u, u, E_s, G', m_v, C_c, C_v$), **c-** Screw boring (to obtain; E_s and G').
3.Bearing capacity	• Pavements: plate bearing ;CBR test, • Footings: plate bearing test, • Piles subjected to vertical loads: load test, • Batter piles: lateral load test.
4.Elastic and shear modulus	• Seismic Tests: **a-** Cross-hole, **b-** Down-hole, and **c-** Surface refraction (to measure R_D, E_s, G', liquefaction resistance and thickness of soil layers).
5. Permeability	• Pumping Test: **a-** Constant head test, **b-** Variable head test, **c-** Piezometers test (or ground water observation).
6. Compaction control	• Field or In-place Density: <u>For Sand:</u> a- Sand cone method, b- Rubber balloon method, <u>For Clay:</u> a- Penetration needle, b- Core cutter method.

2.8 LOGS OF BORINGS AND RECORDS OF TESTS RESULTS

At the beginning, a map giving specific locations of all borings should be available. Each boring should be identified (by number)and its location documented by measurement to permanent features. Such a map is shown in **Fig.(2.8)**. For each boring, all pertinent data should be recorded in the field on a boring log sheet. These sheets are preprinted forms containing blanks for filling in appropriate data. **Fig.(2.9)** shows an example of a boring log sheet.

Soil data obtained from a series of test borings can best be presented by preparing a geologic profile, which shows the arrangement of various layers of soil, the groundwater table, existing and proposed structures, and soil properties data. An example of a geologic profile is shown in **Fig.(2.10)**.

Depending on the results of the laboratory tests and the field observations, the actual subsoil profiles or logs of borings can more accurately be sketched (see **Fig.(2.11)**). In addition to, the actual description of soil strata in each borehole is summarized within records of tests results**.**



DRILLING COMPANY, INC. BORE HOLE NO.: ------------
PROJECT: LOCATION: -------------------

Name ---Date Time Depth Casing at
Address -- --
CASING (SIZE AND TYPE) --------------------------------
SAMPLE SPOON (SIZE AND TYPE) --------------------
HAMMER (CSG): WT. ------------, DROP -----------------
(SPOON): WT. -----------, DROP -----------------
DATE: STARTED ---, COMPLETED ---------------------, DRILLER ---------------------

Field Samples		Depth of Sampling (m)		'N'- Value			Visual Description of Soil
No.	Type	From	To	6″	6″	6″	
1	D	0.0	2.0				Black and grey moist fill,
2	U	2.0	4.0				Black peat.
3	S.S	4.5	5.0	11	14	6	Sandy clay and silt mixture.
4	D	5.0	7.0				Sandy silt and clay mixture.
5	U	7.0	9.0				Silt with fine gravel and traces of fine sand.
6	S.S	9.5	10.0	4	8	3	Sandy clay and silt mixture.

Fig.(2.9): boring log sheet.

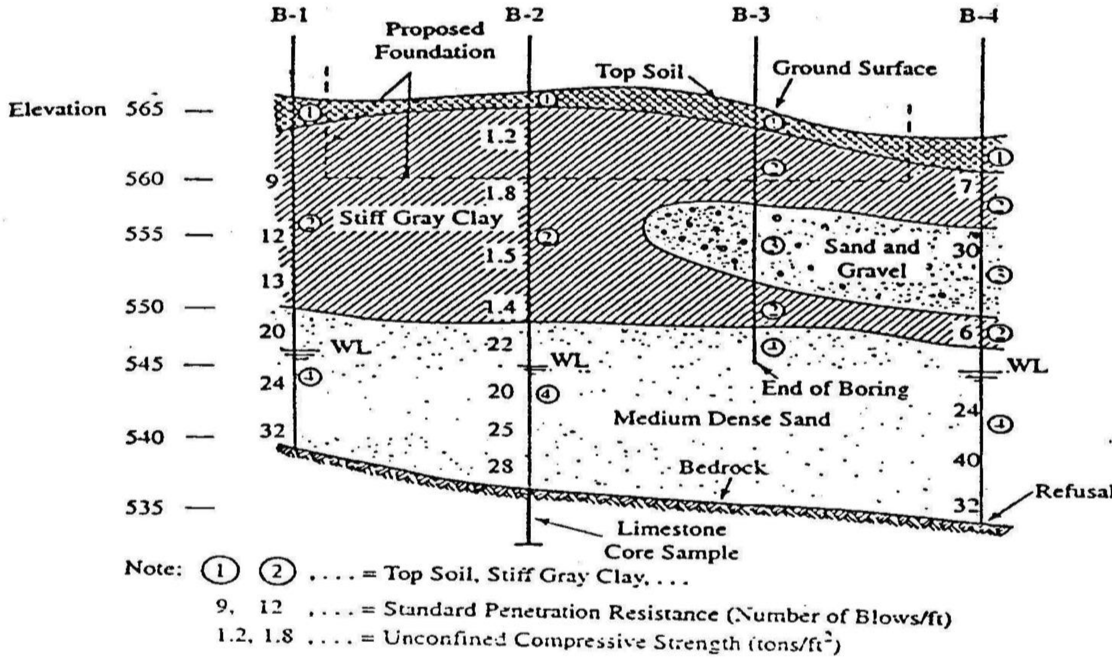


Fig.(2.10):Example of geologic profile.

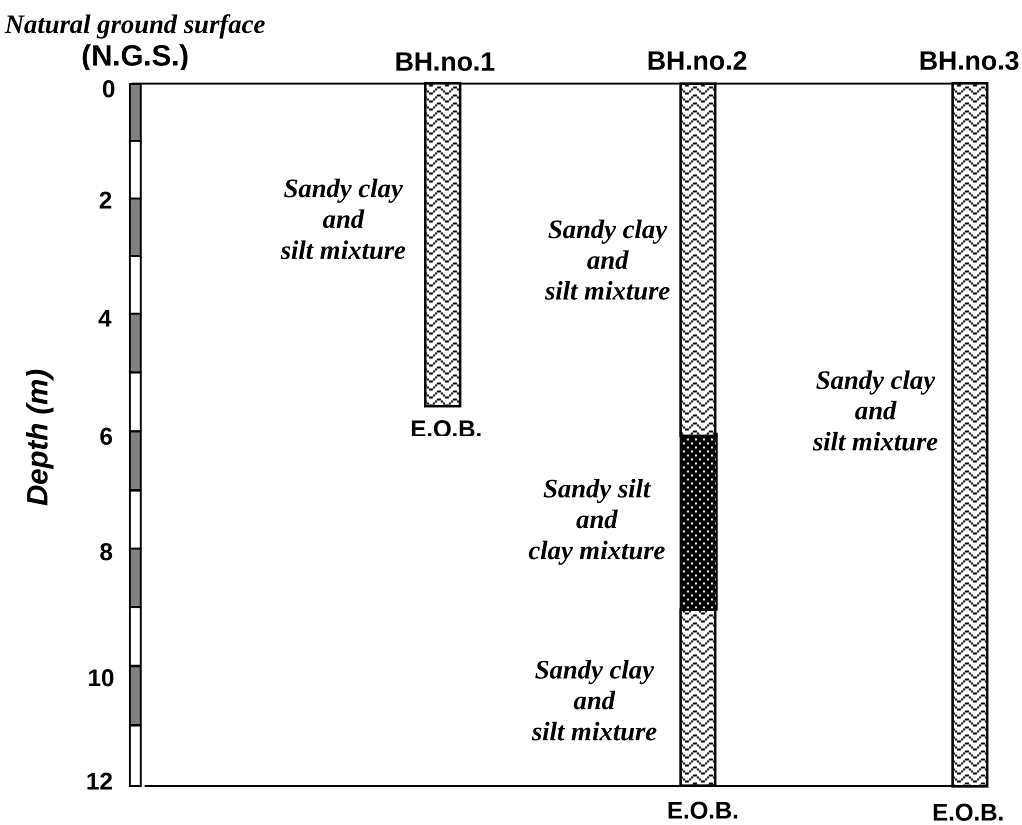


Fig.(2.11): Log of borings for 1st. stage of garden city housing project Tanahi District / Duhok city.

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RECORD OF TESTS RESULTS

PROJECT: Garden City Housing (1st. Stage)
LOCATION: Tanahi/ Duhok City
BORE HOLE NO.: 2
SHEET NO.: 1-2

1 Samples: Number Field	1 Samples: Number Lab.	1 Samples: Type	2 Depth of Sampling (m): From	2 Depth of Sampling (m): To	3 Index Properties: ω %	3 Index Properties: LL %	3 Index Properties: PI %	4 Particle Size Distribution and Hydrometer Analysis: Gravel %	4: Sand %	4: Silt %	4: Clay %	5 Specific Gravity 'Gs'	6 S.P.T 'N' Value	7 Symbol Unified Classification	8 Description of Soil	9 R.Q.D %
1	100	D	0	2.0	22.8	53	27	0	10	55	35	2.70		CH	Sandy clay and silt mixture.	
2	101	U	2.0	2.5	21.7	51	27	0	8	54	38	2.70		CH	Sandy clay and silt mixture.	
3	102	S.S	2.5	3.0				0	12	54	34		12	CH	Sandy clay and silt mixture.	
4	103	D	3.0	5.0	22.2	51	27	0	10	50	40	2.72		CH	Sandy clay and silt mixture.	
5	104	U	5.0	5.5	20.1	56	28	0	8	55	37	2.72		CH	Sandy clay and silt mixture.	
6	105	S.S	5.5	6.0				0	10	44	46		32	CH	Sandy clay and silt mixture.	
7	106	D	6.0	8.0	19.1	59	32	0	13	43	44	2.73		CH	Sandy silt and clay mixture.	

Field	10 Unit Weight (kN/m^3): Wet	10: Dry	11 Unconfined Compressive Strength qu (kN/m^2)	12 Shear Strength Parameters: Drained c' (kPa)	12: Drained φ'	12: Undrained c_u (kPa)	12: Undrained ϕ_u	13 Consolidation Test: eo	13: mv (m^2/kN)	13: Cv (m^2/year)	13: Cc (m^2/kN)	13: Cs (m^2/kN)	13: Pc (kN/m^2)	13: k (cm/sec)	14 Chemical Tests: SO_3^{-2} %	14: T.S.S %	14: ORG. %	14: PH- value	14: Gypsum %	14: $CaCO_3$ %	14: Cl^{-1} %
1															0.65	0.14	2.70	8.0	—	50	0.06
2	20.94	17.21	76			38	0	0.539	0.66 x 10^{-4}	0.371	0.073	0.045		0.77x 10^{-4}							
3																					
4															0.02	0.03	0.07	7.36	—	17	—
5	21.22	17.67	92			46	0	0.510	0.44 x10^{-4}	0.581	0.100	0.050		0.10 x10^{-4}							
6																					
7																					

UNIVERSITY OF DUHOK
COLLEGE OF ENGINEERING
ENGINEERING CONSULTING BUREAU

RECORD OF TESTS RESULTS

PROJECT: Garden City Housing (1^{st}. Stage)
LOCATION: Tanahi/ Duhok City
BORE HOLE NO.: 2
SHEET NO.: 2-2

1 Samples			2 Depth of Sampling (m)		3 Index Properties			4 Particle Size Distribution and Hydrometer Analysis				5	6	7	8	9	10 Unit Weight (kN/m^3)		11	12 Shear Strength Parameters				13 Consolidation Test							14 Chemical Tests						
Number																				Drained		Undrained															
Field	Lab.	Type	From	To	ω %	LL %	PI %	Gravel %	Sand %	Silt %	Clay %	Specific Gravity 'Gs'	S.P.T 'N' Value	Symbol Unified Classification	Description of Soil	R.Q.D %	Wet	Dry	Unconfined Compressive Strength qu (kN/m^2)	c' (kPa)	ϕ'	c_u (kPa)	ϕ_u	eo	mv (m^2/kN)	Cv ($m^2/year$)	Cc (m^2/kN)	Cs (m^2/kN)	Pc (kN/m^2)	k (cm/sec)	SO_3^{-2} %	T.S.S %	ORG. %	PH- value	Gypsum %	$CaCO_3$ %	Cl^1 %
8	107	U	8.0	8.5	21.0	60	35	0	8	45	47	2.73		CH	Sandy silt and clay mixture.		21.60	17.85	136			68	0	0.500	0.39×10^{-4}	2.326	0.070	0.041		0.28×10^{-4}							
9	108	S.S	8.5	9.0				0	12	43	45		36	CH	Sandy silt and clay mixture.																						
10	109	D	9.0	11.0	16.4	62	34	0	6	50	44	2.73		CH	Sandy clay and silt mixture.																						
11	110	U	11.0	11.5	18	57	31	0	10	48	42	2.73		CH	Sandy clay and silt mixture.		21.20	17.96	168			84	0	0.491	1.6×10^{-4}	2.311	0.140	0.050		1.17×0^{-4}							
12	111	S.S	11.5	12.0				0	14	46	40		>50	CH	Sandy clay and silt mixture.																						

No water table is encountered at the time of boring and sampling (12/2/2007).

2.9 NUMBER OF BOREHOLES

It is a good practice in the beginning to take a few numbers of borings so that a soil profile can be drawn with reasonable accuracy and then the preliminary program can be adjusted to suit subsoil conditions.

Obviously, the more boreholes and the closer they are spaced, the more accurate the resulting geologic profile. Boreholes number and layout may need to be changed as more information emerges, so that, an additional boreholes may be required during the survey.

For rough guidelines, if soil conditions are relatively uniform or the geological data are limited, Tables (2.4) and (2.5) can be used as a guide in planning of the preliminary program:

Table (2.4): Number and spacing of boreholes according to the type of project *(after Hvorslev 1949, and Road Research Laboratory 1954).*

Project	Distance between borings (m)			Minimum number of boreholes
	Horizontal stratification of soil			
	uniform	average	erratic	
Multi-story building	45	30	15	4
1 or 2 story building	60	30	15	3
Bridge, pier, abutment, Tv.Tower	----	30	7.5	1-2
Highways	300	150	30	----
Borrow pits	150-300	60-150	15-30	----
Isolated small structures: such as small houses.	-	-	-	1
Special structures: - Retaining walls - Earth dams	120 25- 50			at centerline with some B.Hs. located on both sides
Slope stability problems	-	-	-	(3-4) B.Hs at critical zone and (1) B.H outside this zone

Table (2.5): Number of borings for medium to heavy weight buildings, tanks, and other similar structures on shallow foundations *(after Sowers, 1979)*.

Subsurface Conditions	Structure Footprint Area for Each Exploratory Boring (m^2)
Poor quality and / or erratic	100 – 300
Average	200 – 400
High quality and uniform	300 – 1000

2.10 DEPTH OF BORINGS

Hvorslev (1949) suggested a number of general rules which remain applicable:

- The soft strata should be penetrated even when they are covered with a surface layer of high bearing capacity;

- In case of very heavy loads or when seepage or other considerations are governing, the borings may be stopped when rock is encountered or after a short penetration into strata of exceptional bearing capacity and stiffness, provided it is known from explorations in the vicinity of the area that these strata have adequate thickness or are underlain by still stronger formations. But, if these conditions are not satisfied, some of the borings must be extended until it has been established that the strong strata have adequate thickness irrespective of the character of the underlying material;
- When the structure is to be founded on rock, it must be verified that bedrock and not boulders have been encountered, and it is advisable to extend one or more borings from 3 to 6m into solid rock in order to determine the extent and character of the weathered zone of the rock;

For rough guidelines, the following criteria can be used for minimum depths, from considerations of stress distribution or seepage,:

1. Foundations:

- All borings should extend below all deposits such as top soils, organic silts, peat, artificial fills, very soft and compressible clay layers;
- Boring should be sufficiently deep for checking the possibility of a weaker soil at greater depth which may settle under the applied load;
- Deeper than any strong layer at the surface checking for a weaker layer of soil under it which may cause a failure (see Fig.(2.12a));
- The depth at which the net increase in stress due to the foundation or building load is less than 5% of the effective overburden pressure;
- The depth at which the net vertical total stress increase due to the foundation or building load is less than 10% of the stress applied at foundation level (contact pressure);
- For isolated spread footings or raft foundations, explore to a depth equal 1.5B (B = least width of the footing or the raft)(see Fig.(2.12b));
- For group of interfering footings, explore to a depth equal 1.5B′(where, B′= width of interfering footings)(see Fig.(2.13));
- For heavy structures (pressure > 200 kPa), the depth of borings should be extended to 2B (width of footing);
- For strip footings, explore to not less than 3B (width of footing) for $B > 6\text{m}$ and $\frac{L}{B} \geq 10$.
- For multistory buildings, explore to:
  (i) $D = D_f + 3.S^{0.7}$ (in meter)………*for light steel or narrow concrete buildings*,
  (ii) $D = D_f + 6.S^{0.7}$ (in meter) …….. *for heavy steel or wide concrete buildings*.
  where: D = Depth of boring, D_f =Depth of footing, and S = Number of stories.
- If piled foundation is expected, the borehole depth D = ($D_f + \frac{2}{3}$ L+ 1.5B)or D= (L + 3m) into the bearing stratum (see Fig.(2.14a));

2. Reservoirs: Explore soil to:

(i) The depth of the base of the impermeable stratum, or

(ii) Not less than 2x maximum hydraulic head expected.

3. Dams: Because of the critical factor is the safety against seepage and foundation failure, boreholes should penetrate not only soft or unstable materials, but also permeable materials to such a depth that seepage patterns can be predicted. Thus, Hvorslev (1949) recommends:-

- *For earth structures*, a depth equal to 1.5 times the base width of the dam, and
- *For concrete structures*, a depth between 1.5 and 2.0 times the height of the dam.

4. Roads, highways, and air fields: the minimum depth is 5m below the finished road level, provided that vertical alignment is fixed but should extend below artificial fill or compressible layers. In practice some realignment often occurs in cuttings, and side drains may be dug up to 6m deep or to bore to at least 1.5 times the embankment height in fill areas, and to at least 5m below finished road level in cut.

5. Retaining walls, slopes stability problems: Explore to:

- 1.5B (wall base width) or 1.5H (wall height) whichever is greater below the bottom of the wall or its supporting piles (see Fig.(2.14b)), In addition to;
- It must be below an artificial fills or compressible layers, and deeper than possible surface of sliding;

6. Canals, deep cut and fill sections on side hills: Explore to at least to:

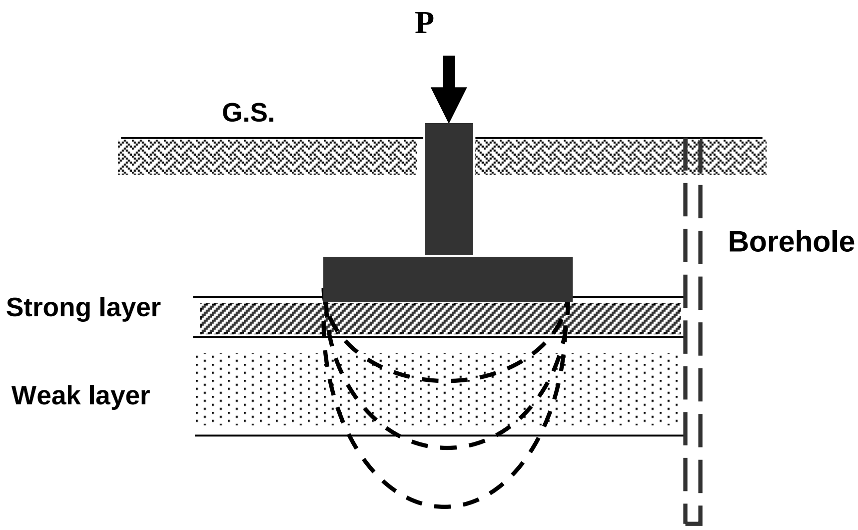
(i)3m below the finished level in cut, or

(ii)B when $B \le H$, or

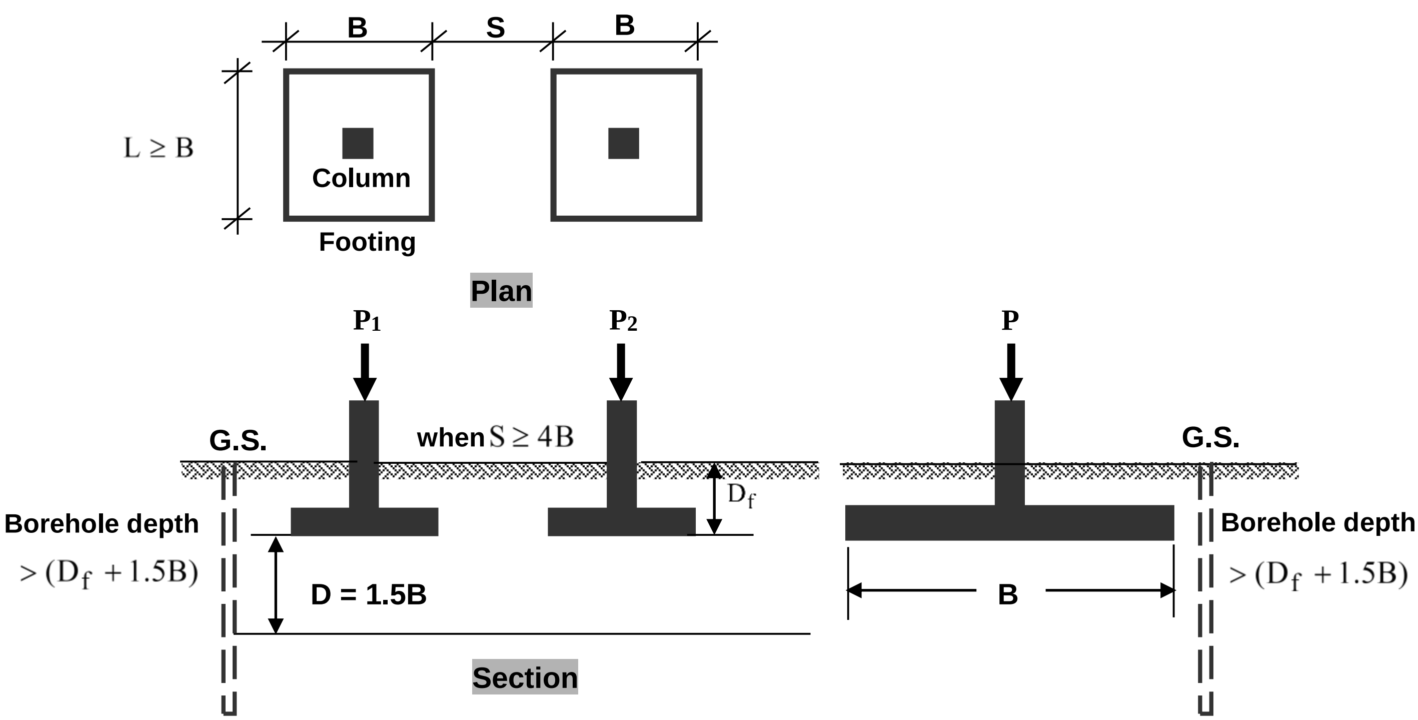
(iii) H when $B > H$(see Figs.(2.15a and 2.15b)).

7. Embankments:

The depth of exploration should be at least equal to the height of the embankment and should ideally penetrate all soft soils if stability is to be investigated. If settlements are critical then soil may be significantly stressed to depths below the bottom of the embankment equal to the embankment width (see Fig.(2.15c)).

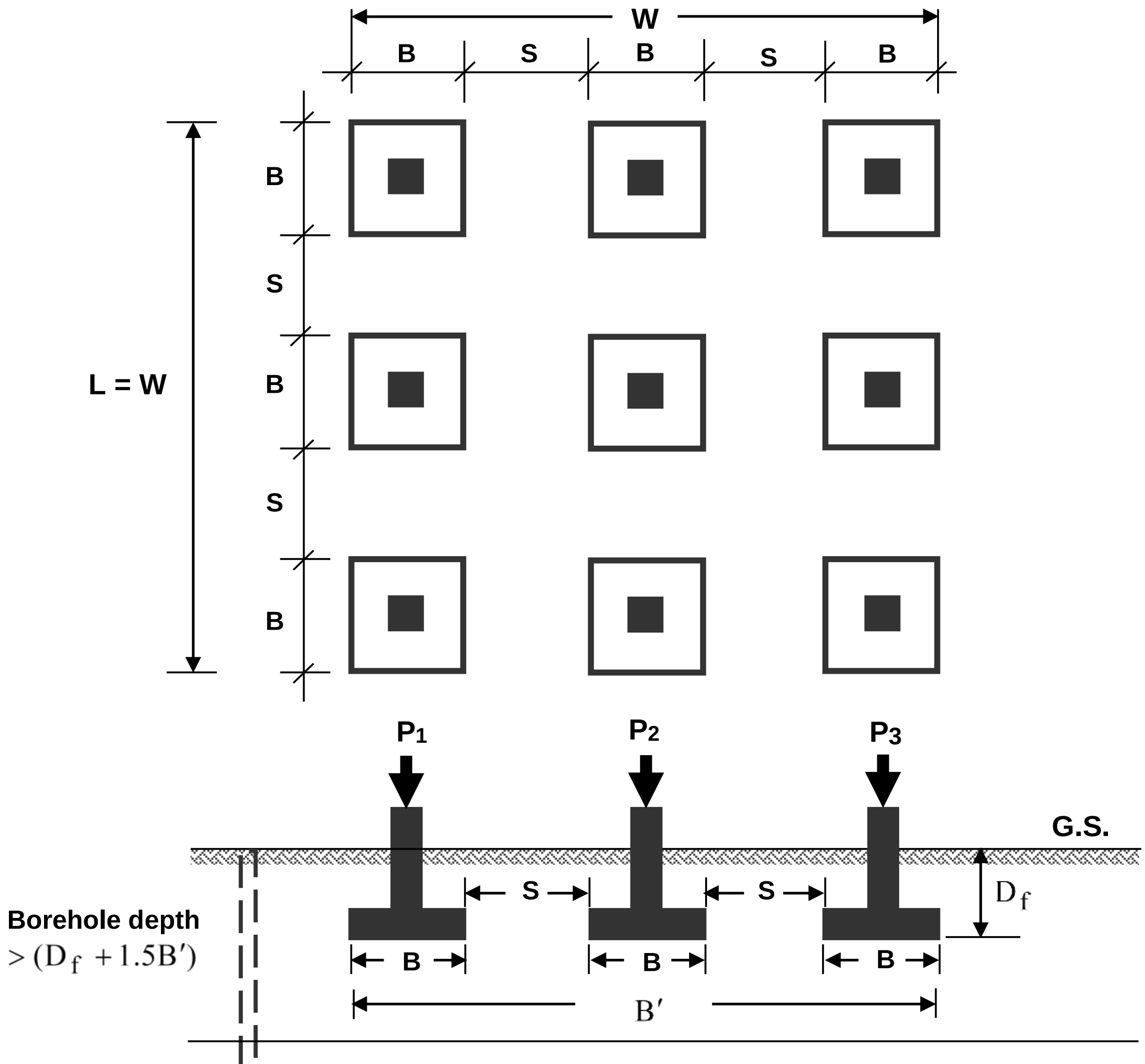


(a) Existence of rock layer

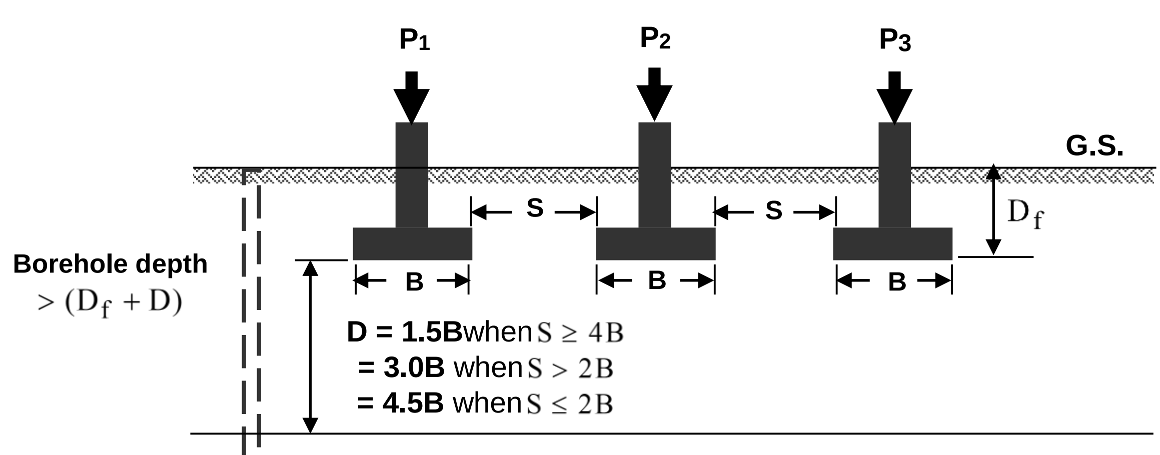


(b) Isolated spread footing **(c) Raft or mat foundation.**

Fig.(2.12):Depth of borings for spread and raft foundations.



(a) Single row of adjacent spread footings when $S < 4B$



(b) Multiple rows of adjacent spread footings.

Fig.(2.13):Depth of borings for adjacent spread footings.

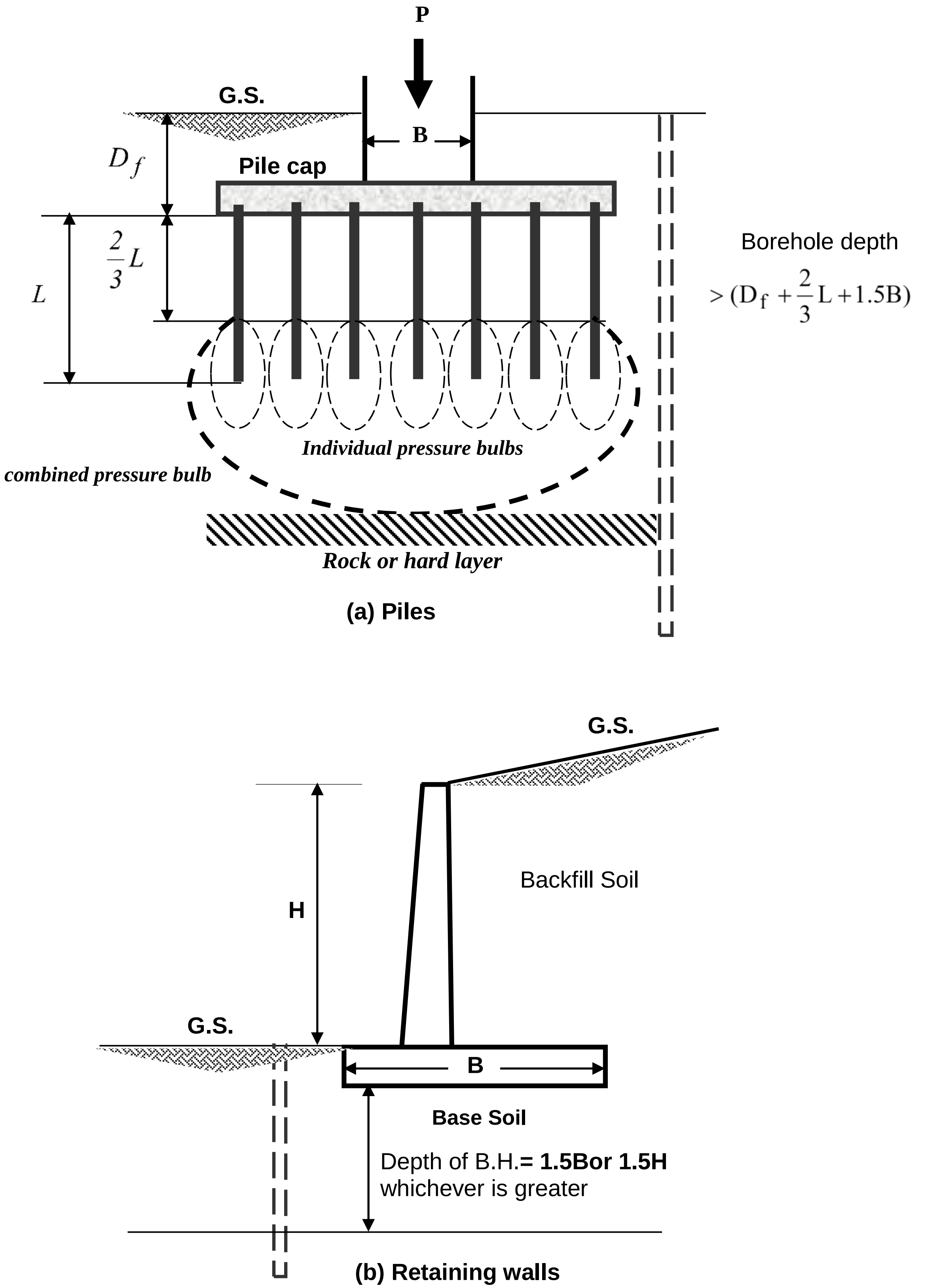
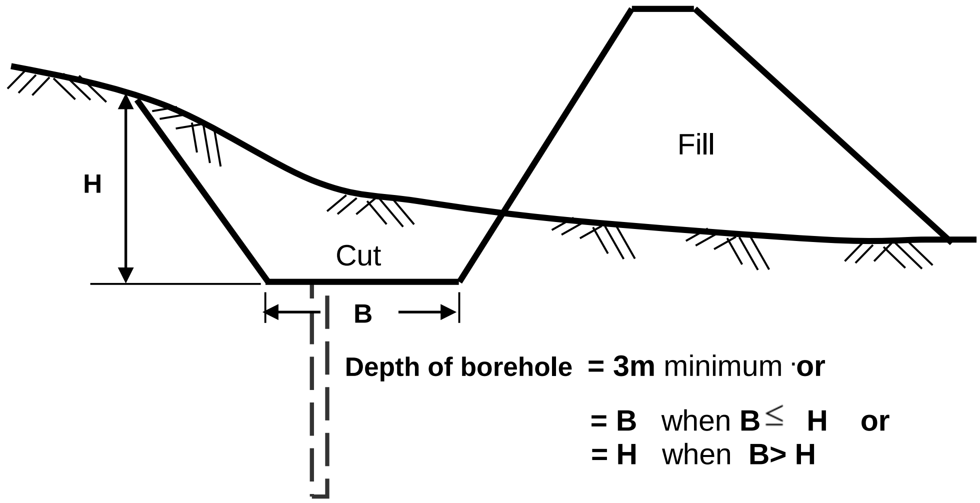
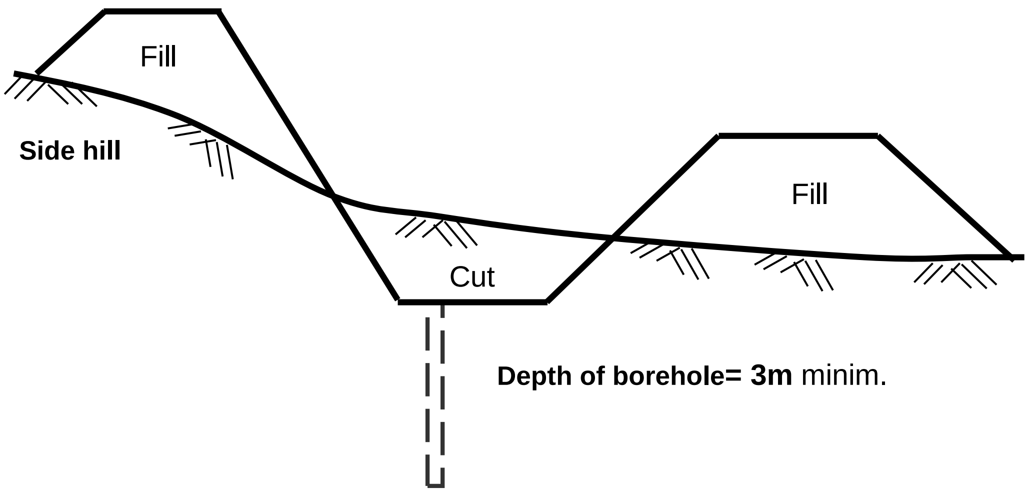


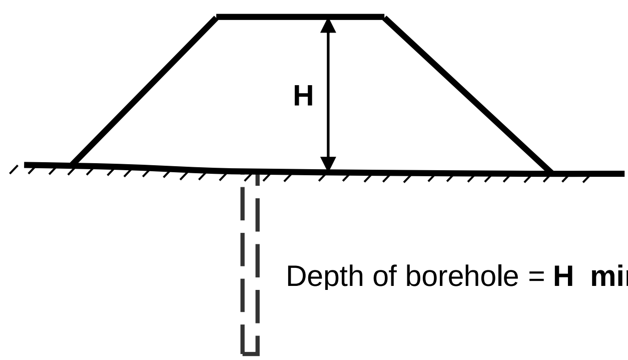
Fig.(2.14): Depth of borings for piles, and retaining walls.



(a) Deep cut and fills sections on side



(b) Normal section of a canal.



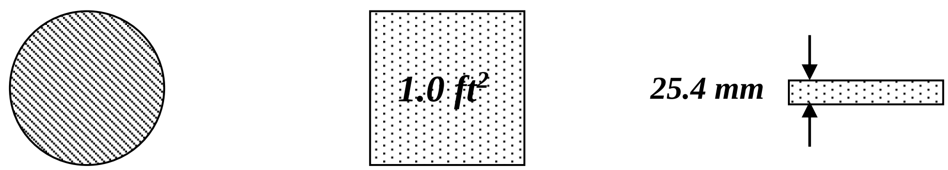
(c) High embankment

Fig.(2.15): Depth of borings for cuts and fills, canals, and embankments.

2.11 FIELD LOAD TEST

It is a method to investigate the stress-strain (or load-settlement) relationship of soils. Then, the results are used in estimating the bearing capacity. In this test, the load is applied on a model footing and the amount of load necessary to induce a given amount of settlement is measured.

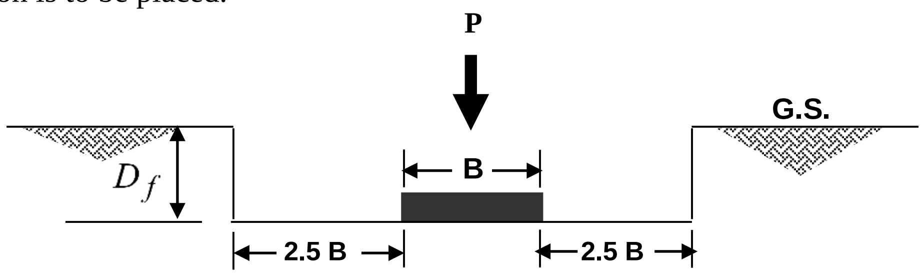
Round plates from (150-750)mm in diameter by 150mm increment (i.e., 150, 300, 450, 600, 750)mm are available as well as square plates of (1.0 ft^2) area. The minimum thickness of plate (1 inch or 25.4mm).



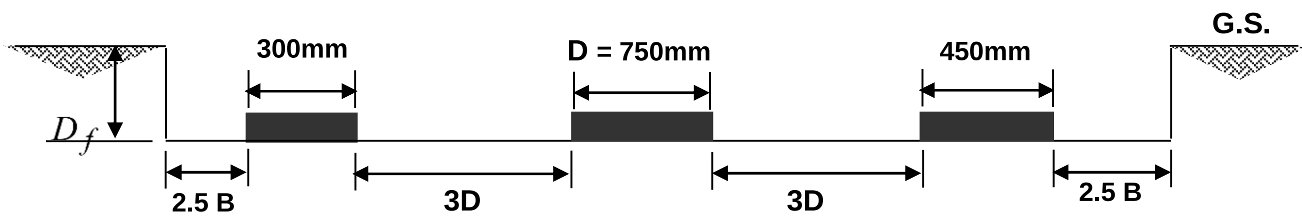
Round plate ***Square plate***

Procedure of load test as given by ASTM D110-72:

(1) Excavate a pit to width at least 6 times as wide as the used plate, and to the depth that the foundation is to be placed.



If it is specified that three sizes of plates are to be used for the test, the pit should be large enough so that, there is an available spacing between tests of 3 times the diameter (D) of the largest plate. This is useful for studying the size effect of footings.



(2) A square loading plate 2.5cm thick and (30 x 30)cm is placed on the surface of the soil at the bottom of the pit. There should not be any surcharge load placed on the soil within a distance of (60cm) from around the plate.

(3) A vertical load is placed on the plate in increments and settlements are recorded as an average from at least three dial gauges accurate to (0.025mm) that attracted to an independent suspension system. Load increment should be approximately 1/10 of the estimated allowable soil pressure. For each load increment, settlement readings should be taken at regular intervals of not less than (1 hr.) until there is no further settlement. The same time duration should be used for all the loading increments.

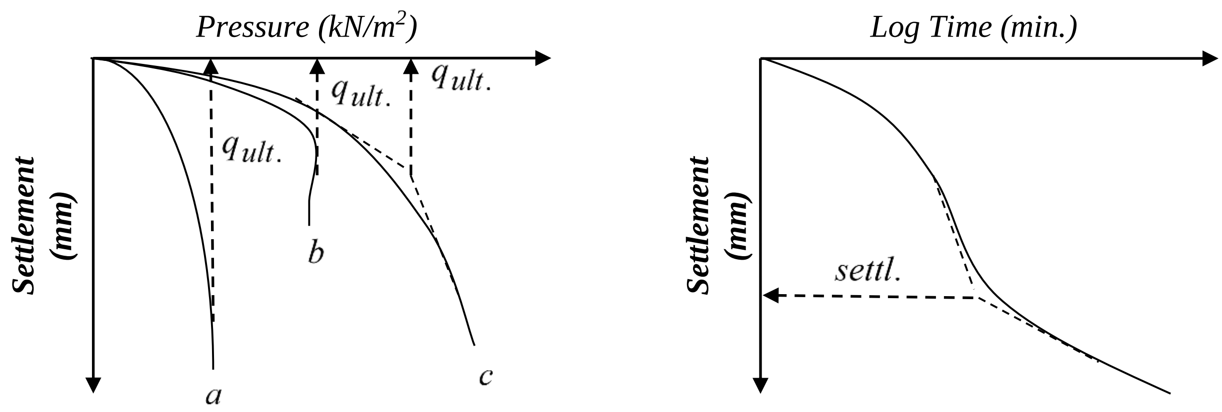
(4) The test is continued until a settlement of 25mm is observed or until the load increments reached 1.5 times the estimated allowable soil pressure.

(5) If the load is released, the elastic rebound of the soil should be recorded for a periods of time equal to the same time durations of each applied load increment.

(6) The result of each test can be represented graphically as follows:-

(a) Settlement versus log time curve (for each load increment),

(b) Load-settlement curve (for all increments) from which $q_{ult.}$ is obtained.



(a) Load - settlement curve **(b) Log time-Settlement curve**

Fig.(2.16): Typical load test results.

- **<u>For cohesive soil</u>***<u>(bearing capacity is independent of footing size):</u>*

$$\begin{cases} q_f = q_p \\ s_f = s_p \dfrac{B_f}{B_p} \end{cases} \qquad \textbf{(2.3)}$$

- **<u>For cohesionless soil</u>***<u>(bearing capacity increases with size of footing):</u>*

$$\begin{cases} q_f = q_p \dfrac{B_f}{B_p} \\ s_f = s_p \left(\dfrac{2B_f}{B_p + B_f} \right)^2 \end{cases} \qquad \textbf{(2.4)}$$

- **<u>Settlement for both cohesive and cohesionless soils:</u>**

$$\frac{s_f / B_f}{s_p / B_p} = \left(\frac{B_f}{B_p}\right)^n \quad \textbf{(2.5)}$$

where, s_f and s_p are settlements of footing and plate, B_f and B_p are their respective widths; provided that B_p = 1.0 ft for $\frac{B_f}{B_p} \geq 5$ as well as the footing and plate carries the same intensity of load, and (n) is an exponent depends on soil type; with some of its values are:

Type of soil	n
Clay	0.03-0.50
Sandy clay	0.08-0.10
Dense sand	0.40-0.50
Medium sand	0.25-0.35
Loose sand	0.20-0.25

- **For** $c-\phi$ **soils (***bearing capacity from two-plate load tests;* **after Housel, 1929):**

$$V = A.q + P.s \quad \textbf{(2.6)}$$

where,
V = total load on a bearing area,
A = contact area of footing or plate,
q = bearing pressure beneath A,
P = perimeter of footing or plate, and
s = perimeter shear.

This method needs data from two-plate load tests so that Eq.(**2.6**) can be solved for q and s (for given settlement). After the values of q and s are known, then, the size of a footing required to carry a given load can be calculated.

2.12 FIELD PENETRATION TESTS

2.12.1 Dynamic or Standard Penetration Test (SPT)

This test is preferred for very hard deposits, particularly of cohesionless soils for which undisturbed samples cannot easily be obtained. It utilizes a split-spoon sampler shown previously in **Fig.(2.7a)** that driven into the soil.

The test consists of driving the standard split-barrel sampler of dimensions (680mm length, 30mm inside diameter and 50mm outside diameter) a distance of 460mm (18") into the soil at the bottom of the boring. This was done by using a 63.5kg (140Ib) driving mass (or hammer) falling "free" from a height of 760mm (30"). Then, counting the number of blows required for driving the sampler the last 305mm (12") to obtain the (N) number (neglecting the no. of blows for the upper first 150mm).

Note: The SPT- value is rejected or halted in any one of the following cases:

(a) if50 blows are required for any 150mm increment, or
(b) if100 blows are obtained, or
(c) if10 successive blow produce no advance.

The number of blows (N) can be correlated with the relative density (D_r) of cohesionless soil (sand) and with the consistency of cohesive soil (clay) as shown in **Tables (2.6, and 2.7)**.

Table (2.6): Relative density of sands according to results of standard penetration test.

SPT- value N/30cm	Relative density $D_r = \frac{e_{max} - e_{insitu}}{e_{max} - e_{min}} x100$		$\phi°$
0-4	0-15	Very loose	28
4-10	15-35	Loose	28-30
10-30	35-65	Medium	30-36
30-50	65-85	Dense	36-41
> 50	85- 100	Very dense	> 41

Table (2.7): Relation of consistency of clay, SPT N-value, and unconfined compressive strength (q_u).

SPT- value N/30cm	consistency	$q_u\,(ksf)$	$q_u\,(kg/cm^2)$
Below	Very soft	0-0.5	0-0.25
2-4	Soft	0.5-1	0.25-0.5
4-8	Medium	1-2	0.5-1
8-15	Stiff	2-4	1-2
15-30	Very stiff	4-8	2-4
> 30	Hard	> 8	> 4

2.12.2 Corrections for N-value

(1) W.T. Correction (in case of presence of W.T.):

$$\text{For } N > 15: \; N_{corr.} = 15 + 0.5(N_{field} - 15) \quad \textbf{(2.7)}$$

and

$$\text{For } N \le 15: \; N_{corr.} = N_{field} \quad \textbf{(2.8)}$$

• ***If N-value is measured above water table, no need for this correction.***

(2) Overburden pressure, C_N; Energy ratio, η_1; Rod length, η_2; Sampler; η_3; and Borehole dia., η_4 Corrections:

$$N'_{70} = N_{field} . C_N .. \eta_1 .. \eta_2 .. \eta_3 .. \eta_4 \quad \textbf{(2.9)}$$

where,

N'_{70} = corrected (N) using the subscript for the energy ratio E_{rb} and (') to indicate it has been adjusted or corrected,

C_N = adjustment for overburden pressure for $\bar{p} \geq 25.(\text{kPa})$ and can be calculated from the following formula:

$$C_N = 0.77 \log \frac{2000}{\bar{P}_o} \quad \textbf{(2.10)}$$

- ***If*** $\bar{p} < 25.(\text{kPa})$***, no need for overburden pressure correction.***

where,

$\bar{p}_o$ = overburden pressure in (kPa),

η_i : *factors obtained from* **(Table 2.9)** *as*:

η_1 = hammer correction = (average energy ratio)/(drill rig energy) = E_r / E_{rb} ;

η_2 = rod length correction;

η_3 = sampling method correction; and

η_4 = borehole diameter correction.

Table (2.9): Hammer, borehole, sampler, and rod η_i correction factors.

Hammer correction η_1					
Average energy ratio E_r					R-P = Rope -Pulley: $\eta_1 = E_r / E_{rb}$ For USA trip/auto w / E_r = 80 η_1 = 80/70 = 1.14
Country	Donut		Safety		
	R-P	Trip	R-P	Trip	
USA					
North America	45	----	70-80	80-100	
Japan	67	78	------	-------	
UK	----	----	50	60	
China	50	60	------	-------	
Rod length correction η_2					
Length	> 10m 6-10 4-6 0-4		η_2 = 1.00 = 0.95 = 0.85 = 0.75		N is too high for L <10 m
Sampling method correction η_3					
Without liner: With liner: Dense sand, Clay Loose sand.			η_3 = 1.00 = 0.80 = 0.90		N is too high with liner
Borehole diameter correction η_4					

Hole diameter	60-120 mm 150 mm 200 mm	η_4 = 1.00 = 1.05 = 1.15	N is too small for oversize hole

Notes:

1. It is evident that all $\eta_i = 1.0$ for the case of a small borehole, no sampler liner, length of drill rod >10 m and the given drill rig has $E_r = 70$. In this case the only adjustment is for overburden pressure (i.e., $N_{corr.} = N_{field} . C_N$).
2. Large values of E_r decrease the blow count (N) linearly (i.e., $N_2 = \frac{E_{r1}}{E_{r2}} .. N_1$). This equation is used to convert any energy ratio to any other base.
3. If $N_{field} = 10...blows/10cm$, then $N_{corr.} = 10.\left(\frac{30}{10}\right) = 30...blows/30cm.$

2.12.3 Static or Cone Penetration Test (CPT)

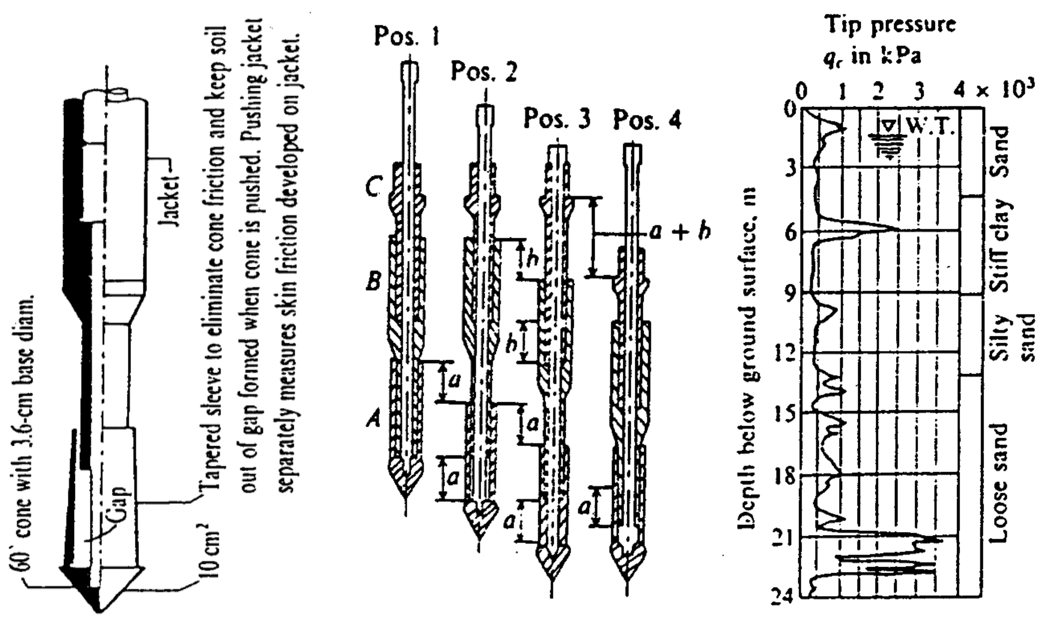
This is a simple static test used for soft clays and fine to medium coarse sands. The test is not applicable in gravels and stiff hard clays. It is performed by pushing the standard cone (according to **ASTM D3441** with a 60^o point and base diameter = 35.7mm with cross-section area of 10 cm^2) into the ground at a rate of (10 – 20) mm/sec. Several cone configurations can be used such as:

1. Mechanical or the earliest "Dutch Cone Type",
2. Electric friction with strain gauges,
3. Electric piezo for pore water measurement,
4. Electric piezo/friction to measure q_c, q_s and u or (pwp), and
5. Seismic cone to compute dynamic shear modulus.

Fig.(2.17b) shows the operations sequence of a mechanical cone as: in position (1) the cone is seated; position (2) advances the cone tip to measure q_c; position (3) advances the friction sleeve to measure q_s ;and position (4) advances both tip and sleeve to measure $q_t = q_c + q_s$ .Therefore, at any required depth, the tip and sleeve friction resistances q_c and q_s are measured and then used to compute a friction ratio f_R as:

$$f_R(\%) = \frac{q_s}{q_c} x100 \;;\; f_R < 1\% \text{ for sands; } f_R > 5 \text{ or } 6\% \text{ for clays and peat.}$$

The data collected from the CPT can be correlated to establish the undrained shear strength S_u of cohesive soils, allowable bearing capacity of piles, to classify soils; and to estimate $\phi, .. D_r$ for sands. A typical data set is shown in **Fig.(2.18b)**.

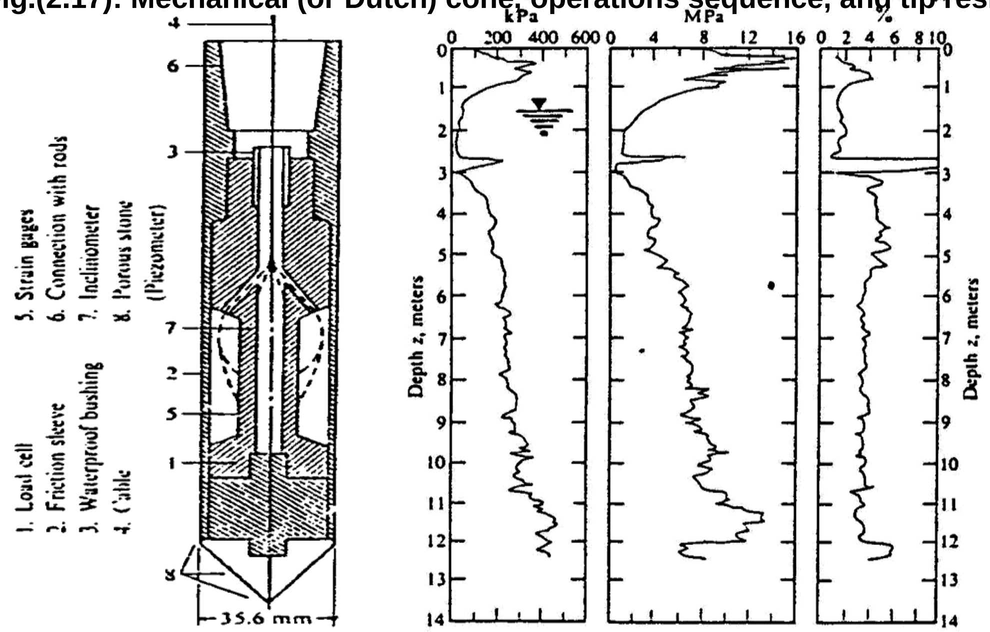


(a) Dutch cone modified to measure both point resistance q_c and skin friction q_f.

(b) Positions of the Dutch cone during a pressure record.

(c) Typical output.

Fig.(2.17): Mechanical (or Dutch) cone, operations sequence, and tip resistance data.



(a) Piezocone.

(b) Cone Penetration record for clay soil.

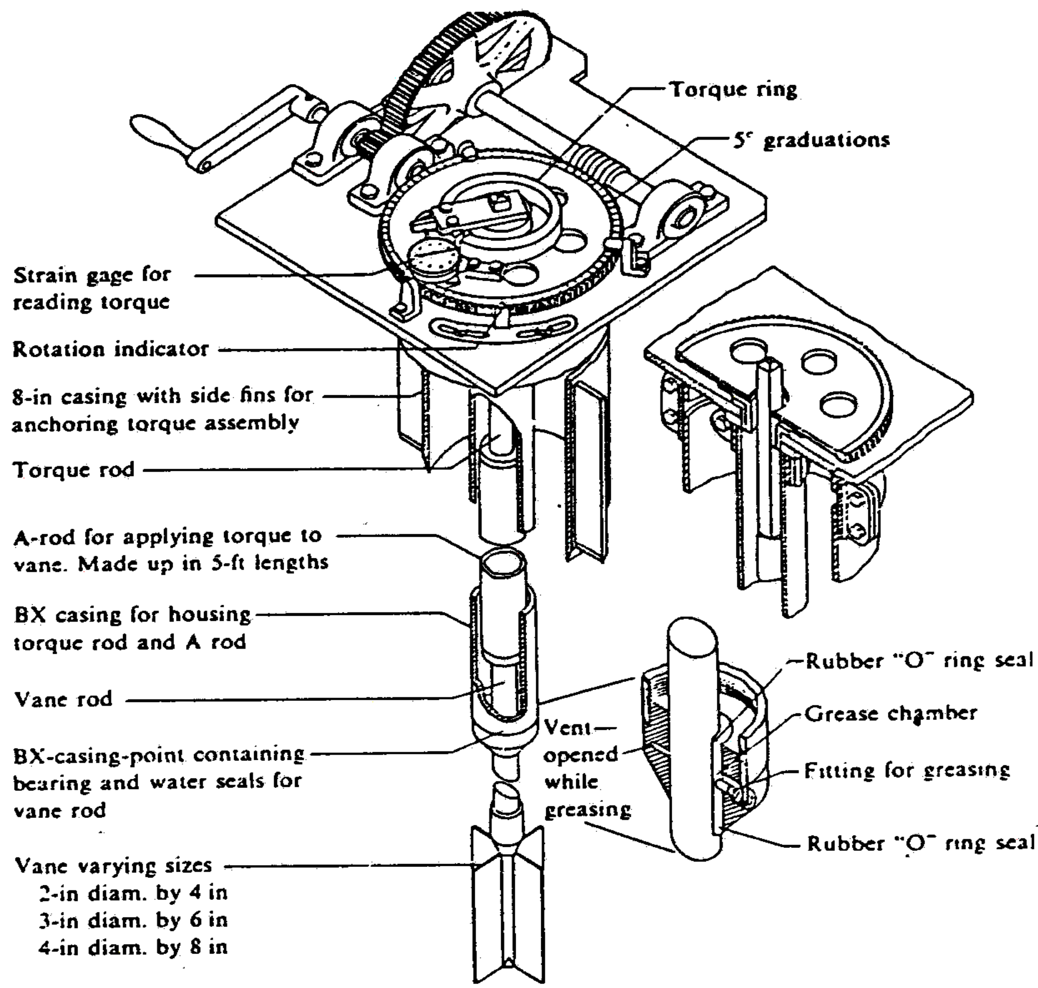
Fig.(2.18): Electric cone and CPT data.

2.13 VANE SHEAR TEST

It is a field test used to determine the in-situ shearing resistance(undrained shear strength) of soft to medium clay and silt clay having U.C.S.< 1.0 (kg/cm^2), then to be used for design of foundations and slopes.

- **Apparatus (see Fig.(2.19):**

1. Van shear test equipment;
2. Drilling rig;
3. Casing (as required); and
4. Other necessary tools and supplies such as stop watch, pipe,… etc..

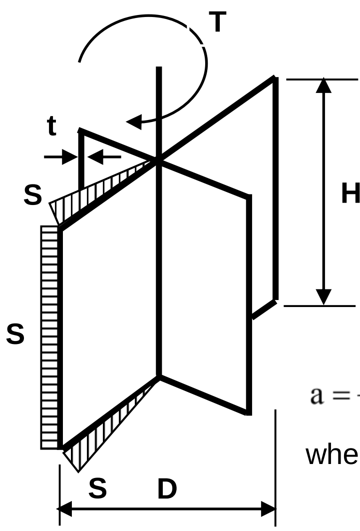


The Bureau of Reclamation vane-shear test aparatus. [*Gibbs et al.* (*1960*), *courtesy of Gibbs and Holtz of the USBR*.)

Fig.(2.19): Vane shear apparatus.

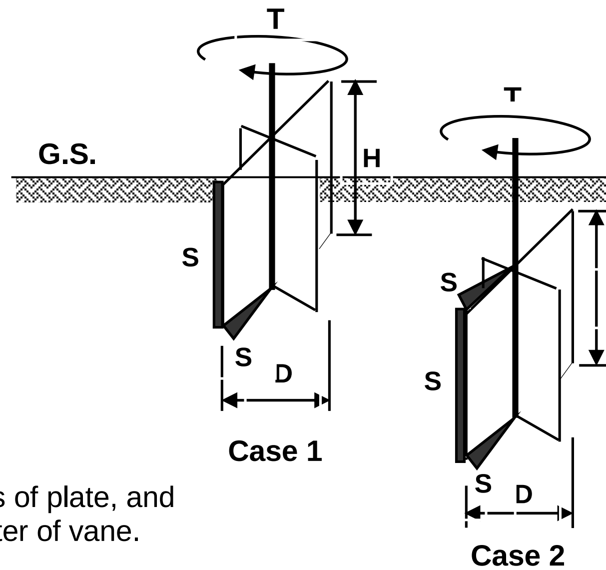
- ## Procedure:

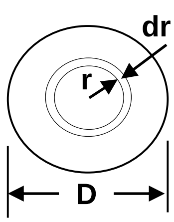
1. The equipment is installed in place properly either at the ground surface without a hole **(case 1)** or at the bottom of a borehole **(case 2)**and then the vane is pushed into the soil layer to the required depth; **(see Fig.(2.20)).**
2. A torque is applied at a uniform rate of 0.1^o per sec. or (1^o-6^o per minute).
3. Readings are taken each minute interval until failure happens.



$$a = \frac{4t}{\pi.D} \le 11\%$$

where,t = thickness of plate, and
D = diameter of vane.





Standard dimensions of vane B.S. 1377 Rate of test (6 -12)deg./ min.		
Soil strength (kPa)	H (mm)	D (mm)
< 50	150	75
50-75	100	50
> 75	Not suitable	

Fig.(2.20): Vane shear standards.

- ## Calculation:

(i)Case (1): In this case, the vane is not embedded in soil, so that only the bottom end takes part in shearing. If the soil is isotropic and homogenous, then:

(a) Total shear resistance at failure developed along cylindrical surface = $\pi.D.H.S$

(b) Total resistance of bottom ends, considering a ring of radius r and thickness dr

$$= \int_{0}^{D/2} (2\pi.r.dr).S$$

(c) The torque T at failure will then equal: $T = (\pi.D.H.S)\dfrac{D}{2} + \int_0^{D/2}(2\pi.r.dr).S.r$

$$\text{or}\; T = \frac{\pi.D^2 S_u}{2}\left(H + \frac{D}{6}\right) \quad \text{.......................................}\textbf{(2.11)}$$

(ii) Case (2): If the top end of the vane is also embedded in soil, so shearing takes place on top and bottom ends:

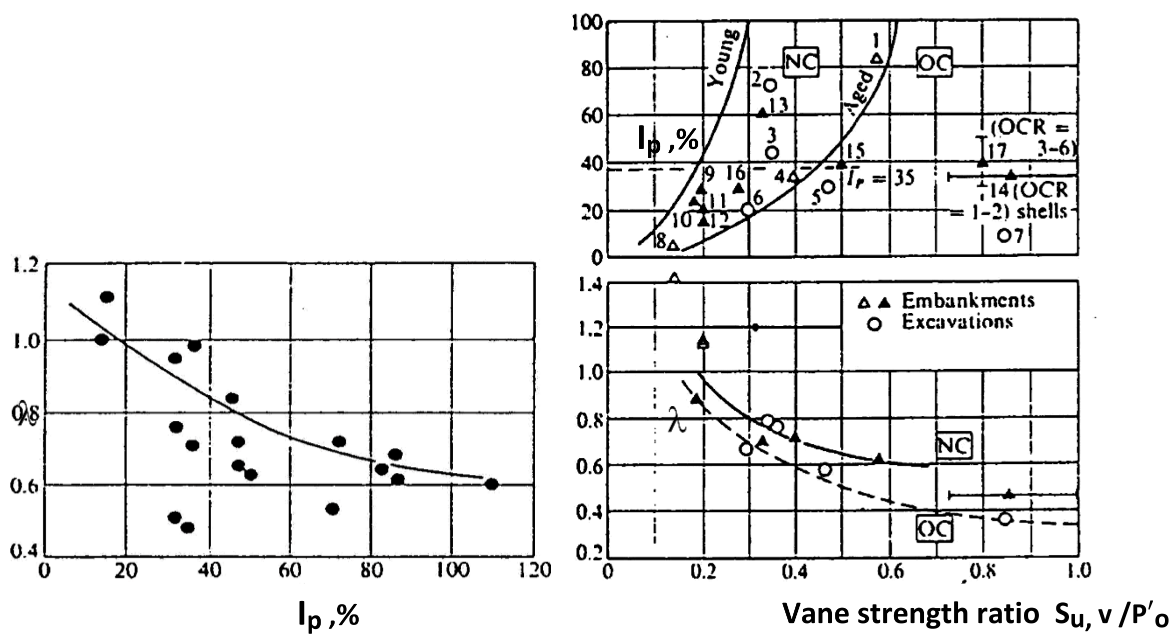
$$\text{or}\; T = \frac{\pi.D^2 S_u}{2}\left(H + \frac{D}{3}\right) \quad \text{.......................................}\textbf{(2.12)}$$

Notes:

- Use consistent units, such as: T in (kg-cm); S_u in (kg/cm^2);and H and D in (cm).
- It is found that the S_u values obtained by vane shear test are too large for design. Therefore, **Bjerrum's (1972)** proposed a reduction factor using the following formula:

$$S_{u\,,\text{design}} = \lambda..S_{u\,,\text{field}} \quad \text{.......................................}\textbf{(2.13)}$$

where, λ is a correction factor depends on plasticity index I_p and obtained from **Fig.(2.21a)**; Also, **Aas et al. (1986)** proposed another charts **(see Fig.(2.21b))** taking into account the effects of aging and OCR (Overconsolidation ratio).



(a) Bjerrum correction factor for vane-shear test.
[(Bjerrum, 1972) and Ladd etal., 1977)].

(b) Reinterpretation of the Bjerrum chart of part a by
(Aas et al. (1986) to include effects of aging and OCR).

Fig.(2.21): Vane shear correction factor λ .

SOLVED PROBLEMS

Problem (2.1): A thin-walled tube (OD = 76.2mm, ID = 73mm) was pushed into a soft clay at the bottom of a borehole a distance of 600mm. When the sampler was recovered a measurement done inside the tube indicated a recovered sample length of 575mm. Calculate the recovery and area ratios.

Solution:

Recovery ratio: $L_r = \frac{575}{600} = 0.958$

Area ratio: $A_r = \frac{(76.2)^2 - (73)^2}{(73)^2} x100 = 8.96\%$

Problem (2.2): A three story steel frame office building will be built on a site where the soils are expected to be of average quality and uniformity. The building will have a (30m x 40m) footprint and is expected to be supported on spread footing foundations located about (1m) below the ground surface. The site appears to be in its natural condition, with no evidence of previous grading. Bedrock is several hundred feet below the ground surface. Determine the required number and depth of the borings.

Solution:

- **Number of borings:**

  From **Table (2.5)**, one boring will be needed for every 200 to 400 m^2 of footprint area.
  Since the total footprint area is 30 x 40 =1200 m^2, use **(4) four borings**.

- **Depth of borings:**

  For subsurface condition of average quality, the minimum depth is:

  $5.S^{0.7} + D_f = 5(3)^{0.7} + 1 = 12m.$

  However, it would be good to drill at least one of the borings to a slightly greater depth to check lower strata. In summary, the exploration plan will be 4 borings with, **3 borings to 12 m, and 1 boring to 16 m.**

Problem (2.3): Given: Available information about:

Structure: Multistory building with 3 stories and basement

No. of columns = 16, Column load = 1000 kN

Raft dimensions: 16m x 16m x 1m, Foundation at 3m below G.S.

Soil profile: γ_d = 16 kN/m^3 , γ_{sat} = 20 kN/m^3 , W.T. at 6m below G.S.

Required: Number, layout, and depth of B.Hs.?

Solution:

- **Number and layout of borings:**

From Table (2.4b)**,** for poor quality and/or erratic subsurface conditions, one boring is needed for every (100 to 300) m^2 of footprint area. Since the total footprint area is 16x16=256m^2> 200m^2 (average value), **use one or two borings.**

- **Depth of borings:**

(a) $d = 1.5(16) = 24m$

(b) 10% of contact pressure:

$$q_{contact} = \frac{16.(1000) + 24(16)(16)(1)}{(16)(16)} - (3)(16) = 38.5..kPa$$

$$0.1.(38.5) = \frac{38.5(16)(16)}{(16+d)^2}, \ldots\ldots\ldots d = 34.6m$$

(c) 5% of overburden pressure:

$$0.05.[16(6) + (d-3)(20-10)] = \frac{38.5(16)(16)}{(16+d)^2}, \ldots\ldots d = 15.5m$$

From (**b** and **c**) take the smaller d = 15.5m

(d) $d = 6.S^{0.7} = 6.(4)^{0.7} = 15.83m$

From all (24m, 15.5m, and 15.83m) take the larger d = 24m

$\therefore$ use...$D = 24 + 3 =$ **27m from G.S.**

Problem(2.4): A wide strip footing applying net pressure of 35 kPa is to be constructed 1m below the surface of uniform soil having unit weight of 19 kN/m^3. The footing is 5m wide and the water table is at ground surface. Is 12m depth of boring (measured from ground surface) sufficient for subsoil exploration program.

Solution:

(a) $d = 3(B) = 3(5) = 15m$

(b) 10% of contact pressure: $0.1.(35) = \frac{(35)(5)(1)}{(5+d)(1+d)}, \ldots\ldots d = 4.3m$

(c) 5% of overburden pressure: $0.05(9 + 9d) = \frac{(35)(5)(1)}{(5+d)(1+d)}, \ldots\ldots d = 5.2m$

From (**b** and **c**) take the smaller d = 4.3m

From all(15m, and 4.3m) take the larger d =15m, and so the depth from ground surface $D = 15 + 1 = \mathbf{16m}$, $\therefore$ **12m is not sufficient.**

Problem (2.5): A standard penetration test SPT has been conducted in a coarse sand to a depth of 4.8m below the ground surface. The blow counts obtained in the field were as follows: 0 – 6 in: 4 blows; 6 -12 in: 6 blows; 12 -18 in: 8 blows. The test was conducted using a USA-style donut hammer in a 150mm diameter boring with a standard sampler and liner. If the vertical effective stress at the test depth was 70 kN/m^2, determine N'_{60} ?

Solution:

The raw SPT value isN = 6 + 8 = 14

Since $p'_o = 70...kPa > 25$ kPa $\therefore C_N = 0.77.\log_{10}\dfrac{2000}{70} = 1.12$

From **(Table 2.9):**

$\eta_1 = E_r / E_{rb} = 45/60 = 0.75$

$\eta_2 = 0.85$(for $L = 4.8$m (rod length) < 6m),

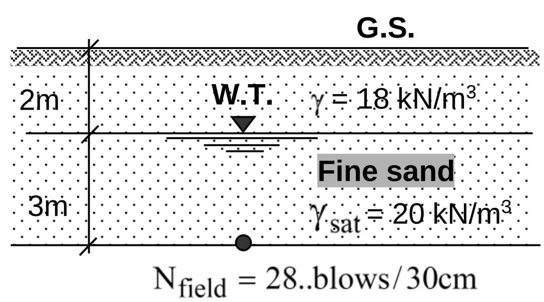
$\eta_3 = 0.90$ (for loose sand with liner),

$\eta_4 = 1.05$ (for B.H. diameter = 150mm),

$N'_{60} = N_{field}..C_N..\eta_1..\eta_2..\eta_3..\eta_4$ = 14(1.12)(0.75)(0.85)(0.90)(1.05) = **10blows**

Problem (2.6): A standard penetration test was carried out in sand at 5m depth below the ground surface gave (N = 28) as shown in the figure below. Find the corrected N-value?

Solution:



- **Water table correction:**

For N > 15 ...... $N' = 15 + 0.5.(N_{field} - 15)$

N' = 15 + 0.5(28 -15) = 21

- **Overburden correction:**

P'_o = 2(18) + 3(20 – 9.81) = 66.57 kPa $>$ 25 kPa

$\therefore C_N = 0.77\log\dfrac{2000}{P'_o} = 0.77\log\dfrac{2000}{66.57} = 1.14$

$\therefore N'_{corr.} = N'..C_N$ = 21(1.14) = **23 blows**

Problem (2.7): It is proposed to construct a spread wall footing of (3m width) in sand at (1.5m) below the ground surface to support a load of 12 Ton/m. The SPT results from a soil boring are as shown below. If the water table is located at 0.9m from G.S. and $\gamma_{soil(sat.)} = 17.6$ kN/m^3, determine the average corrected N-value required for design?

SPT sample depth (m)	1.5	2.25	3.0	3.75	4.5	5.25	6
N_{field}	31	25	22	20	28	33	31

Solution:

Find P'_o at each depth and correct N_{field} values up to at least a depth B below the foundation according to the magnitude of overburden pressure in comparison of 25 kPa.

Overburden pressure correction: $C_N = 0.77 \log \dfrac{2000}{P'_o}$

For 1.5m depth:

P'_o = 0.9(17.6) + (0.6)(17.6– 9.81)= 20.5 kPa $<$ 25 kPa, therefore, C_N =1.00

For 4.5m depth:

P'_o = 0.9(17.6) + (3.6)(17.6– 9.81) = 43.9 kPa $>$ 25 kPa, therefore, C_N =1.28

Find the average corrected N-value as a cumulative average down to the depth indicated, and then, choose the N-value for design as the lowest average N-value.

SPT sample depth (m)	N_{field}	P'_o (kN/m^2)	C_N	$N' = C_N . N_{field}$	$N'' = 15 + 0.5(N' - 15)$	$N''_{avg.}$
1.5	31	20.5	1.00	31	23	23
2.25	25	26.3	1.45	36	25	24
3.0	22	32.2	1.38	30	22	23
3.75	20	38.0	1.32	26	20	22
4.5	28	43.9	1.28	35	25	23

For 1.5m depth: $N'_{avg.} = 23$

For 2.25m depth: $N'_{avg.} = \dfrac{23+25}{2} = 24$

For 3.0m depth: $N'_{avg.} = \dfrac{23+25+22}{3} = 23$

For 3.75m depth: $N'_{avg.} = \dfrac{23+25+22+20}{4} = 22$

For 4.5m depth: $N'_{avg.} = \dfrac{23+25+22+20+25}{5} = 23$

N-value for design = $N'_{avg.}(\text{lowest}) =$ **22 blows**

Problem (2.8): The load-settlement data obtained from load test of a square plate of size (1ft) are as shown below. If a square footing of size (7ft) settles (0.75 inch), what is the allowable soil pressure of the footing? Consider sandy soil.

Load (Tsf)	2	5	8	10	14	16	19
Settlement (inch)	0.1	0.2	0.3	0.4	0.6	0.8	1.0

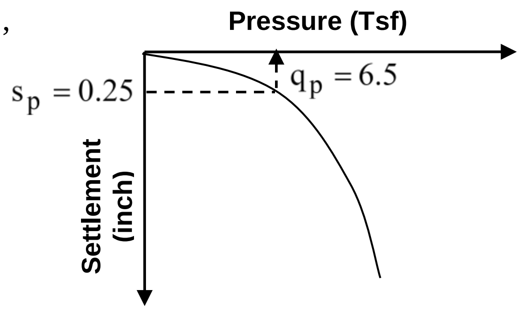
Solution:

For cohesionless soil:
$$\begin{cases} q_f = q_p \dfrac{B_f}{B_p} \\ s_f = s_p\left(\dfrac{2B_f}{B_p + B_f}\right)^2 \end{cases} , \therefore s_p = \left(\frac{0.75}{\left(\frac{2x7}{1+7}\right)^2}\right) = \frac{0.75}{3.05} = 0.25''$$

Now by drawing the given data and for $s_p = 0.25''$,

$q_p = 6.5$ T/ft^2,and

$q_f = q_p \dfrac{B_f}{B_p} = 6.5\dfrac{7}{1} =$ **45.5** T/ft^2 .



Problem (2.9): Use Housel method to determine the size of square footing required to carry a column load P = 45 tons if the two plate loading tests results are as given below:-

- Plate size (1) = 35x35cms, corresponding load= 5.6 tons; relative to 1.0 cm settlement.
- Plate size (2) =50x50 cm, corresponding load =10 tons; relative to 1.0 cm settlement.

Solution:

From **Housel's method**(Eq. 2.6):V = A. q + P. s

5.6 = 0.123 q + 1.4 s

10= 0.25 q + 2 s

Solving the two equations, gives: q = 26.9 and s = 1.63.

Again from Eq.(2.6) shown above, the footing area required to carry 45tons load is calculated as:

45 = B^2 q + 4B s

45 = B^2(26.9) + 4B (1.63)

26.9 B^2 + 6.52 B – 45 = 0

B^2 + 0.24 B – 1.67 = 0

$$B = \frac{-0.24 \pm \sqrt{(0.24)^2 + 4(1)(1)(1.67)}}{(2)(1)} = \frac{-0.24 \pm 2.59}{2} = 1.18\,\text{m}$$

Take the footing **1.20 m x 1.20 m**.

Problem (2.10): A vane tester with a diameter d = 9.1cms and a height h = 18.2 cms requires a torque of 110 N-m to shear a clay soil sample, with a plasticity index of 48%. Find the soil un-drained cohesion S_u ?

Solution:

For **CASE (2)** with top and bottom vane ends embedded in soil, the torque is given by:

$$T = \frac{\pi . D^2 . S_{u,field}}{2}(H + \frac{D}{3})$$

or

$$S_{u,field} = \frac{T}{\frac{\pi . D^2}{2}(H + \frac{D}{3})} = \frac{0.110}{\frac{\pi .(0.091)^2}{2}\left[0.182 + \frac{0.091}{3}\right]} = 40\,\text{kN/m}^2$$

From **Fig.(2.27a)** for a plasticity index of 48%, Bjerrum's correction factor λ = 0.80, and

Therefore, $S_{u,design} = \lambda . . S_{u,field} = 0.8(40) =$ **32 kPa**