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P R E F A C E

The Organising Committee appreciates the good response received from the invitees who were requested to present papers at this Seminar and wish to thank them for devoting their precious time to make the seminar successful.

A total of ten papers under three sessions have been received at the time of printing of this volume.

Papers received late will be included in a seperate volume along with discussions at the seminar and written discussions received till 20th January 1988.

The assistance given by Prof. A. Thurairajah and Dr. Jayantha Ameratunga, the members of the organising Committee & Dr. T. Sivapatham, in organising the Seminar and by Mrs. Julianne Bowen, Mrs. Marina Hannon, Miss. Deepani Senaratne and Mr. C.C. de Silva in preparing this volume is deeply appreciated.

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SEMINAR ON SHALLOW FOUNDATIONS FOR BUILDINGS

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SITE INVESTIGATIONS FOR SHALLOW FOUNDATIONS

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1. INTRODUCTION

Site investigations encompass all the techniques and investigations that can be used to obtain information on a particular site by the study of maps, site reconnaissance etc as well as by ground investigations, borehole investigations and other means.

Site investigation is often thought of in terms of boreholes but it is often not appreciated how much useful information can be gained from a brief office study, a trial pit or hand auger hole or an inspection of the site. This does not mean that specialized techniques do not provide useful information. They most certainly do and, on some occasions, it is essential to utilize them. It is therefore important to know what specialized techniques are normally adopted and when and how they can be gainfully employed. Most engineers have to work within a small budget and it is making the most of the available funds that reveals true skill, experience and professionalism.

Foundations can be divided into two classes - shallow and deep. A shallow foundation is one which is constructed immediately beneath the lowest part of the superstructure which it supports and generally it is less than 3 m below finished ground level. The choice of 3 m is abitrary. The depth/breadth ratio of shallow foundations are normally low. A deep foundation is one which is constructed considerably below the lowest part of the superstructure. There is no clear demarcation between the two classes and one merges into the other.

Shallow foundations may be divided into the following groups :-

1. Footings or spread footings.
2. Mats or rafts.

In the first type the base of a column or wall is enlarged to provide individual supports for the load. In the second type, a large number of loads in a spread out area are supported by a single slab. Types which include combined footings, (where several footings are joined to form a small mat) strap or strip footings (where footings are joined to form a long, narrow, continuous slab) fall in between.

The three basic requirements of a satisfactory foundation are as follows :-

1. It should be properly located, particularly the depth, with respect to any future influence which could adversely affect it.
2. It should be stable or safe from failure with respect to the strength of the soil beneath it.
3. It should not deflect or settle sufficiently to damage the structure or impair its usefulness.

The above facts should be considered when carrying out site investigations for shallow foundations. Shallow foundations investigation for light structures would involve only simple techniques of trial pitting or hand augering. However, for shallow foundations of heavy structures there are situations where techniques used in deep foundation such as diamond core drilling may have to be utilized even occassionally. Therefore techniques used in deep foundations are also discussed in this paper for completeness.

All site investigations should be systematically carried out and should be comprehensive according to the needs of the Engineering conditions. The cost of adequate exploration is usually one or two percent of the total cost of the work but varies with the type of structure and the nature of the ground.

2. GENERAL PROCEDURE FOR SITE INVESTIGATIONS OF FOUNDATION.

If a fairly detailed study is required, the following general procedure should be carried out during the course of a site investigation :

(1) Survey the general topography and the available access for construction vehicles and machinery.
(2) Investigate the location of buried services such as sewers, water mains, electric power, gas mains and telephone cables.
(3) Study the general geology of the area.
(4) Study the previous history and use of the site and neighbouring sites including any problems and failures associated with foundation conditions.
(5) Obtain information on any special features eg. climatic factors such as flooding, seasonal swelling and shrinkage or soil erosion.
(6) Survey the availability of construction materials such as aggregates, sand and water etc.
(7) For maritime or river structures, obtain information on high, low, tidal ranges, seasonal river levels and discharges, scour levels etc.
(8) Carry out subsurface exploration of the soil and rock strata and ground water conditions within the zones affected by foundation bearing pressures and construction operations or of any deeper strata affecting the foundation conditions.
(9) Carry out insitu tests.
(10) Obtain suitable samples and carry out laboratory tests on soil and rock samples required for the particular foundation design or constructional problems.
(11) Carry out chemical analysis on soil or ground water to investigate possible chemical effects on foundation structures.

Items (1) to (7) above, can be obtained from a general survey of the site and from a study of available information including maps, records etc. Items (8) to (11) are obtained from boreholes or other methods of subsurface exploration together with field and laboratory testing of soils, rocks and ground water.

After the completion of the site investigation a foundation engineering report has to be prepared. Most reports on foundation investigation follow nearly the same pattern. (Thomlinson, 1969).

3. SUBSURFACE EXPLORATION

This section is further discussed under the following:-

3.1 Borehole layout and depth.
3.2 Methods of subsurface exploration.
3.3 Description and classification of soil and rock.
3.4 Borehole records.

3.1 Borehole layout and depth.

Borehole layout and frequency are partly controlled by the complexity of the subsurface conditions. Suitable borehole layouts for various sites are given in Teng 1962, Thomlinson 1969. Borehole layout and frequency may need to be changed as more information is obtained.

The required number of boreholes which need to be sunk on any particular location is related to the relative costs of the investigation and the importance of the project. An economic limit is reached when the cost of borings out weigh any savings in foundation costs and merely adds to the overall cost of the project.

For major building projects when raft foundations are designed and for bridges, it is a good practice to sink at least one deep borehole to establish the solid geology. The depth to which boreholes should be sunk is governed by the depth of soil affected by foundation bearing pressures. The vertical stress on the soil at a depth of one and half times the width of the loaded area is still one-fifth of the applied stress at foundation level and the shear stress at this depth is still appreciable. Thus, borings in soil should be taken to a depth of at least one and half times the width of the leaded area. Deeper borings are required if soft compressible soils such as soft clays, organic silts, peaty soils and uncompacted fills are suspected to be present below these depths. For example, it has been found out that peaty soil strata are present even at depths of 12 to 15 m in some areas of Colombo. Therefore in these areas if a raft is to be constructed, it is prudent to investigate up to bedrock which is found generally between 20 to 30 m. Where foundations are taken down to rock, either in the form of strip or mat foundations, it is necessary to prove that rock is in fact present at the assumed depths. Where the rock is shallow this can be done by direct examination of exposures in trial pits or trenches, but when the borings have to be sunk to locate and prove bedrock it is important to ensure that boulders or layers of cemented soils are not mistaken for bedrock. This necessitates percussion boring or rotary diamond core drilling to a depth of at least 3 m in bedrock in areas where boulders are known to occur.

3.2 Methods of subsurface exploration.

Methods of determining the stratification and engineering characterities of subsurface soils are as follows:-

1. Trial pits and trenches.
2. Auger borings including hand auger borings (post-hole auger).
3. Percussion borings.
4. Wash borings.
5. Rotary core drilling.
6. Geophysical.

3.2.1 Trial pits and trenches.

Trial pits are the cheapest method used for exploration to shallow depths. They are preferable to boreholes in dry ground which require little support. Pits can be excavated by hand using any local labour but small mechanical excavators when used are rapid in operation. If it is necessary for men to work at the bottom of pits then support of the sides of pits deeper than 1½ m should be provided if there is the slightest risk of failure. Alternatively, it may be possible to excavate the sides to a safe profile by means of a series of benches. The maximum depth of excavation is about 5 m. In sands, there is likelihood of difficulty in excavating below the water table. Trial pits and trenches permit the insitu conditions of the ground to be examined in detail and provide access for taking samples and for carrying out insitu tests. They are the only reliable means of obtaining adequate information on filled ground.

3.2.2 Auger Borings.

Shallow borings are generally made by means of hand augers of the Posthole or Iwan type. By this method auger holes can be sunk upto about 6 m in peaty soil; 4 m soft clayey soil; and 5 m in sandy soil (above the water table because the material will not adhere to the auger). If the hole fails to stand open because of caving or squeezing from the sides, a casing with an inside diameter slightly larger than the diameter of the auger could be driven and the augering continued.

Power augering as a method of conducting preliminary subsoil investigations can be considered quick and easy. The advantages are low cost, speed and mobility.

With continuous flight augers, depths of 30 m or more are possible depending on sub-soil conditions. The variety of cutter heads and augers presently available permit successful augering in most soils. Hollow stem auger, a variation of the continuous flight auger, permit sampling below the bottom of the auger without removing the auger from the holes. Typically, augers with hollow stem of approximately 75 mm and 125 mm diameter produce boreholes of about 150 mm and 250 mm respectively to depths of 30 m to 50 m.

3.2.3 Percussion Borings.

Light cable percussion boring rig is commonly used for vertical boring upto about 25 m in depth. This is an adaptation of standard well boring method. Light cable percussion boring is suitable for soil and weak rock. The sizes of boring casing and tools are generally 100 mm, 150 mm, 200 mm and 300 mm, giving a maximum borehole depth of about 60 m in suitable strata. The tools consist of clay cutters for clays and shells or balers for sandy soils and chopping bits for small boulders and thin strata of rock. When the boring is in soils, SPT can be carried out and cores can be obtained at intervals.

3.2.4 Wash Borings.

Wash boring is best suited to sandy silts and clays and is normally carried out using borehole tools and borehole casings. The drill rig consists of simple winch and tripod. In this method, a tube is sunk by means of a strong jet of water issuing from a pipe lowered down the hole. The properties of soils are determined by carrying out standard penetration tests. Open tube samples or piston samples can be taken in cohesive soils.

3.2.5 Rotary Core Drilling.

Rotary open hole drilling and rotary core drilling are the traditional methods of drilling for rock exploration in which the drill bit cuts the rock by rotating at the bottom of the borehole. The drilling water, and air or mud which is pumped down to the bit through hollow drill rods lubricates and keeps the bit cool and flushes the debris up to the ground level.

There are two basic types of rotary drillings :

1. Open hole drilling in which the drill bit cuts all the material within the diameter of the hole.
2. Core drilling in which an annular core bit cuts a core using a double tube core barrel.

A rotary core drill working in conjunction with a pressure pump and borehole casings, drill rods, core barrels and other drilling accessories along with standard penetration test apparatus and equipment to take 'undisturbed' samples can be regarded as the most satisfactory equipment for foundation investigation of civil engineering structures.

Rotary drill equipment is manufactured in a wide variety of forms, which vary from light weight skid or truck mounted to heavy stationery plants with drilling capacities to depths of hundreds of metres.

Essential accessories for a drilling rig are a pump for circulating water to the corebits and for flushing the hole; a cathead winch and derrick for driving casing and for hoisting and lowering, and the necessary driving weights, drill rods, corebits and core barrels.

The sizes of casings, drill rods and other drilling accessories used in soil and core borings have been standardised. These sizes are generally used throughout the world, and they conform to the Diamond Core Drill Manufacturers Association DCDMA standard sizes. The standard sizes used in soil exploration are designated EW, AW, BW, NW and HW casings, EW, AW, BW and NW drill rods, EWG or EWM, AWG or AWM, BWG or EWM, NWG or NWM core barrels and corebits. The core barrels of the single and double tube type are provided at their lower ends with detachable shoes or core bits which carry tungsten carbide inserts for drilling of over burden soil or industrial diamond chips in a matrix of metal for rock coring.

3.2.6 Geophysical methods.

This method is a specilized subject, but given suitable ground conditions it can be made to produce a cheap and rapid means of detecting anomalies or variation in strata in site or between boreholes. These methods depend on measuring variations in certain physical properties of soils and rocks, such as resistivity, speeds of shock waves (seismic), gravitational field (gravimetric) and the magnetism. They work best when there is a marked difference in these properties. There are number of specilist techniques for overwater investigations. These are described in standard test books M.D. Joyce 1982 and C.K.I. Clayson, N.E. Simons & M.C. Mathews 1982.

3.3 Description and classification of soils and rocks.

It is important to establish a universally understood method of sample description and classification and its relevance to site investigation problems. This will enable the engineers to draw conclusions from available knowledge of the type of material.

For Civil Engineering purposes, engineers divide geological deposits into two major groups soils and rocks.

There are many different systems of soil classification, some of which are intended for a specific purpose. One of the most up to date methods of sample description is described in the British Code of Practice for Site Investigation (BS 5930).

3.4 Borehole records.

During the sinking of boreholes at the site, logs are prepared, giving a record of the description of soil or rock as determined by visual examination. All other data, such as changes in strata, depth, ground water level, samples taken and record of insitu tests are also given in this log. This gives preliminary information to the designer and enables the laboratory programme to be drawn up.

The final borehole record is prepared from the field log with the description of soils and rocks amended where necessary in the light of information obtained from laboratory examination and testing.

4. INSITU TESTS

As it is difficult to obtain relatively undisturbed samples from some soil strata, the engineering properties of such soils are best determined by measuring the relative density or shear strength by a insitu test. They are particularly useful in soft sensitive clays and silts or loose sands. The following insitu tests are briefly described in this section :-

4.1 Standard penetration tests.
4.2 Dynamic cone penetration tests.
4.3 Vane shear tests.
4.4 Plate bearing tests.

4.1 Standard Penetration Tests.

The standard penetration test is carried out in boreholes by means of a standard 50 mm outside diameter split spoon sampler. It is an open-ended steel cylinder which splits longitudinally into two halves. These two halves are held together by a cutting shoe at the lower end and a coupling connects the sampler to the drill rods. The sampler is driven through to a depth of 450 mm by repeated blows of a 63.5 kg. monkey falling through 750 mm. The number of hammer blows required to drive the second and third 150 mm of penetration is called the standard penetration resistance N which represents the number of blows per 300 mm. After the blow counts are recorded, the spoon is withdrawn from the borehole and the tube is taken apart for examination of the contents.

There are many factors which can give rise to give erroneous N values when carrying out standard penetration tests. (Joyce, 1982). Precautions should be taken to eliminate them whilst carrying out these tests.

The results of the standard penetration test can usually be correlated in a general way with physical properties of the soils. (Terzaghi & Peck 1948 & 1967, Peck, Hanson and Thornburn, 1974. Gibbs and Holtz 1957).

bridges, building civil engineering structures and road and airport runway embankments whilst working in the former Highways Department and the Road Development Authority is discussed in this section. In this paper it is proposed to discuss these investigations under the following topics :-

6.1 Shallow Foundations for buildings and other structures.
6.1.1 Low lying areas in and around Colombo.
6.1.2 Lateritic soil areas in and around Colombo.
6.1.3 Other soil areas.

6.2 Shallow foundations for bridges.

6.1 Shallow Foundations for buildings and other structures.

6.1.1 Low lying areas in and around Colombo.

Several deposits of peat are found in and around Colombo, particularly in low lying areas liable to flooding. Generally such areas are associated with high water-table and a mat of grass, moss etc. would normally be above the deposits of peat.

These deposits, however, are of significance to the engineer only when the need to construct buildings, roads or other structures over such deposits becomes unavoidable. In the City of Colombo and the suburbs there are many areas of peat and peaty soil where examples of single to four storeyed low cost housing units, 3 to 4 storeyed office buildings and light to heavy industrial structures are being constructed and are planned to be constructed in the near future. Under the major development scheme embarked in the late 1970S, these peaty soil areas were earmarked to be developed by the year 2001. These include investigations for housing schemes, road embankments, bridges and other civil engineering structures in Peliyagoda, Orugodawatta, Maligawatta, Rajagiriya, Baththaramulla, Yakbedda, Nawala, Kolonnawa, Narahenpita, Wattala and Jaela.

The boreholes were sunk by using a manually operated percussion boring rig capable of drilling about 15 m. Standard penetration tests were carried out within the boreholes in order to determine the strength of the soil strata. Several hand auger holes even to a depths of 8 m were done to supplement the borings. Samples of soils were taken by manual means within the hand auger holes and the boreholes of the percussion boring rig by using shelby open tube samplers. A significant feature at some of these sites was the inaccessibility of the borehole locations due to the inherent soft nature of the peaty soil and the high water table. However this problem was overcome by constructing a pathway with timber planks for the workmen to carry the drilling equipment to the borehole locations.

At some of these sites for low cost housing it was seen that they have been previously filled with lateritic soils. At a site at Devi Balika Vidyalaya, Borella, no indications of peaty soils was seen from the surface of the ground other that the fact that there were no tall buildings in the area. Hand Auger holes carried out in 1½ m test pits revealed a 1½ m thick deposit of peaty soil overlain by a 1½ m thick layer of lateritic filled material.

Work done by K. Ray 1983, K.S. Senanayake 1986 and the author and others indicate that layers of soil present in these areas can be schematically represented as given in Figure 6.1.

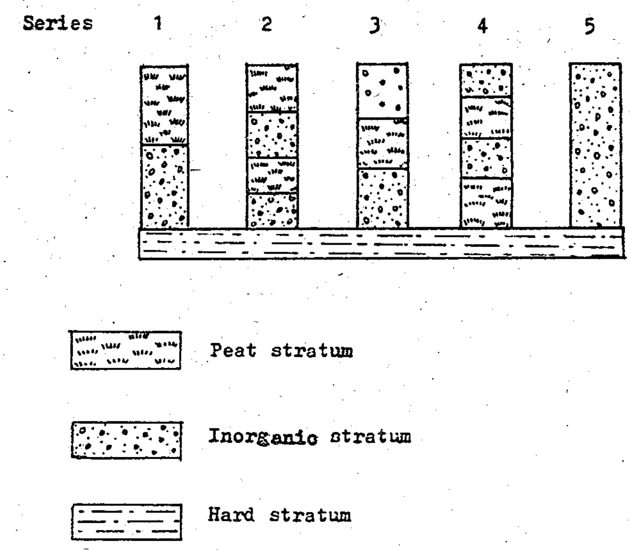


Figure 6.1 Schematic Representation of Strata in Low Lying areas in and around Colombo.

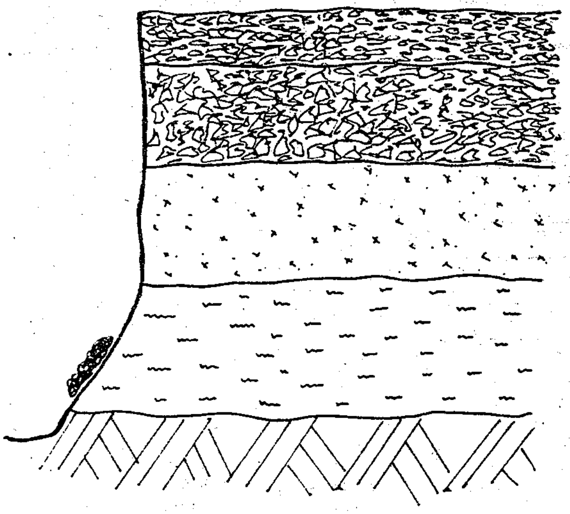
Also the first two studies stated above give approximate thicknesses of the several strata present in these areas. These studies and the studies done by the author indicate that the properties of the peaty soil vary greatly at different locations both with depth and location within them. Consequently, the depth and frequency of the investigation would have to be more extensive in these areas than in the normal investigations. Generally, bedrock is present within 10 to 20 m depth. In view of the fact that the second peaty soil layer could be found even at depths of 20 m, it may be necessary to investigate up to bedrock unless the building is a one or two storeyed structure, as otherwise the influence of the peaty layer in the computations of settlements will not be taken into account.

At these low lying sites, it is important to determine the highest flood level and to raise the ground floor or the buildings and the garden level of the premises well above this flood level so that it would always be above the flood level even after the completion of the long term settlement of the peaty soils due to weight of the fill and the foundations. Otherwise flood waters may enter the premises after several years.

6.1.2 Lateritic soil areas in and around Colombo.

Laterite can be defined as a tropically weathered, clayey, rock material which has decomposed partially or totally with a concentration of iron or aluminimum oxide but poor in silica; it is usually reddish, purplish, brownish or yellowish in colour. The laterite, locally known as cabook, is found in the South Western part of Sri Lanka within a belt extending six to seven miles inland from the coast and up to about 30 m above mean sea level (Cooray, 1967).

Good examples can be seen at several locations within this area, where the laterite is exposed in quarry faces and in road and railway cuttings. In nearly all these localities the residual soils can be seen to have developed by the weathering of many types of crystalline rocks such as charnockites, garnet-biotite gneisses and amphibolites. (Cooray, 1967). A typical cross-section of these laterities is given in Figure 6.2 (D.B Pattiaratchi and J.W. Hereth, 1963 quoted in Cooray, 1967).



(1) Ironstone cap forming a hard crust which, when exposed, breaks down to laterite nodules.
(2) Laterite hard and cellular with clay filled cavities.
(3) Soft to medium stiff yellowish brown silty clay predominantly kaolin with low plasticity with some lateritic gravels.
(4) Weathered rock.
(5) Parent rock

Figure 6.2 Cross section of a typical laterite.

The author was involved in the site exploration of the following projects in lateritie areas :-

(1) State Fertilizer Manufacting Corporation Site at Sapugaskanda along with Geological Survey and Irrigation Departments.
(2) Earth Sattelite Station, Padukka.
(3) Proposed hydrocracker project at Sapugaskanda.
(4) Nylon 6 project at Sapugaskanda.
(5) Overseas telecommunication tower, Colombo.
(6) Oruwela Steel Corporation - Extension to the rolling mill.
(7) Housing Scheme Site at San Sabestian Street, Colombo.

In all these projects, a Acker Terrodo skid mounted diamond core drill driven by a petrol engine with rotary wash and water flush techniques was used. At the first site, this machine was considered to be the best among the Joy 12B drills used by the Geological Survey and the Acker skid mounted rig but of non-rotary percussion type used by the Irrigation Department.

This Acker Terrodo rig used by the Highways Department had certain mechanical features which makes it more suitable for soils investigation work; in particular, hydraulic retraction of the power head thus facilitating sampling operations. Throughout the work at this site, this rig performed well. At this site, the shelby open tube samplers used by the Highways Department proved a quick and reasonably succesful sampling method, even in the more granular areas. The thin sharp edge of the sampler appeared to cut through the weaker laterite gravels rather than twist or push aside the particles and hence the resulting samples were less disturbed.

The significant feature at all these sites is that the standard penetration value reduced and the compressibility increased as the boreholes were sunk through horizons 1, 2 & 3 as given in Figure 6.2. The standard penetration tests values are not very reliable in horizons 1 & 2 containing gravel particles. If the spoon is obstructed by a large piece of gravel, or when a gravel particle is wedged inside the spoon, excessively large resistance may be expected. In coarse gravel deposits the split tube sampler tends to slide into the large voids giving low penetration resistance.

The significant feature at the site for the Earth Sattelite Station, Padukka was the drilling in rock of depths more than 6 m in order to prove bedrock for pad foundations. At other sites, a depth of 1 to 2 m was cut in rock to prove that there was bedrock.

6.1.3 Other soil areas.

Projects in other soil areas include :

(1) New Police Headquaters - Fort.
(2) Slave Island Housing Schemes.
(3) Fisheries Research Institute - Crow Island.
(4) Agrarian Research & Training Institute, Wijerama Mawatha.
(5) National Institute of Business Management, Wijerama Mawatha.
(6) Telecommunication Office & Stores, Maradana.
(7) Consolexpo site, Fort.
(8) Several sites for low cost housing schemes and buildings.
(9) Proposed abattoir at Alawwa.

The first site was investigated using the Acker Torredo diamond core drilling machine. The investigation revealed mainly sands, with some gravels and clay layers present with N values of 20 to 60 with bedrock at about 15 to 18 m depth from surface. An interesting feature of this investigation was the existence of 1½ m thickness of peaty soil at depths of 14 m, at one of the boreholes. This made the designer to change the raft foundation to that of piles in this area.

The sites (2), (3), (4), (5), (6) and (7) had very loose to medium sand close to the surface with rock levels at 15 to 30 m. In some places N values of less than 5 were obtained and therefore special precautions in design and construction were taken. At the Agrarian Training and Research Institute site, peaty soil was found in one corner of the building resulting in the construction of a pile in that area with considerable cost to the client and inconvenience to the construction staff. However this could have been avoided if the layout of the building could have been slightly changed by the architects.

Some of the investigations given in this section were carried out using a skid mounted Ace model 'C' diamond core drill with cathead hoist with a petrol driven engine. This is a compact portable drill which could be used, even in remote areas.

When carrying out SPTs at the sites with loose sand above water table, sufficient head of water in the borehole was maintained to prevent piping. Otherwise erroneous test results will be obtained.

Several site investigations for low cost housing schemes and buildings were done manually in and around Colombo by using the hand auger and shelby open tube samplers for clayey soils. Designs were given after testing the soil for strength and compressiblity. At the proposed Ratmalana Railway Housing Scheme Site, a dark brown sand stone layer at depths of 1½ to 2 m was found to be suitable for the construction of foundations.

At the Pettah Central Bus station site it was seen that this area had been previously filled with refuse and coal etc without any compaction. At these sites where haphazard or uncontrolled filling has been done previously, a detailed site investigation should be carried out.

At the proposed Aluwwa abattoir site investigations were carried out by means of hand augering inside test pits, for one and two storeyed buildings. In the filled areas, the degree of compaction(by carrying out field density tests) and strength of the fill material; and the strength and compressibility properties of the insitu silty clay, were determined. The investigations revealed that the construction of the new fill had been carried out (a) without the removal of the organic top soil of thickness 150 mm and (b) without any control of compaction. The recommendations gave the following alternatives :-

1. Construct the individual footings on the insitu soil below the organic layer.
2. Remove the new fill and layer of organic top soil and refill to required height according to standard specifications with proper quality control of construction.
3. Shift the site to avoid the filled areas.

The lesson to be learnt from the above is that when fillings are done it is necessary to remove the top soil and compact the fill according to standard specifications.

6.2 Shallow foundations for bridges and causeways.

One of the types of bridge foundations adopted in Sri Lanka is the shallow foundation. Spread foundations are the simplest and the most economical type of foundations, if rock or a very strong soil layer is present close to the bed level. If it is bedrock, the investigation must determine the profile of the sound bedrock for purposes of dowelling during construction. If it is a strong soil layer that is available, the scour depth of the river after the construction of the bridge should be assessed during the site investigations.

The Research & Development Division of the former Highways Department and presently Road Development Authority has carried out more than 100 foundation investigations of medium to large sized bridges in Sri Lanka in the recent past.

Some of these sites require investigations of foundations over water. The equipment and technique for drilling boreholes for these investigations are essentially the same as for land borings. In the case of investigations close to land or existing structures such as cylinders or in shallow water,trestles with B.I. pipes scaffolding to carry the rig were used. For boreholes up to 15 to 30 m from land,the trestles were extended from land to take the drilling rig. For greater distances it was economical to erect a trestle at the borehole position and lower the drilling rig as a unit from a crane or to place it on a pontoon and tow it to the borehole position. One of the methods which could be adopted for boring in water deeper than about 7.5 m or at some distance away from the seashore or banks or a river is to mount the drilling rig on a platform cantilevered over the side of a barge.

The following four bridges have been constructed with shallow foundations and would be discussed below :-

1. Bridge No. 44/1 Colombo - Kandy Road.
2. Bridge No. 6/5, Ratnapura - Palawala - Karawita Road.
3. Bridge No. 18/2, Ekala - Gampaha Road.
4. Bridge No. 12/7, Malabe - Athurigiriya Road.

In these investigations a Yoshida skid mounted diamond core drill driven by a diesel engine was used. The significant difference in the operation of this drill when compared with the petrol driven Acker drills is the more difficult maneuverability of the machine due to the bulkiness of the diesel engine.

In the first investigation, several layers of boulders were cut by the use of the following techniques. Firstly the cobbles and small boulders were demolished by means of chopping bits and removed from the borehole. Then the large boulders were drilled, casings of smaller diameter were inserted and the drilling was carried out with a smaller diameter core barrel. Bedrock was reached at varying depths of 1 to 6 m from the bedlevel of the stream.

At the second site, stiff clay followed by dense sand was found below a depth of 6 m with bedrock level at 30 to 40 m. The foundations were constructed on the stiff clay statum.

At the third site, strong lateritic gravel-sand-clay soil at a depth of about 3.5 m followed by soft clay at depths of 9 m with bedrock at 18 to 20 m was found. The shallow foundations were constructed on the strong lateritic layer at depth of 3.5 m.

At the fourth site, a dense sand was found at depth of 5.5 m, where the shallow foundations were constructed.

The proposed causeway of length 120 m on the Nainamadama - Thoduwawa Road was investigated by carrying out two boreholes using a manually operated percussion drilling machine supplemented with SPTs within the boreholes and nine DCP tests. The tests revealed a sandstone layer at a depth of 1.3 to 5 m from the surface overlain by loose sand. It was proposed to construct the foundations on this sandstone layer.

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REFERENCES

1. Thomlinson, M.J. (1969) "Foundation Designs and Construction", Pitman.
2. Teng, W.C (1962) "Foundation Design". Prentice-Hall.
3. Joyce,M.D. (1982) "Site Investigation Practice". E & F. N. Spon.
4. Clayson, C.K.I. Simons, N.E & Mathews, M.C (1982) "Site Investigation".
5. B.S 5930 (1981) "Code of practice for Site Investigations".
6. Terzaghi, K & Peck, R.B. (1948 & 1967), "Soil Mechanics in Engineering Practice". Wiley.
7. Peck, R.B., Hanson, W.E & Thornburm, T.H. (1974)- "Foundation Engineering" Wiley.
8. Canadian Foundation Engineering Manual (1978).
9. Cooray, P.G (1967) "The Geology of Ceylon". National Meseums of Ceylon.
10. Ray, K. (1983) "Geotechnical Problems Associated with Development of Low Lying Areas in and around Colombo". Journal of the Institution of Engineers, Sri Lanka.
11. Senanayake K.S (1986) "Geotechnical Mapping of Low-Lying Areas in and around Colombo City", Asian Regional Symposium of Geotechnical Problems and Practice in Foundation Engineering, Colombo, Sri Lanka.

SOIL SAMPLING FOR SHALLOW FOUNDATIONS

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INTRODUCTION

Shallow foundations for buildings can be classified into three basic types;

i) Isolated pad footings
ii) Strip footings and
iii) Raft or mat foundations.

In designing foundations for buildings at a site, the geotechnical engineer needs to assess subsoil conditions in the particular area. Basically, he needs to determine stratigraphy (layering), identification and classification of subsoil, physical properties and engineering properties such as strength and compressibility characteristics of underlying soil. Obtaining good quality soil samples from the ground and subsequent testing of them in the laboratory are essential requirements in order to evaluate these properties with great accuracy. Thus, soil specimens play a major role in decision making process in foundation design.

Classification of Samples & Sampling Techniques

Soil samples can be broadly classified into disturbed and undisturbed. In disturbed samples the fabric or structure is partially or completely destroyed, thus density, permeability strength and deformation characteristics cannot be evaluated in the laboratory from such samples. However, these samples can be used for identification, classification, particle size analysis, natural moisture content, specific gravity tests, etc. These disturbed samples can generally be obtained by less expensive sampling techniques. The undisturbed samples usually preserve the fabric or structure, thus suitable for all kinds of testing indicated above. However, samplers and sampling technique required to obtain undisturbed samples are sophisticated and expensive.

The soil specimens are expected to be representative of the soil deposits from which they are obtained. Hvorslev (1949) classified soil samples as either non-representative, representative, or undisturbed. This classification is made by comparing the state of the samples to that as exists in situ. According to Hvorslev's classification, a non-representative sample is one in which soils from different strata have been mixed, or some of the soil constituents of the sampled soil are missing. A representative soil sample is one in which there is no change in soil constituents but whose structure, water content, or void ratio have been altered. An undisturbed sample, ideally is one that represents the in situ conditions.

The variability of the materials encountered and the need of samples for various purposes have resulted in the development of many different techniques of sampling and type of samplers. The choice of the sampler and the sampling technique has been based on the type of soil, quality requirements and economical and practical demands. Sampling tools reflect the type of sample and soil to be obtained. Generally, samplers can be categorized into five basic types;

i) Exploration samplers
ii) Drive samplers or thick wall tubes
iii) Thin wall tubes
iv) Block samplers and
v) Core barrels.

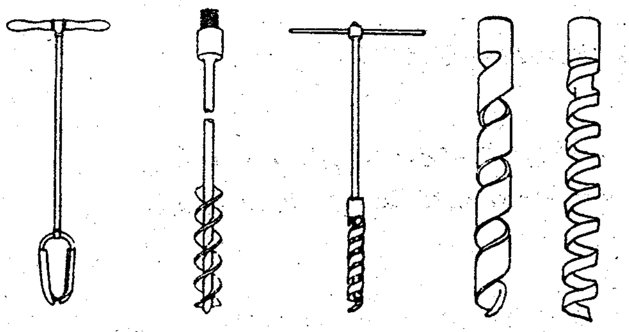
EXPLORATION SAMPLERS

Exploration samplers are generally used when the subsurface conditions are unknown at a proposed site and when the information on ground conditions, soil types etc. is required for preliminary planning. It gives relatively simple, but quick information of underground conditions. The exploration samplers or augers can be divided into two broad categories depending upon whether they are advanced manually or by a power rig.

Hand Auger Sampling

The most commonly used hand augers include the Iwan, helical, ship, closed spiral and open spiral. Generally the sample is obtained by pressing the auger into the ground and turning it at the same time. When the blades are loaded with all that can be held, the tool is withdrawn. As the hole progress downwards extension rods can be added.

The Iwan augers (Fig.1a) are available in diameters ranging from 3 in. (76 mm) to 9 in. (229 mm) and can be used to even a depth of 20 ft (6.1 m) to 25 ft (7.6 m). In general, they are useful for sampling all types of soils except cohesionless material below the water table and hard or cemented soils. The helical auger, which consists of a helical flight on a solid stem (Fig. 1b), is used in either cohesive or cohesionless soils above the water table.



(a) (b) (c) (d) (e)

Fig. 1 Hand Augers:
(a) Iwan auger; (b) Helical auger; (c) Ship auger; (d) Close spiral auger; (e) Open spiral auger.

The ship auger shown in Fig. 1c is best suited to use in cohesive materials. Both the helical and ship auger are available in diameters from approximately 2 in. (51 mm) to 3.5 in. (89 mm). Closed or open spiral augers were developed for use in those cases where helical and ship augers provide poor recovery of sample. The closed spiral auger (Fig. 1d) is used in dry clay and gravelly soils while the open spiral auger (Fig. 1e) is most useful in loosely consolidated deposits. These augers are available in the same size range as the preceding group.

Hand augers are used primarily in cases where there is no need for undisturbed samples and where the drilling will be in soils and the borehole will stay open without casing or drilling mud i.e., generally above ground water table. They are used extensively in surveys to obtain representative samples of the various materials present for general identification and classification purposes. Boreholes drilled with augers have the disadvantage that the samples are mixed and, in general it is difficult to locate the exact changes in the soil strata.

Power Auger Sampling

Soil samples can also be obtained by advancing power augers into the ground. Continuous flight augers, single flight cutter and spindle with auger samplers and hollow stem auger samplers are widely used. The first two use primarily for general soil survey purposes and the hollow stem auger can even be used for undisturbed sampling. The hole is advanced in the same manner with the power auger as with the hand auger.

Continuous Flight Auger

Continuous flight augers can be used to depths of 100 ft (30.5 m) or more depending on equipment available and condition encountered. They are available in sizes ranging from 2 in. (51 mm) to 48 in. (1219 mm) in diameter.

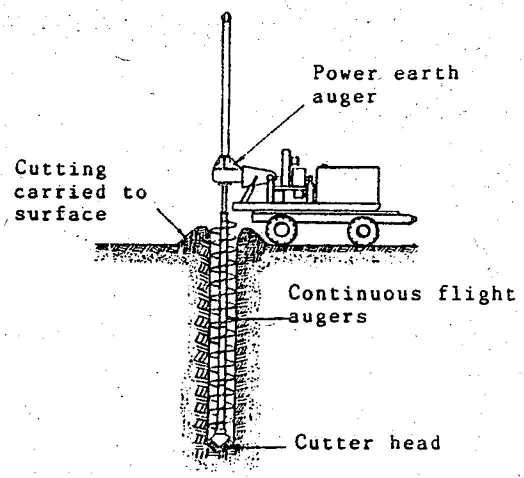


Fig. 2 Continuous Flight Auger

Fig. 2 shows a trailer mounted continuous flight auger in operation. Soil samples can be recovered by two methods when using continuous flights. Samples can be recovered from the cutting deposited at the top of the hole where the sample is not only highly disturbed but also from an unknown elevation. The second method is to pull the flight auger out of the hole and takes a sample from the material adhering directly to the auger and cutter head.

Single Flight Cutter & Spindle with Auger Samplers

Fig 3 shows a single flight cutter head mounted on the spindle of an auger drill. The cutter head is used to make repeated passes in and out of the hole limited only by the travel of the auger spindle. After the hole has been bored to a given depth, a special auger type sampler is attached to the spindle to obtain a better sample. Fig. 4 shows three types of auger samplers that are commonly used in practice.

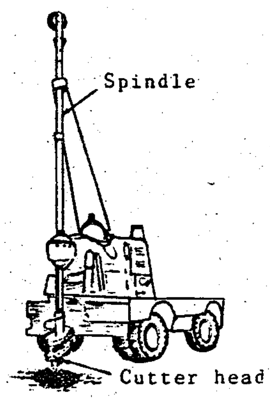


Fig. 3 Single Flight Cutter head & Spindle

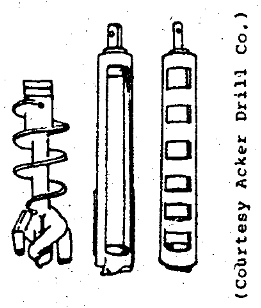


Fig. 4 Auger Samplers

Hollow Stem Auger

The hollow stem auger (Fig.5) provides a faster mean of advancing a hole through many types of soils by eliminating the use to remove auger during sampling. It provides the facility for readily obtaining undisturbed or representative samples..

The disadvantage of hollow stem augers, besides their cost is the size of the equipment required to operate them. In general appearence, hollow stem augers are very similar to the continuous flight auger except it has a large hollow center to collect sample.

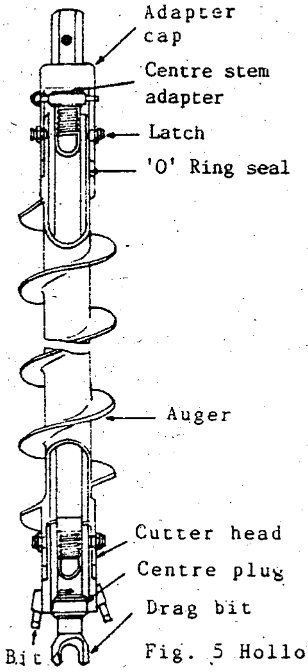


Fig. 5 Hollow Stem Auger

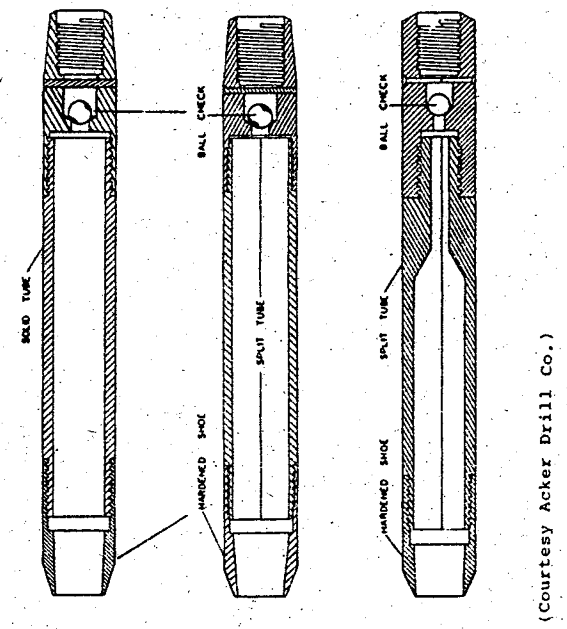
The uses of power auger sampling methods are for obtaining relatively fast general subsurface information and quick search for bedrock. Power augers other than the hollow stem type in which the auger flight remains in the hole and serves as a casing, are generally restricted to operations above ground water level unless casing is used.

DRIVE SAMPLERS OR THICK WALL TUBES

Basically drive sampler consists of a thicker steel tube, detachable hardened steel shoe with a cutting edge and an adapter head with a ball check valve. The ball check valve in the head section prevents samples from being washed out of sampler upon withdrawal from the hole. When taking the samples, the entire assembly which is connected to the drive rod is lowered into the bottom of the borehole where the sample is required. It is driven into the soil by dropping a drive weight usually 140 lbs (63.5 kg) or 300 lbs (136.4 kg) and when the drive is completed the sampler is rotated and retrieved from the borehole.

The area ratio is defined as the area of soil displaced by the sampler divided by the cross-sectional area of the sample (Hvorslev, 1949). Generally the thick wall drive sampler has an area ratio of about 25%. It produces a representative but disturbed sample. The state of the structure of the sample obtained by this technique is quite reasonably destroyed and used mainly for identification, classification, composition, Atterberg limit tests, moisture content determination, specific gravity tests etc. The thick wall sampler is more durable and can be used in a variety of soils.

Fig.6a illustrates a solid tube with the cutting shoe and adapter head assembled together. It has the advantage of simplicity and ruggedness. Its only disadvantage is that the sample must be pushed out of the tube, thus resulting in a broken-up specimen.



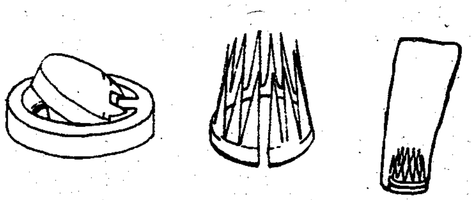
(a) (b) (c)

Fig. 6 Drive Samplers:
(a) Solid tube; (b) Split tube; (c) Lynac sampler

Fig. 6b. shows a split barrel sampler which is used in the Standard Penetration test. It is one of the most commonly used soil sampling devices. Major difference of this sampler from the solid tube is that it consists of a thick wall steel tube which is split longitudinally. Thus, when the adapter head and the cutting shoe are removed the barrel opens in two halves exposing the entire sample. In general, the split spoon samplers are available with inside diameters ranging from 1.5 in. (38 mm) to 4.5 in. (114 mm). Barrels are available in standard length of 18 in. (457 mm) and 24 in. (610 mm) and have a wall thickness of 0.25 in. (6.4 mm). The 1.5 in. (38 mm) sampler is popular since correlation have been developed between the number of blows (N), required for penetration of this sampler and the relative density of cohesionless soil and consistency and the shear strength of cohesive soils (Terzaghi et al, 1948).

Fig. 6c shows a thick wall sampler referred to as the Lynac sampler. It has heavier walls than the regular split barrel sampler and is better adapted to hard driving conditions due to its thickened head section.

Three types of sample retainers, which can be very useful in non-cohesive materials are shown in Fig. 7. The trap valve retainer (Fig. 7a) is used for thin mud and other watery substances. The spring sample retainer (Fig. 7b) is used for coarse sand and small gravel. Lad sample retainer (Fig. 7c) is ideally suited for free flowing sands, silts and other difficult materials.



(a) (b) (c)

Fig. 7 Sample Retainers

These sample retainers are inserted inside of the sampler between the cutting shoe and the sample barrel.

THIN WALL TUBE SAMPLER

Thin wall tube sampler which is commonly referred to as shelby tube is the simplest and most widely used of all the available undisturbed samplers. It consists of a thin wall tube screwed to an adapter head containing a ball check valve (Fig. 8). The head is threaded to receive standard drill rods. The ball check valve in the head section vents the inside of tube to the outside, permitting the rapid escape of air and fluid above the sample when the tube is pushed downwards. Undisturbed samples are recovered by pushing the tube into soils.

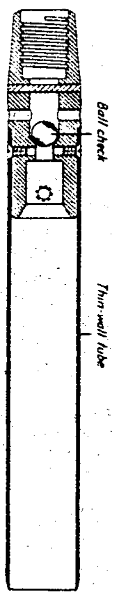


Fig. 8 Thin Wall Tube Sampler

The bottom of the tube is rolled inwards to provide an inside clearence of approximately 1% of the diameter, so that a sharp cutting edge is achieved. Since most cohesive soils have a tendency to swell during sampling, this 1% restriction assists in retaining the sample as it is withdrawn from the borehole.

Hvorslev (1949) defined a thin wall sampler as a sampling tube with a wall thickness less than 2.5% of the diameter, corresponding approximately to an area ratio of 10% when the inside clearance of the cutting edge is not taken into consideration.

The most commonly used shelby tubes are 2 and 3 in. (76 mm) in diameter and 24 in (610 mm) and 30 in. (762 mm) in length. However, the use of tubes in the 4 in. (102 mm) to 6 in. (152 mm) diameter range has increased in the past few years.

The thin wall tube sampler is used mainly for sampling soft to stiff cohesive soils. It provides a sample with minimum disturbance. However, it is easily damaged by the dense soil and hard objects in the soil such as gravel stumps etc. A major disadvantage of the use of steel for the sampler is the danger of corrosion. This can be avoided by treating the tube with a rust inhibitor prior to use. Sampling tubes of brass and stainless steel also have been used to minimise or avoid corrosion.

THIN WALL PISTON SAMPLER

The thin wall stationary piston sampler (Fig. 9) is basically a thin wall tube sampler with a piston, piston rod and modified sampler head. It has been found useful in reducing the effects of some of the disturbing forces during sampling.

The piston sampler is placed on the bottom of the hole with the piston flush with the bottom end of the thin wall tube. The tube is then advanced beyond the piston into the undisturbed soil. The piston assists the sampling operation by preventing the entrance of excess soil into the tube and by helping to hold the sample in the tube during recovery. Good contact between the piston and top of the sample develops a greater vacuum and prevent loss of sample when withdrawing. The piston samplers are further classified into three types according to the operation of control procedure of the piston.

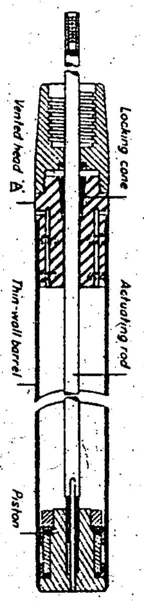


Fig. 9 Stationary Piston Sampler

A stationary piston sampler functions with the piston secured at a fixed elevation during sampling. The retracted piston sampler has the piston withdrawn to the top of the sampler immediately before taking the sample. The third type of piston sampler has a free moving piston.

The stationary piston sampler is used for sampling soft to stiff cohesive soils. The quality of samples obtained is better than that of thin wall (shelby) tubes. It provides good recovery and the head used on this sampler also acts more positively to retain the sample than the ball check valve of the thin wall tube samplers. Undisturbed samples taken with piston samplers are usually limited to a few meters.

BLOCK SAMPLING

The sample quality usually improves with increasing sample diameter. Samplers for undisturbed samples described in the preceding section, however, have practical dimensional limitations. Therefore large samples are often taken as block samples. Block samplers are also taken when no other samplers are available. In these cases the Gilboy Block sampling method (Hvorslev 1949) is used (Fig.10).

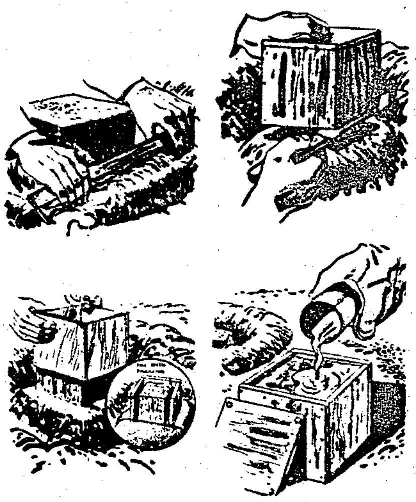


Fig. 10 The Gilboy Block Sampling Method

Carefully prepared hand trimmed block samples are taken from the walls of excavated pit. It produces very good quality undisturbed samples. Because of the depth limitation on the normal trial pit this method is usually restricted to about 5 m below ground level. Sometimes, the presence of high ground water table further limits this depth of sampling. Cubes with 300 mm sides are most common and they are usually waxed and boxed in situ before removing. For many soils with sensitive fabric or structure, this is the only satisfactory method.

For higher quality sampling, the box or cylinder with special cutting edges is pressed into the soil under controlled conditions as in ordinary push sampling.

Large samples, which are necessary for special laboratory tests, are taken as block samples. Large diameter cylindrical samples (914 mm diameter, 517 mm height) and large box samples (1250 mm x 1250 mm X 750 mm) are used by the Norwegian Geotechnical Institute (NGI). Due to the large dimensions, the cylindrical shape is preferred as it ensures a better rigidity of the casing. It is also easier to press the bottom plate into position. The square box required added stiffness by providing outside buttresses and ribs.

CORE BARRELS

Core samplers are used when drilling in very stiff soil or rocks. Core drilling differs from drive sampling in that, sampling and advancement of the borehole is done simultaneously. The cutting is usually done by diamond bits or tungsten carbide bits. The sample is contained in a stationary piston sampling tube inside the rotating core barrel. The ground-up material is removed by circular drilling fluid or by air.

Denison Core Barrel

The Denison sampler is used to recover an undisturbed sample where the thin wall sampler or the piston sampler cannot operate advantageously. i.e. in hardpan, hard clays, highly cemented soils or extremely difficult stiff deposits. The sampler relies on a combination of jacking and coring operations to obtain a sample. The basic components of the sampler (Fig.11) are an outer rotating core barrel

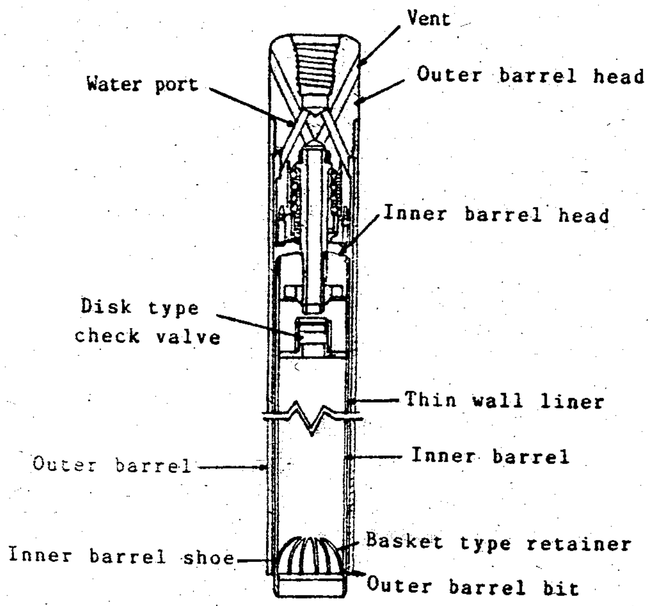


Fig. 11 Denison Double Tube Core Barrel

with a cutting bit; ~~an inner stationary sample barrel with a cutting bit~~; an inner stationary sample barrel with a cutting shoe; inner ~~rod~~ and outer barrel heads; an inner barrel liner; and an optional basket type core retainer. The bit may be either a carbide bit or a steel sawtooth bit, depending on the material to be sampled.

The sampler is available in sizes which cores having nominal diameters ranging from 2.4 in. (60 mm) to 6.3 in. (160 mm). Standard core lengths are 2 ft (0.6 m) and 5 ft (1.5 m).

Continuous Undisturbed Sampling

Continuous undisturbed samples over 50 ft (15 m) long or more have been collected with the Swedish foil sampler by pushing the sampler into the ground in a single sampling operation. This complex device uses thin strips or metal foil as a liner, preventing the soil from touching the tube. Generally, good quality undisturbed samples can be obtained by this technique. It is primarily used for sampling soft cohesive soils and thinly stratified soils.

The LGM Delft continuous sampler gives good quality samples at a lower cost. It is capable of taking 29 mm or 66 mm diameter samples into a nylon stockinette sleeve for a continuous length of 20 to 25 m. For routine investigations for shallow foundations, 6 to 8 m long samples are generally taken. The samples are usually split, described and are often photographed in a semi-dried state.

SAMPLE DISTURBANCE

The true meaning of "undisturbed" sample is that its physical structure and properties should be unchanged from the in situ conditions. The soil fabric or structure, water content and configuration of the individial strata must be carefully preserved. There must be no distortions or contamination of the sample. The deviation from the in situ conditions can be described at least qualitatively by the degree of disturbance.

The areas of disturbance to which a soil sample may be subjected have been reviewed by Hvorslev (1949) as:

i. Changes in stress conditions
ii. Changes in water content and void ratio
iii. Disturbance of soil structure
iv. Chemical changes
v. Mixing and segregation of soil constituents.

The extent of effect of the above on the sample depends on soil properties and techniques used for sampling. The quality of the undisturbed sample is measured by the above disturbance factors which may alter the engineering properties of the soil. The quality level required will vary depending upon the type and importance of the structure.

Changes in stress conditions evolve from the drilling and sampling operations, and the changes in stress state from an anisotropic to isotropic system. Removal of the sample from the ground to the surface reduces the total stresses on the boundaries to zero. When the sample attempts to rebound or dilate, less than atmospheric pore pressure develops in saturated soils.

Changing the void ratio of a saturated soil involves a change in moisture content. Changes in volume and moisture can occur prior to sampling, during the sampling operation, and after the sample has been removed. A 100% humidity room helps retard water loss.

Disturbance of the soil structure can take place when the sampler is driving/pushing into the ground, retrieval from the ground, handling and transporting to the laboratory, storing, extruding and trimming for testing. Even temperature changes could cause changes in soil structure.

Chemically induced changes also take place in soil samples. They may result from the samples' oxidation, contamination from the drilling fluid and reaction between the sampler and the soil. The problem of chemical changes may be intensified by the presence of acids or alkaline in the porewater and organic material. Testing samples soon after retrieval minimises internal migration of water and structural or chemical changes within the sample

Mixing and segregation of soil constituents in the top portion of the sample caused by improper cleaning of the borehole.

Most of the above sampling disturbances can be reduced to a reasonable extent by careful and appropriate sampling techniques. However, the disturbance cannot be fully avoided.

Nelson et al (1971) indicated that an undisturbed soil sample is virtually impossible to obtain because some disturbance will, always occur during sampling due to the change in the state of stress in the soil. Therefore, the term "undisturbed" is contradictory and thus in actual practice no sample is completely undisturbed. The "perfect sample" concept has been used to describe by Ladd et al (1964) to describe a sample which has received no disturbance by virtue of the processes of boring, sampling, trimming etc., but which has been subjected to relief of stress due to its removal from the ground.

SAMPLING PRACTICES IN SRI LANKA

In Sri Lanka, a wide variety of practices is used for routine soil sampling operations. Boreholes from 2 in. (51 mm) tc 6 in. (152 mm) in diameter are generally drilled with assorted boring equipment. Various types of augers are used to obtain disturbed soil samples. Thin wall tubes of 1.5, 2 and 3 in. in diameter (38, 51 and 76 mm) used in these boreholes for taking undisturbed samples. Of these samplers, 3 in. (76 mm) shelby tube is widely used. However, use of these shelby tubes is limited to soft to medium stiff cohesive soil deposits. In very stiff clays and residual soil deposits somewhat disturbed samples are obtained from thick wall tube samplers such as split spoon barrel.

3 in. (76 mm) diameter and 4 in. (102 mm) diameter Denison samplers are also used with the rotary wash boring technique. Samples obtained by this method are found to be good for hard clays, shale and sandstone.

With the precussion boring technique, thick wall open drive samplers are used to obtain 4 in. (102 mm) diameter and 18 in. (457 mm) long samples in gravelly soils. These sampler's lower end is armed with a hard steel cutting shoe.

Practice of obtaining undisturbed samples in sand is rare. Disturbed samples in sand are usually obtained by thick wall drive samplers with or without sample retainers. Bailers, with a flap valve at the bottom, are also used to obtain disturbed sand samples below ground water table.

In trial pits, 300 mm cube block samples are usually taken by carefully trimming the walls and placing it in a wooden box. This method is usually carried out in clays and gravelly soils when large undisturbed samples are required. Sometimes open drive samplers are hammered or jacked into the ground in trial pits to obtain undisturbed samples.

Hiller's sampler, which is perhaps best known as the peat sampler is occasionally used in the island to obtain reasonably disturbed samples in peat. This sampler, which has a longitudinal slot is filled with soil when rotated. It is easy to operate because of its low weight. The samples obtained can be used for visual inspection, peat quality and the depth of peat layer.

Undisturbed samples in peaty soil is usually collected by pressing a 300 mm wooden block or cylindrical mould (such as CBR mould) with cutting edges. The depth of sampling by this method, however, is limited to top few meters. A special type of 'piston sampler' is also seldom used to obtain good quality undisturbed peat samples at moderate depths.

In routine boring operations the usual practice of various organisations in the country, is to carry out Standard Penetration tests at selected intervals continuously, thus providing reasonably disturbed but representative soil samples at these depths. Whenever a soft cohesive deposit is encountered an undisturbed sample is taken. However, occasionally continuous sampling is practised.

CONCLUDING REMARKS

Large deposits of residual soils occur in various parts of Sri Lanka. Inhomogeneity and anisotrophy are common features of these deposits, which cause problems in sampling. Some of the difficulties associated with sampling in these deposits are: small samples can be unrepresentative and misleading; influence of relict joints, boulders, variability of soil matrix may not be reflected by laboratory strength tests; presence of gravel, cobbles and boulders in the soil matrix, could cause damages to cutting edges of shelby tubes and cutting shoes; borehole results are often not representative (Thurairajah et al 1985). In the view of Lee et al (1985), much higher frequency of sampling is generally required in the residual deposits than that are necessary for other soils. However, the author feels that this process of sampling is somewhat costly for shallow foundations and should be carried out only for special and important structures. For other structures, in situ tests may be preferred to obtain design parameters.

Undisturbed sampling in compacted fill (very often the fill may be laterites) is difficult. The bearing capacity for shallow foundations on such soils can be evaluated with reasonable accuracy by carrying out plate load tests. However, this is seldom practised in Sri Lanka, probably due to high cost of testing, tedious and cumbersome test procedures and lack of equipment and accessories. Nevertheless, the plate load test is carried out by some specialist organisations for large scale housing development schemes and for important structures on compacted fill. Checking the state of compaction of a particular fill can help the geotechnical engineer to assess the ground conditions in relation to bearing capacity and settlement aspects, though it does not provide direct values.

Undisturbed samples in sand can be obtained by Bishop's compressed air sampler. The availability of such sophisticated sample in the island is not known.

Reasonably disturbed sand samples can be obtained by split spoon barrel or any other thick wall tube samplers. Recovery of sand in these thick wall samplers below ground water table is generally low and this can be improved by using appropriate sample retainers.

The Standard Penetration test results (SPT 'N' values) in sand, are useful in evaluating bearing capacity for shallow foundations (Terzaghi et al 1948). The N values can also be used to predict settlement due to foundations loads (Schmertmann, 1970). Static cone penetration test (Dutch cone) produces data for bearing capacity and settlement for shallow foundations in cohesionless deposits. No sample is collected by this method as it measures just resistance to penetration of soil.

In soft to medium stiff cohesive soil deposits, both shelby tubes and piston samplers are used to obtain undisturbed samples. The stationary piston sampler procuces better quality sample than shelby tube. One advantage of the stationary piston sampler is that in soft soils, sample can be collected by pushing even without drilling a borehole. In very soft sensitive clays, undisturbed sampling is found to be difficult, thus, prediction of settlement by carrying out laboratory consolidation test is not possible. In situ vane shear test is usually carried out to obtain undrained shear strength values which may be required to estimate bearing capacity.

Undisturbed sampling in peaty soils is difficult mainly due to presence of undecomposed vegetation such as roots, stumps, etc. Lack of cohesion and high natural moisture content also cause sampling problems particularly in fiberous peat.

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REFERENCES

1. Terzaghi, Z.C. and Peck, R.B. (1948). Soil Mechanics in Engineering Practice. John Wiley & Sons Inc. pp. 423-426.

2. Hvorslev, M.J. (1949). Sub-surface Exploration and Sampling of Soils for Civli Engineering Purposes. US Waterways Experiment Station.

3. Ladd, C.C. and Lambe, T.W. (1964). The Strength of "Undisturbed" Clay Determined from Undrained Tests. ASTM Spec. Tech. Pub. No. 361 (Laboratory Shear Testing of Soils), pp. 342-371.

4. Schmertmann, J.H. (1970). Static Cone to Compute Static Settlement over Sand. ASCE Vol. 96, SM 3. pp 1011.

5. Nelson, J.D., Brand, E.W., Moh, Z.C. and Mason, I.D. (1971). The use of Residual Stress to Define Sample Quality. Proc. of Specialty Session: Quality in Soil Sampling, Fourth Asian Conference, ISSMFE. Bangkok, pp. 82-87.

6. Acker III. (1974). Basic Procedures for Sampling and Core Drilling. Acker Drill Co. Inc. Scranton, PA 18501 USA.

7. Lee, S.L., Lo, K.W. and Leung, C.F. (1985). Sampling and Testing of Residual Soils in Singapore. A Review of International Practice: Sampling and Testing of Residual Soils. pp. 153-157.

8. Thurairajah, A. and Wijekulasuriya, V.(1985). Sampling and Testing of Residual Soils in Sri Lanka. A Review of International Practice: Sampling and Testing of Residual Soils. pp. 169-173.

SOIL TESTING

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INTRODUCTION

One of the key factors governing the successful performance of a building is the proper foundation. The success of the foundation in turn depends on the selection of appropriate design parameters, design and analysis, and the construction techniques. The quality of the end product, i.e., the foundation, can only be as good as the poorest of all these components involved. In other words, the efforts involved in a sophisticated design and analysis are not justified if the design parameters are in error. For buildings on difficult or unknown subsoil, or for multi-storeyed buildings, thorough site investigation and testing programs are prerequisites.

Soil testing can be categorized into two major divisions; in situ or field testing, and laboratory testing. The advantages of in situ testing are:

(a) Ability to test the soil in its in-situ state.

(b) A larger volume of soil being represented.

(c) Ability to test soil deposits where good quality sampling is difficult. e.g., off shore deposits, sands.

(d) Ability to test the soil with minimal disturbance or stress relief.

The major limitation of in situ testing is that the boundary conditions and the state of stress around the in situ device are not known. Thus the interpretation is often purely empirical. On the other hand, laboratory tests usually have well defined boundary conditions and enable a rational interpretation of the test results with sound theoretical basis. The pros and cons of in situ testing and laboratory testing have been reviewed by Ladd, et al. (1977), Jamiolkowski, et al. (1985), and Robertson(1986).

Between in situ and laboratory testing, it is often wrongly argued that one is advantageous over the other. Such argument is merely based on individual prejudices and preferences. These are two different aspects of testing and they only supplement each other. Depending on the subsoil conditions and nature of the super structure, there has to be an adequate balance between the insitu and laboratory testing. One should never be blindly favoured over the other.

DESIGN PARAMETERS

A good foundation design requires a thorough knowledge of the sub surface soil profile and precise determination of the relevant soil properties.

It is important to assess the problem beforehand, based on field reconnaissance and trial borings. This is followed by appropriate laboratory and in situ testing, depending on the parameter required for the analysis and design.

(a) Sands
In sands, drainage is always immediate and the analysis is carried out in terms of effective stresses. The shear strength characteristics are represented by the angle of internal friction ϕ', which in turn is governed by the relative density D_r, Grain size, and grain shape. Modulus and Poissons ratio are also generally required in settlement computations.

(b) Clays
In clays, when the long term stability is critical, the analysis is performed in terms of effective stresses. Here, the failure envelope for normally consolidated clays pases through the origin. (i.e.,c' = 0). Shear strength and deformation characteristics are represented by the effective angle of internal friction ϕ', and the drained modules E_d. When short term stability is critical, the analysis is carried out in terms of total stresses. The failure envelope becomes horizontal ($\phi_u = 0$). The shear strength and deformation characteristics are represented by the undrained shear strength c_u and undrained modulus E_u respectively.

Time dependent deformation such as consolidation and creep contribute significantly to the total settlements in clays. The soil properties required to compute these settlements are : compression index (C_c), recompression index (C_r), preconsolidation pressure (σ_c'), coefficient of consolidation (c_v) and coefficient of secondary compression (C_α). When it involves building on compacted fields, compaction characteristics such as maximum dry density and optimum moisture content are also required. For foundation below water table, when dewatering is necessary, some knowledge about the permeability of the soil would be very helpful.

IN SITU TESTING

The major in situ tests, their perceived applicability to different soil conditions, and their use in obtaining various geotechnical information are listed in Table 1 (Robertson, 1986). Only a few of these, which are commonly used in this country will be discussed below in detail. A few lines are also added about some in situ tests which have great potential but rather new to Sri Lankan soils. (eg. pressure meter).

TABLE 1. IN SITU TESTS AND THEIR APPLICABILITIES
(After Robertson 1986)

Test method	Soil type	Profile	Piezometric pressure (u)	Angle of friction (φ)	Undrained shear strength (S_u)	Density (D_r)	Compressibility (m_v, C_c)	Rate of consolidation (c_h, c_v)	Permeability (k)	Modulus: shear and Young's (G, E)	In situ stress (K_0)	Stress history (OCR)	Stress–strain curve	Hard rock	Soft rock—till, etc.	Gravel	Sand	Silt	Clay	Peat—organics
	Geotechnical information													Ground conditions						
Dynamic cone (DCPT)	C	B	—	C	C	B	—	—	—	C	—	—	C	—	C	B	A	B	B	B
Static cone:																				
Mechanical	B	A	—	B	C	B	C	—	—	C	C	C	—	—	C	—	A	A	A	A
Electronic friction (CPT)	B	A	—	B	C	B	C	—	—	B	C	C	—	—	C	—	A	A	A	A
Electronic piezo	B	A	A	B	B	B	C	A	B	B	C	B	B	—	C	—	A	A	A	A
Electronic piezo/friction (CPTU)	A	A	A	B	B	B	C	A	B	B	C	B	B	—	C	—	A	A	A	A
Electronic seismic/piezo/friction (SCPTU)	A	A	A	B	B	B	C	A	B	A	B	B	B	—	C	—	A	A	A	A
Acoustic probe	B	B	—	C	C	C	C	—	—	C	—	C	—	—	C	—	A	A	A	A
Flat plate dilatometer (DMT)	B	A	C	B	B	C	B	—	—	B	B	B	B	—	C	—	A	A	A	A
Field vane shear (VST)	C	C	—	—	A	—	—	—	—	—	C	B	—	—	—	—	—	B	A	B
Standard penetration test (SPT)	A	B	—	B	C	B	—	—	—	B	—	C	—	—	C	B	A	B	C	C
Resistivity probe	B	B	—	B	C	A	C	—	—	C	—	—	—	—	C	—	A	A	A	A
Electronic conductivity probe	A	B	—	C	C	A	B	—	—	B	C	C	C	—	—	—	A	A	A	B
Total stress cell	—	—	—	—	—	—	—	—	—	—	B	B	—	—	—	—	—	C	A	A
K_0 stepped blade	—	—	—	—	—	—	—	—	—	—	B	B	—	—	—	—	B	A	A	B
Screw plate	C	C	—	C	B	B	B	C	C	A	C	B	B	—	—	—	A	A	A	A
Borehole permeability	C	—	A	—	—	—	—	B	A	—	—	—	—	A	A	A	A	A	A	B
Hydraulic fracture	—	—	A	—	—	—	—	C	C	—	B	B	—	B	B	C	C	B	A	C
Borehole shear	C	C	—	B	C	—	—	—	—	C	—	C	—	B	B	C	B	B	C	C
Prebored pressuremeter (PMT)	B	B	—	C	B	C	C	C	—	A	C	C	C	A	A	B	B	B	A	B
Push-in pressuremeter (PPMT)	A	B	B	C	B	C	C	A	B	A	C	C	C	—	—	—	B	A	A	B
Full-displacement pressuremeter (FDPMT)	C	B	B	C	B	C	C	A	B	A	C	C	C	—	—	—	A	A	A	A
Self-boring pressuremeter (SBPMT)	B	B	A	A	B	B	B	A	B	A	A	A	A	—	C	—	B	A	A	A
Self-boring devices:																				
K_0 meter	B	B	—	—	—	—	—	—	—	—	A	A	—	—	—	—	B	A	A	A
Lateral penetrometer	B	B	—	B	B	B	—	—	—	B	C	C	C	—	—	—	B	A	A	A
Shear vane	B	B	—	—	A	—	—	—	—	—	C	B	—	—	—	—	B	A	A	A
Plate test	B	B	—	C	B	B	B	C	C	A	B	A	C	—	—	—	B	A	A	B
Seismic cross/downhole/surface	C	C	—	—	—	—	—	—	—	A	—	—	—	A	A	A	A	A	A	A
Nuclear probes	—	—	—	B	—	A	—	—	—	—	C	—	C	—	—	—	A	A	B	A
Plate load tests	C	C	—	C	B	B	B	C	C	A	C	B	B	B	A	B	B	A	A	A

Note: A = high applicability, B = moderate applicability, C = limited applicability, — = not applicable.

STANDARD PENETRATION TEST (SPT)

The Standard Penetration test (SPT) is the most popular of all in situ tests. The test, developed in 1900's is essentially for cohesionless soils. Later, it has been extended for clays also with limited reliability. It was estimated that 80-90% of the routine foundation designs in the U.S.A. are accomplished using the SPT N-value (Robertson, 1986). The interpretation is purely empirical based on correlations. Resonably good estimates of ϕ', Dr, E_d can be obtained for sands. Terzaghi and Peek (1967) suggested some approximate correlations between N-values and undrained shear strengths of clays.

The desirable features of SPT which make it so attractive in the field are:

- relatively simple and rugged equipment.
- simple procedure with frequent measurements.
- gives samples within the split spoon tubes.
- can be carried out in most soil types.
- availability of well established correlations.

VANE SHEAR TEST

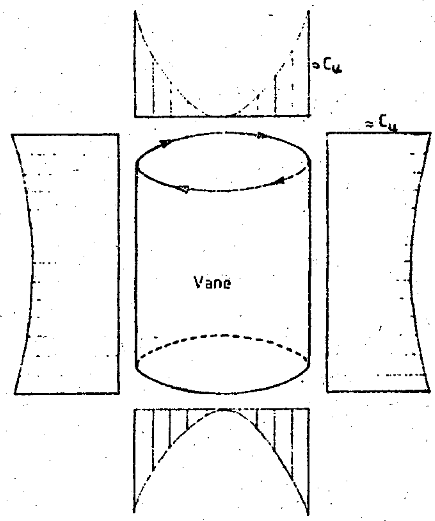
Vane Shear Test is ideally suited to measure the undrained shear strength of soft clays. It has moderate applicability in fabrous peat too. Usually the vane is driven to the desired depth is rotated at a rate of 0.1 degrees per second and the required torque is measured. As a result of this rotation a cylindrical failure surface is created, over which the shear strength is assumed to be uniform. Neglecting the drainage during the test, the strength mobilized is assumed to be the undrained shear strength. The undrained strength anisotropy is also usually neglected in the analysis.

The interpretation of vane shear test has better rationale than SPT. For a vane of diameter D and height H, the torque required is given by:

$$T = 2\int_0^{D/2} (2\pi r\, dr) r\, c_u + (\pi D H)\frac{D}{2} c_u$$

$$= \left[\frac{\pi D^3}{6} + \frac{\pi D^2 H}{2}\right] c_u$$

In reality, the stress distribution along the failure surface, especially over the top and bottom, is not uniform. Typical stress distribution suggested by Donald, et al. (1977) is shown in Fig. 1. Fortunately, the top and bottom surfaces contribute very little (less than 10%) to the total torque when H/D is greater than 2. Therefore, for practical purposes, it is reasonable to assume uniform shear stress along the entire failure surface.



Shear Stress Distribution for Vane Shear Test from Elastic Analysis (after Donald, et al, 1977)

FIG. 1

PLATE LOADING TEST

The idea of a plate loading test is to model the performance of the prototype foundation. Here, instead of measuring specific soil properties, the average deformation response of the subsoil is obtained. This test is very valuable in fills where the engineering characteristics are random and sampling is difficult. In developed countries, plate loading test is quite expensive due to the heavy loading arrangements required. In Sri Lanka, when labour is cheap, this still can be a viable alternative to the other in situ or laboratory tests.

Usually, 1.5 to 3.0 times the maximum anticipated contact pressure of the prototype foundation is applied on to the test plate in increments and the load-deformation characteristics are obtained. If economically practicable, it is preferable to load up to failure. The test results are their extrapolated to the prototype foundation.

The extent of subsoil over which the load-deformation characteristics are averaged increases with the increasing size of the plate. On the other hand, increasing the plate size means an increase in the required loads. For example, when the size of the plate is doubled, the load has to be quadrupled to achieve the same pressure. Therefore, the largest practicable size should be used. It is also a good practice to use two or three different sizes and extrapolate to the prototype.

DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic Cone Penetration test (DCPT) is very similar to SPT. A rod equipped with a conical tip of 10 sq. cm cross sectional area is driven into the soil by an arrangement similar to that in SPT. Number of blows per 300 mm is considered as a measure of resistance. SPT N-values and the number of blows from DCPT are expected to be same (Rodin, 1961).

Unlike SPT, DCPT is not standarized. For example, while going for the number of blows per 300 mm in North America, 200 mm of penetration is recommended in Europe. Due to the variability in the test procedure, equipment characteristics etc. there aren't sufficient well established correlations for DCPT. Melzer and Smoltczyk (1982) suggested some correlation for shear strength, compressibility and bearing capacity of shallow and deep foundations. In spite of it's crudeness, DCPT is quite popular for compaction control and in assessing the stratification, where sample are not required. Here, it may be possible to develop regional correlations between the blow counts and the relative compaction.

STATIC CONE PENETRATION TEST (SCPT)

Static Cone Penetration Test (SCPT), orginally developed in Europe, is now gaining wide acceptance in North America and other parts of the world. A cone with 60° apex angel and 10 sq.cm cross sectional area is pushed into the ground mechanically and measurements of end resistance (q_c) and the side friction are made. The interpretation methods had been reviewed by Robertson and Campanella (1983 a,b), Worth (1984), and Jamiolkowski, et al, (1985). The test works well in assessing the stratification of sub soil, compaction control and in studying the lequefaction potential.

The equipment is quite rugged and the testing procedure is simple. Provided verticality is maintained, depths greater than 100 m can be explored in soft soils. A major obstruction to the penetration is the presence of gravel layers, boulders and heavily cemented zones, which may damage the cone. Under such circumstances DCPT may be better choice. Other advantages of SCPT are the availability of continuous record, reproducibility of the test results and the more rational interpretation than SPT or DEPT. The cone resitance q_c (usually in Kg/cm^2) has been correlated with bearing capacity, settlements, SPT N-Values, undrained shear strength and deforamtion modulus.

PRESSUREMETER TEST

Pressuremeter test is the only in situ test which is performed under well defined boundary condition and thus allows a rational interpretation with strong theoretical basis. The device, invented by Menard in 1950s has undergone numerous modifications and backed up with sound interpretation techniques is emerging as the in situ device for any soil condition. Review of pressuremeter testing and its limitations were given by Battaglio, et al. (1981), Jamiolkowski, et al. (1985) and Sivakugan, et al (1986).

Two of the commonly used types are the conventional Menard type pressuremeter and the self boring pressuremeter (SPBM). Menard pressuremeter is quick in operation, inexpensive, and the interpretation is semi-empirical. SPBM is more complex, expensive and the interpretation is very sound. Pressuremeter test can provide measurements of modules (E or G), horizontal coefficient of consolidation (c_h) and undrained shear strength (c_u) in clays, angle of internal friction (ϕ') and angle of delation (ν) in sands; in addition, in all soils, the in situ horizontal stress (σ'_{ho}) and the stress strain curve can be obtained.

LABORATORY TESTING

Most of the commonly adopted laboratory tests are reviewed below. Emphasis is given to tests which have direct relavance to design of shallow foundations. The experimental procedures which may be obtained from the laboratory manuals and other literature are not repeated here.

GRAIN SIZE DISTRIBUTION

In granular soils, the grain size distribution has significant influence on the engineering behaviour. This is not the case with cohesive soils. Grain size distribution is determined by sieve analysis for particles greater than 0.074 mm (# 200 sieve size), and by hydrometer analysis for particles less than 0.074 mm. All the materials less than 0.002 mm are classified as clays.

ATTERBERG LIMITS

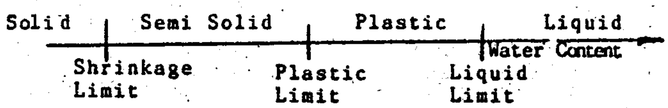


FIG.2.

The concept of Atterberg limits, originally proposed by a Swedish Scientist A. Atterberg for fine grained soils, were later modified and quantified by A. Casagrande (1932) who also suggested the test procedures to determine these limits. Depending on the mineral type and the water content, the fine grained soil can exist in four distinct states (Fig. 2). The boundaries between the adjoining states in terms of water content are called shrinkage limit, plastic limit and liquid limit. Of these, plastic limit and liquid limit are the most relavent to geotechnical engineering applications.

Liquid limit may be determined by Casagrande's percussion cup or Swedish fall cone method. The pros and cons of these two different methods have been discussed by several researchers. For all soils, it has been observed that the shear strength at the liquid limit is about 10-25 g/sq. cm.

Based on some observations by Skempton and Northey (1953), it is assumed in critical state theories that the shear strength at the plastic limit is 100 times that at the liquid limit.

Liquid limit and plastic limit are very useful indices in the classification of fine grained soils and in identifying the type of clay mineral. These simple tests also provide additional information on shear strength and compressibility characteristics of the soils. These would be very helpful in the preliminary studies, and would supplement the other laboratory test results on shear strength and compressibility. The shear strength and compressibility characteristics may be estimated from empirical correlations. Some widely accepted correlations are :

$C_c = 0.009$ (LL - 10%) for undisturbed soil. (Terzaghi & Peck, 1948).

$c_v = 0.11 + 0.0037$ PI for normally consolidated clays (Skempton, 1957)

CONSOLIDATION TESTING

Consolidation tests are generally performed to determine the compressibility and consolidation characteristics of cohesive soils. It is the most relevant test in determining the right parameters for settlement analysis in cohesive soils. These parameters and their uses in fouondation engineering are listed below:

(i) Prediction of total consolidation settlements
- Compressive Index (C_c)
- Recompression index (C_r)
- Preconsolidation pressure (σ_c')
- Variation of coefficient of volume compressibility (m_v) with stress level.

(ii) Prediction of the rate of consolidation
- Coefficient of consolidation (c_v)

(iii) Analysis of the effectiveness of the Preloading technique
- Coefficient of consolidation (c_v)

(iv) Prediction of the secondary compression
- Coefficient of secondary compression (C_α)

Consolidation test is usually carried out on circular specimens of 2 - 3 cm in height and 6 - 10 cm in diameter, held in a circular ring called Oedometer. Friction along the walls can be minimized by the application of a thin coat of silicone oil or similar lubricant. Larger the diameter to height ratio lesser is the effect of friction. On the other hand, larger diameter would require higher loads. For organic soils, especially in peats, it is desirable to test specimens of larger diameter to height ratio.

In the conventional incremental type of loading test, the loads are applied in increments, each one lasting a specific duration (typically a day). For each increment the deflection - time variation is obtained and c_v , m_v and C_α are computed. At the end of the consolidation test, the void ratio is plotted against the logarithm of effective stress and C_c , C_r, σ_c' are computed.

Oedometer test results are generally interpreted using Terzoghis one dimensional consolidation theory, assuming that the soil is saturated. To ensure saturation throughout the test it is preferable to apply a back pressure of 50 -300 kPa , using a special arrangement. In organic soils the primary consolidation occurs very fast, and the validity of Terzaghi's theory is questionable. Nevertheless, in the absence of any other sound and better theories and due to its simplicity, Terzaghi's theory is still being used by many.

TRIAXIAL TEST

Triaxial test is the most popular of all laboratory tests to determine the shear strength characteristics of all soils. In addition, complete stress-strain curve, and the modulus of deformation can be obtained. In spite of its usefulness, the test requires considerable skill and a thorough knowledge of the apparatus details, control panels etc.

Triaxial tests are usually carried out on a undisturbed samples of cohesive soils or reconstituted samples of cohesionless soils. For cohesive soils, the test can be any of the following three types:

(i) Unconsolidated Undrained (UU) test also called Q-test (Q for quick)

(ii) Consolidated Undrained (CU) test also called R-test

(iii) Consolidated Drained (CD) test also called S-test (S for slow)

In addition, the test can be either strain controlled or stress controlled type. For strain softening soils, peak behaviour can be studied only in strain controlled tests. This is important in problems involving large strains such as landslides or earth quakes.

The sampling in the field involves stress relief and mechanical disturbance. To obscure these effects, it is a practice to reconsolidate the sample to its estimated in situ effective vertical overburden stress (σ_{vo}') before shearing. Due to simplicity of operation, consolidation is often carried out isotropically. Nevertheless, the in situ state of stress is anisotropic and thus the tendency is to consolidate the sample anisotropically for a better simulation of in situ conditions. To achieve this K_o - Triaxial cells can be used (Campanella and Vaid, 1972). The undrained shear strength (c_u) is governed primarily by σ_{vo}' and not influenced significantly by the mode of consolidation. For isotropic and anisotropic consolidation, the difference in shear strength is generally less than 20% for the same σ_{vo}'. Sivakugan, et al. (1987) proposed a simple method to predict CK_0UC shear strength from CIUC test. Therefore, the conventional triaxial cells are sufficient for practical purposes.

Irrespective of the mode of consolidation, it is possible to have different modes of shearing. Upon consolidation the specimen may be loaded under axial compression, lateral compression, axial extension or lateral extension.

Depending on the field loading conditions, the appropriate triaxial test may be adopted. For example, to study the excavation problem, an isotropically consolidated undrained extension (CIUE) test may be recommended.

UNCONFINED COMPRESSION TEST

Unconfined compression test is a special case of the triaxial test. Since there is no lateral confinement it is performed only on cohesive soils. The test is very simple and quick. It gives the undrained shear strength, modulus and the stress-strain curve. Due to pronounced disturbance, the test results are not as reliable as the triaxial tests.

COMPACTION TEST

In shallow foundations, the influence zone is usually confined to shallow depths and compaction is the best means of improving the ground conditions. Generally, the compaction is carried out by hand held rammers or mechanical rollers on existing loose ground, and in placement of fills. As far as the shallow foundations are concerned, compaction achieves two major objectives:

i. Reduction in the compressibility, thus minimizing settlements.

ii. Increase in shear strength.

Both these factors lead to an increase in the bearing capacity.

The adequacy of compaction is checked by obtaining the water content and dry density of the in situ soil and comparing them with those of the laboratory compacted specimens. In the field, the density is obtained by sand cone method or balloon density method. In the laboratory, Standard Proctor Compaction test is carried out to determine the optimum water content and the maximum dry density.

Three of the commonly used specifications for compaction tests are :

(i) Standard Proctor Test with 12400 ft-lb/cu. ft of compactive energy.

(ii) Modified Proctor Test with 56300 ft-lb/cu. of compactive energy.

(iii) Low Energy Proctor Test with 7500 ft-lb/cu. of compactive energy.

Of all these, the Standard Proctor test is the most common. In all Proctor Compaction tests, the soil is thoroughly mixed with water and compacted into a standard mold to given specifications. The test is repeated at different water contents and the dry density is plotted against the water content. The maximum dry density and the optimum water content are obtained from the plot.

COMPUTER APPLICATIONS

The advent of microcomputers has had a tremendous impact on laboratory testing of soils. Using microcomputers and other electronic components, it is relatively easy to develop specialized interactive data acquistion and control systems for automation of laboratory tests. Sivakugan, et al. (1987) developed a cuboidal shear(true triaxial) device with automatic dara acquistion and control system. Similar systems could be developed for triaxial testing, oedometer testing etc. In developed countries, the rising costs of wages and the declining cost of microcomputers and the other electronic components have resulted in rapid advances in such systems. In Sri Lanka, where time factor is not given due consideration and labour costs are low, we are far behind in exploiting the full potential of microcomputer applications in laboratory testing.

Data Acquisition:

For an automatic data acquisition system operated by a microcomputer, the pressures and displacements can be measured by electric transducers and Linear Variable Differential Transformers (LVDT) respectively. The LVDTs and pressure transducers continuously emit voltage signals propertional to the deformation or pressure. In order to be handled by the microcomputer, these voltage signals must be converted into digital signals. This task is performed by a unit known as analog-to-digital (A/D) convertor. To enable a single A/D convertor to handle several channels of measurements, multiplexers are used. Depending on the nature of measurement, some form of signal conditioning (e.g., amplification, filtering) also may be required. Finally the data is stored in magnetic disks.

Control:

In controlling, the criterion is to perform the test while satisfying certain pre-defined conditions. Data received from the A/D converter are processed by the microprocessor where the decision are made regarding the control. Other electronic devices which help to accomplish the control are solenoid values, stepper motors, and digital-to-analog (D/A) convertors.

To simplify the operation further, the A/D and D/A converters with built-in signal conditioners are available on printed circuit boards (PCB) which can be readily installed into one of the expansions slots of the microcomputer.

SUMMARY

The role of soil testing in shallow foundation design, various soil parameters involved, and the appropriate method of determining them were examined. Emphasis was given to in situ and laboratory tests which have direct relevance to shallow foundation designs. Finally, a bried mention was made of microcomputer applications in laboratory testing.

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REFERENCE

1. Battaglio, M, Ghionna, V, Jamiolkowski, M, and Lancellotta, R. (1981). "Interpretation of self-Boring Pressuremeter Tests in Clays", Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm, Vol. 2, pp. 433-438.

2. Donald, I.B., Jordan, J.O., Parker, R.J., and Toh, C.T. (1977). "The Vane Test - A Critical Appraisal", Proceedings of the 9th International Conference on Soil Mechanics and foundation Engineering, Tokyo, Vol. 1, pp. 81-88.

3. Jamiolkowski, M., Ladd, C.C., Germanie, G.T., and Lancellotta, R., (1985). "New Developments in Field and Laboratory Testing of Soils", Theme Lecture, Proceedings of 11th International Conference on Soil Mechanics and fouondation Engineering, San Francisco, Vol. 1, pp. 57 - 153.

4. Ladd, C.C., Foott, R., Ishihara, K., Schlosser, F., and Poulos, H.G. (1977). "Stress Deformation and Strength Characteristics State of the Art Report", Proceedings of the 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Vol. 2, pp. 421 - 494.

5. Melzer, K.J. and Smoltczyk, U. (1982). "Dynamic Penetration Testing, State-of-the-Art Report, Proceedings of the Second European Symposium on Penetration Testing, Amsterdam, Vol. 1, pp. 191 - 202.

6. Robertson, P.K. (1986) "In situ Testing and Its Application to Foundation Engineering", Canadian Geotechnical Journal, Vol. 23, No.4, pp. 573 - 594.

7. Robertson. P.K. and Campanella, R.G. (1983a). "Interpretation of Cone Penetration Tests. Part I : Sand ", Canadian Geotechnical Journal, Vol. 20, No.4, pp.718-733.

8. Robertson, P.K. and Campanella, R.G. (1983b). "International of Cone Penetration Tests.Part II : Clay", Canadain Geotechnical Journal, Vol.20, No.4, pp. 734 - 745.

9. Skempton, A.W. (1957). Discussion on the planning and Design of the New Hong Kong Airport, Proceedings of the Institution of Civil Engineers, Vol. 7.

10. Skempton, A.W. and Northey, R.D. (1953). "The Sensitivity of Clays", Geotechnique, Vol. 3, No. 1, pp. 30 - 53.

11. Sivakugan, N., Chameau, J.L., Holtz, R.D., and Altschaeffl, A.G. (1987). "Servo - Controlled Cubodal Shear Device", Geotechnical Testing Journal, ASTM, Accepted for publication.

12. Sivakugan, N., Holtz, R.D., and Chameau, J.L. (1987). CK UC Shear Strength of Normally Consolidated Clays from CIUC Test", Journal of the Geotechnical Engineering Division, ASCE, Accepted for publication.

13. Sivakugan, N., Prapaharan, S., Altschaeffl, A.G., Holtz, R.D., and Chameau, J.L. (1986) "Review of Pressuremeter Testing in Clays", Proceedings of the Asian Regional Symposium on Geotechnical Problems and Practices in Foundation Engineering, Sri Lanka, pp.138-145.

14. Terzaghi, K. and Peck, R.B. (1948) Soil Mechanics in Engineering Practice, John Wiley & Sons, New York, 422 pp.

15. Wroth, C.P. (1984). "The Interpretation of In Situ Soil Tests", 24th Rankine Lecture, Geotechnique Vol. 34, No. 4, pp. 449 - 489.

DESIGN OF SHALLOW FOUNDATIONS - THEORETICAL APPROACH

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1. INTRODUCTION

The function of a foundation whether deep or shallow, is to transmit the loads and moments from the super-structure safely into the ground. Deep foundations are normally used in cases of weak shallow soil layers and/or heavy loads. A properly designed foundation should be able to withstand extreme loads without duress and should not excessively settle under the working load. A bearing capacity calculation and a settlement analysis should be carried out to check the above requirements. This paper is confined to the design of isolated shallow foundations.

2. BEARING CAPACITY

2.1 Simple Cases

A bearing capacity formula for a vertically loaded strip footing on a horizontal ground was first proposed by Terzaghi(1). A modified form of this formula is;

$$Q/B = \tfrac{1}{2}\,\gamma B\,N_\gamma + q\,(N_q-1) + c\,N_c$$

where Q is the net bearing capacity, B, foundation width, γ, unit weight of the soil, c, cohesion and q, effective soil pressure at the foundation level. This seperation of contributions from cohesion, surcharge and, self weight, though not exactly correct in all cases, is used in almost all bearing capacity formulas. In sands and silts effective stress parameters are used; in clays under undrained conditions total stress parameters for strength and unit weight are used.

Prandtl(2) using the theory of plasticity derived the exact solution for N_q and N_c as;

$$N_q = e^{\tan\phi'}\tan^2(45° + \phi'/2)$$

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

for effective stress analysis and,

$$N_c = \pi + 2\ ,\ N_q = 1$$

for undrained clay. $\phi'$is the effective angle of internal friction of the soil. $N_\gamma$is zero for undrained clay but depends on the friction at the foundation base in the other cases. A value of $N_\gamma = 1.8\,(N_q-1)\tan\phi'$ is recommended by Hansen (3)-(4).

2.2 General Cases

An actual foundation may deviate in several aspects from the simple cases considered above. The load may be eccentric, and/or inclined. The foundation is placed at a depth D below the ground surface and has a finite length L; its shape may not be even rectangular. Also the foundation base and the ground surface may be inclined.

There are several methods available for analysing these cases but easily computable Hansen's method (Hansen (3) and (4)) is discussed here. In Hansen's method the eccentricity of loading is eliminated by using an effective area for the foundation. This is obtained by replacing the actual foundation by a rectangle so that its geometric centre coincides with the load centre and it follows the nearest contour of the actual foundation as close as possible. The dimensions of the effecive area (new B and L) are used for the rest of the analysis.

The effect of shape (s), depth (d), load inclination (i), base inclination (b), and ground inclination (g), are incorporated into the calculations by modifying the original bearing capacity factors N_c, N_q and N_γ using s,d,i,b and, g factors. Hansen proposed that N_q and N_γ in the original Terzaghi formula for c', ϕ' soils to be modified as;

$$N_q' = N_q.s_q.d_q.i_q.b_q.g_q \text{ and,}$$

$$N_\gamma' = N_\gamma.s_\gamma.d_\gamma.i_\gamma.b_\gamma.g_\gamma.$$

New N_c, i.e. N_c', is taken as $(N_q'-1)\cot\phi'$.

In the case of cohesive soils only N_c needs to be modified. This is done using a set of additive factors;

$$N_c' = N_c\,(1 + s_c' + d_c' - i_c' - b_c' - g_c').$$

These factors are expressed in terms of empirical formulas which are consistent with existing theoretical solutions and experimental observations. For e.g. inclination factors for undrained clay are derived from the theory of plasticity; shape factors for vertical loading in clay agree with those proposed by Skempton(5) using loading tests. Hansen has published several papers which give different sets of these formulas. One such set of these factors is given below.

Refer the Fig.1 given below for the configuration and forces acting on the foundation.

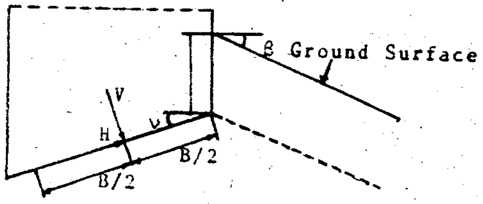


Fig.1 Configuration of General Case

Shape Factors

$s_c' = 0.2(1 - i_c')\ B/L$

$s_q = 1 + \sin\phi' B i_q / L$

$s_\gamma = 1 - 0.4\ i_\gamma\ B/L$ or 0.6 whichever is higher.

Depth Factors

$d_c' = 0.4 \tan^{-1} D/B$

$d_\gamma = 1$

$d_q = 1 + 2 \tan\phi'(1-\sin\phi')^2 \tan^{-1} D/B$

Inclination Factors

$i_c' = 0.5 - 0.5\sqrt{(1-H/Ac)}$

$i_q = (1 - 0.5H/(V + Ac \cot\phi'))^5$

$i_\gamma = (1 - (0.7 - \nu°/450°)H/(V + Ac \cot\phi'))^5$

Base Slope Factors

$b_c' = \nu°/147°$

$b_q = e^{-2\nu \tan\phi'}$; $b\gamma = e^{-2.7\ \nu \tan\phi'}$

Ground Slope Factors

$g_c' = \beta°/147°$

$g_q = g_\gamma = (1-0.5 \tan\beta)^5$

where A_f, L and, B are the area, length and, width of the effective foundation area respectively. H is the horizontal force parallel to B direction. When load inclination in L direction is significant, a check should be made for failure along L direction. This can be done by interchanging L and B in the above expressions and using H as the shear force paralell to L direction. H should be less than $V \tan\phi' + cA_f$ to prevent sliding at the base. Also i_q and i_γ should be greater than zero and, β + ν should be less than 90°.

2.3 Effect of Water Table

The position of the water table does not affect the total stress analysis of undrained clay. Anyway the water table is likely to be high in this case. However, in the effective stress analysis the bearing capacity can reduce even by 50% depending upon the position of the water table. In calculation of bearing capacity in such cases it is easier to consider three simple cases at first. In case A, the water table lies at a depth more than B below the foundation level. The bearing capacity in this case is not influenced by the water table. In the case B, the water table is at the foundation level. In this case use the submerged unit weight in calculating the N_γ term and saturated or dry unit weight in calculating the N_q term. In case C the water table is at the ground surface, hence the submerged unit weight is used in all calculations. In cases in between A and B and, B and C, use linear interpolation to determine the bearing capacity. The possibility of the water table rising during or after construction should be considered in the analysis.

2.4 Soil Testing and Factor of Safety

Soil parameters required for the design are the undrained shear strength for total stress analysis of undrained clay or the strength parameters c and ϕ' for the effective stress analysis. The undrained shear strength can be determined by field vane shear test or laboratory unconfined compression test or by unconsolidated undrained triaxial test. For the effective stress analysis unconsolidated undrained or consolidated undrained triaxial test is the most suitable. However, direct shear test can be used for non-cohesive soils.

A sample prior to testing should represent the worst condition in the field during and immediatly after constructon as close as possible. It is not necessary to consider long term effects unless the foundation is going to be heavily loaded only after a long time. It may be necessary to soak the samples to obtain the correct moisture content. The all round pressures used in triaxial tests should be similar to those in the field at the time of construction. In a special case it is better to follow the stress path of the field sample as close as possible in the laboratory test.

A factor of safety (rather a load factor) of 2-3 is usually applied on the bearing capacity to obtain the safe bearing pressure. Higher factor of safety should be used on sites with unreliable and varying soil formations and soil conditions. Hansen's factors given here are derived for plane strain problems. As the angle of internal friction is slightly higher (by about 10%) in plane strain than in triaxial case increase the triaxial values by 10% for use in the effective stress analysis.

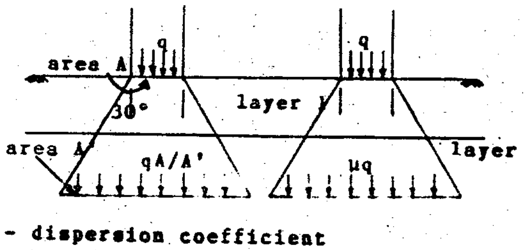


Fig.2 Bearing capacity of layered soil

2.5 Layered Soils.

Foundations in layered soils can fail in bearing at any layer. A simple way of design in such cases is the use of a load dispersion line (see Fig.2). Instead of the surface pressure fully acting on a deeper layer it is assumed that the foundation load is distributed over a larger area than the foundation area. Then the pressure on each layer can be calculated and checked against the bearing capacity of that layer. When a soft material underlies a stiff soil Tomlinson(6) gives the inclination of the dispersion line to vertical as 30° (Fig.2). Alternatively, the linear elastic stress distribution for the particular case can be used to find the value of the dispersion coefficient.

3. ANALYSIS OF SETTLEMENTS

3.1 Introduction

A properly designed foundation should not settle excessively under the working load. The differential setttlement between one part of a structure and another is of greater significance to the stability of the super--structure than the total settlement. Differential settlements are caused by variations in soil strata or soil conditions (like stress history or position of the water table). It can also be due to difference in time of construction of adjacent parts of a structure. They should be minimised wherever possible. This can be done by carefully choosing a construction sequence, and/or by redistributing the loads sometimes increasing the loads in lightly loaded areas. In some cases foundations are taken to deeper layers and basements introduced to reduce differential settlement. Jacking pockets can be provided under beams to reduce the effect of differential settlements on the super-structure. In some cases Plinth beams are used to tie the columns together to reduce differential settlements; alternatively a raft foundation can be used if it is more economical.

3.2 Settlements in clay

3.2.1 Introduction

The settlement of a foundation in a cohesive soil consists of three components, namely immediate, consolidation and, secondary or creep settlements. Immediate settlement is due to distortion of the soil at undrained state and occurs at constant volume in incompressible soils. Consolidation settlement is associated with the dissipation of excess pore water pressure within the soil. Secondary settlement is due to creep or viscous flow which would not affect the pore pressure or effective stress. Unless a soil exhibits substantial creep behaviour (e.g. peat) secondary settlement is not separately calculated but implicitly incorporated into the consolidation settlement.

3.2.2 Immediate Settlement

Immediate settlement which occur at undrained state is normally estimated using the theory of elasticity.

The possibility of local yielding leading to underestimation by elastic method was investigated by Davis & Poulos (7). They showed for a strip footing that, unless the soil is heavily overconsolidated local yielding is likely to be insignificant under normal working load. The immediate settlement depends upon the undrained Young's modulus and Poisson's ratio, width and shape of the foundation, foundation depth, rigidity of the foundation and, on the extent of the compressible soil layer. Poisson's ratio is assumed to be 0.5 for an incompressible soil. For a homogeneous clay deposit with a finite thickness, it is convenient to use the charts given by Janbu et al (8) and later improved by Christian & Carrier (9). The average immediate settlement of the foundation (ρ_i) is given by;

$$\rho_i = \mu_1 \mu_0 \, q_n B/E_u$$

where q_n is the bearing pressure, E_u, undrained Young's modulus and μ_1, μ_0 factors taken from the charts (see Fig.3).

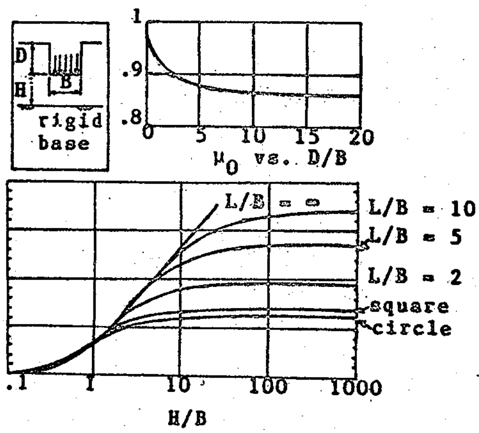
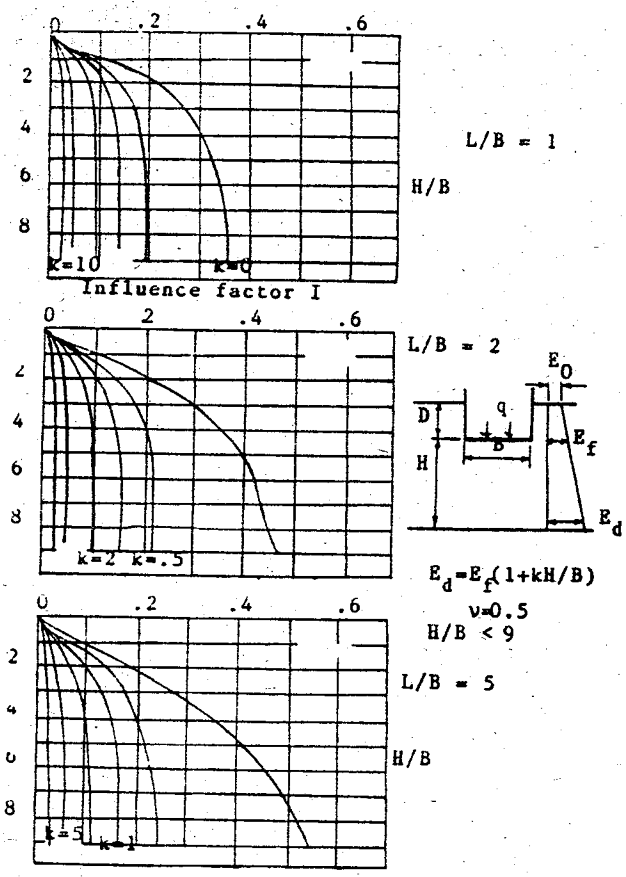


Fig.3 Charts for μ_0 and μ_1

These charts take into account the effect of shape (circular or rectangular), depth, rigidity of the foundation and, the thickness of the compressible layer. In cases of Young's modulus increasing with depth it is convenient to use the charts given by Butler (10) for a modulus increasing linearly with depth (see Fig.4).

For a soil with a number of compressible layers, the method given by Simons & Menzies (11) is particularly useful. This technique is illustrated in Fig.5. Consider a rigid base on top of the nth layer and calculate the settlement using a constant E_u, $(E_u)_n$ equal to that of nth layer. Do the same again but assuming a rigid base at the bottom of the nth layer. Subtract the second result from the first to get the contribution from the nth layer for the total immediate settlement $(\rho_i)_n$. Follow the same procedure for the other layers and add up to get the total settlement.



Settlement at corner of foundation = qBI/E_f

Fig.4 Butler's Charts

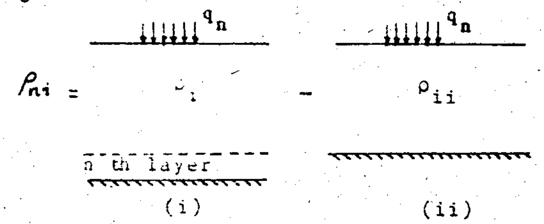


Fig.5 Simons & Menzies Method

3.2.3 Consolidation Settlement

Consolidation settlement is usually calculated using Terzaghi's one dimensional theory of consolidation. The procedure is as follows; Determine the vertical stress distribution beneath the centre of the foundation using a linear elastic solution. There are many methods available for this; for e.g, Newmark (12) tabluted the vertical stresses beneath the corner of a rectangular area(these are reproduced in Tomlinson(6)). It is usually sufficient to calculate the vertical stress distribution down to a depth of 1.5B. However if a very weak soil layer exists at a depth then it has to be more. One or several consolidation tests should be carried out to determine the effective pressure - voids ratio relationship and the preconsolidation pressure of the soil. By knowing the increase in vertical stress and its insitu value it is possible to calculate initial and final voids ratios (e_0 and e). Then the settlement is found by the integral of $(e_0-e)/(1 + e_0)$ down the depth. This can be done in a numerical manner by dividing the soil into a number of layers.

There is also a popular but less accurate method of using the coefficient of compressibility m_v. However, when using this method care should be taken in lightly overconsolidated clays as m_v can suddenly increase if the soil get normally consolidated due to foundation loading. The change of lateral stress during consolidation is different in a consolidometer from a field situation. To overcome this, Skempton & Bjerrum (13) has suggested a correction factor μ_g by which the estimated settlements should be multiplied. The value for μ_g is recommended to be,

1-1.2 for very sensitive clays,

0.7-1.0 for normally consoildated clays,

0.5-0.7 for over-consolidated clays and,

0.2-0.5 for heavily-overconsolidated clays.

If the foundation is located at a depth the calculated settlement should be corrected for depth. Even though derived for elastic analysis only, Fox's curves (Fox(14), see also Fig.6) are used to apply this correction conveniently.

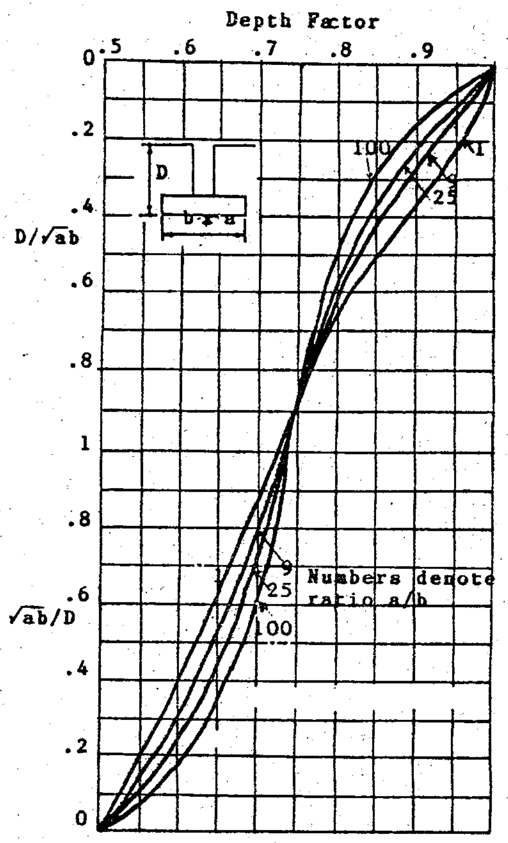


Fig.6 Fox's Depth correction curves

3.2.4 Secondary Settlement

Secondary settlements had been observed both in the field and in laboratory in many soils though it is more pronounced in organic soils. The rate of secondary settlement can be expressed as a straight line in a e - $\log_{10}$ t plot. The rate of secondary compression given by ε_α is equal to $C_\alpha/(1 + e_0)$ where C_α is the gradient of the straight line mentioned above. The amount of secondary compression in a foundation can be estimated if the variation of ε_α with e_0 is determined from a consolidation test.
The value of ε_α for normally consolidated compressible soils is known to increase in general with natural moisture content. Mesri(15) had proposed a linear relationship in a log - log plot in this regard (see Fig.7). Even though the range of values is large, the relationship gives an idea of the upper limit of secondary settlement. If the soil is overconsolidated, secondary settlement is known to be low unless it becomes normally consolidated due to the foundation loading. Mesri had also classified the soil according to the value of ε_α; a value below 0.004 is low while a value above 0.008 is high.

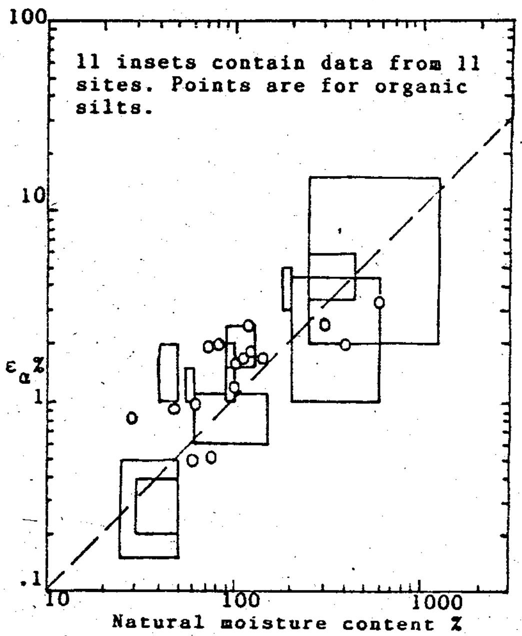


Fig. 7 Secondary consolidation of clays

3.2.5 Special Techniques

Shallow foundations are not important enough in most of the cases to apply sophisticated analysis like finite elements. However, Lambe's stress path method (Lambe (16)) can be used to calculate the settlements in a more realistic manner. In this method the soil is divided into a number of layers and a sample taken from each layer. The insitu vertical and lateral stresses on each sample are restored in a triaxial apparatus.

Then the vertical and lateral stress changes due to foundation loading, estimated from the theory of elasticity, are applied to each specimen and immediate and consolidation vertical strain determined. The immediate and, consolidation settlements of the foundaion are now calculated by integrating the measured strains over the soil depth. This method has the advantage that it follows the stress path of the field situation closely. However, a correction should be applied for the depth effect of the foundation.

3.2.6 Rate of Settlement

The rate of consolidation settlement is calculated using Terzaghi's one dimensional theory of consolidation. The settlement at time t, ρ_t is expressed as;
$\rho_t = \rho_i + U\rho_c$ where U is the degree of consolidation. The relationship between U and time factor (T_v) given by $C_v t/H^2$ where C_v is the coefficient of consolidation, t time and H length of the drainage path, is shown in Fig.8 for various cases of initial excess pore water pressure distributions.

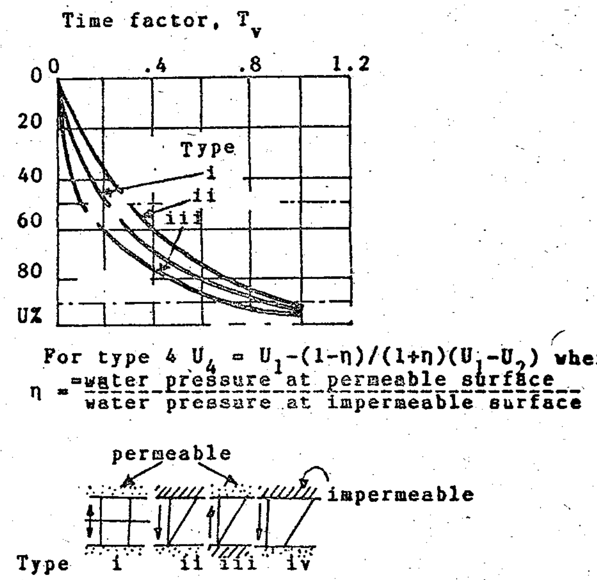


Fig. 8 Relation between U and T_v

Concrete is normally much more permeable than most of the clays hence concrete foundations are considered as permeable boundaries in many cases. It had been found that the field settlements occur at a much faster rate than laboratory based predictions. This has led to speculation that changes in the macro-fabric which is not represented in small samples is the cause for this. Once the consolidation settlement is known, the settlement of each foundation with time, hence the differential settlements can be calculated by considering the construction schedule. It is also possible to alter the construction schedule at this stage to minimise the differential settlements.

3.2.7 Soil testing

The soil parameters required for settlement analysis are undrained Young's modulus and coefficients of compressibility and consolidation of the soil. The undrained Young's Modulus is normally determined by an unconsolidated undrained triaxial test. When testing it is advisable to apply a cell pressure equivalent to the mean normal total stress in the field. Sample disturbance is known to considerably affect the value of undrained Young's modulus. If the field conditions are known it may be better to reconsolidate the field samples to the insitu values prior to testing. In any case the stress range of testing should be kept as close as possible to that in the field to avoid large errors.
When immediate settlements are separately calculated, immediate settlements observed during a laboratory consolidation test should not be added when determining the coefficient of compressibility or voids ratio-pressure relationship. Otherwise the calculation will unnecessarily duplicate the immediate settlements. This also applies to secondary settlements if they are separately calculated. In the cases where secondary settlements are not separately calculated, taking 24 hours settlement in determining the coefficient of compressibility will account for some creep effects. When calculating the coefficient of compressibility and voids ratio changes, the pressure-voids ratio relationship for the field situation reconstructed from the behaviour of laboratory sample should be used (for e.g. laboratary sample may be overconsolidated but the field sample is normally consolidated).

3.3 Settlements in Sand

3.3.1 Introduction

With granular soils laboratory based methods of evaluation are remote due to the basic difficulty of obtaining representative and undisturbed samples from the ground, especially below the water table. The most popularly used mathods of calculation are based on insitu penetration tests, namely Standard Penetration Test (SPT) and Static Cone Penetration Test (SCPT). Though expensive, Plate Bearing Test is also used in some cases, particularly in gravelly and boulderly soils where any other method of sampling is extremely difficult. Self boring pressuremeter is also becoming increasingly popular in obtaining compressibility and strength parameters for the analysis of foundations.

3.3.2 Standard Penetration Test

The standard penetration test is conducted by driving a 50 mm diameter split spoon sampler using a 63.5 kg. weight falling freely through 760 mm. The SPT value N is taken as the number of blows required to drive the sampler through the last 300 mm of a 450 mm total penetration. In saturated, fine or silty, dense or very dense sands, N values measured can be abnormally high due to dilatancy of the soil under undrained conditions.

Terzaghi & Peck (17) recommended modifying the measured N values by,
$N_c = 15 + 0.5(N_m - 15)$ for $N_m > 15$,
where, N_m is the measured value and N_c, corrected value. If N is less than 5 it is better not to use isolated footings. As N values depends on the stress level and relative density of soil some correlation between stiffness of the soil and N values can be expected.
The most popular method of design using SPT is the method proposed by Terzaghi & Peck (17) which was later modified by Peck et al (18). The method is based on a correlation between plate bearing test results and SPT values. An allowable differential settlement of 25 mm is assumed for isolated footings. The following design procedure had been suggested by Peck et al.
a) Depending on the subsoil condition apply the dilatency correction.
b) Correct N values for the overburden pressure. The given design charts were prepared for an overburden pressure of 1 tonf/ft.^{2}, hence, measured N values should be corrected as shown in Fig.9.

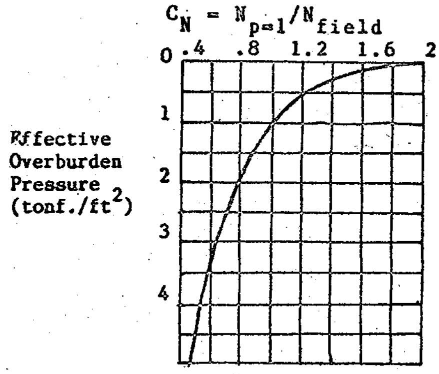
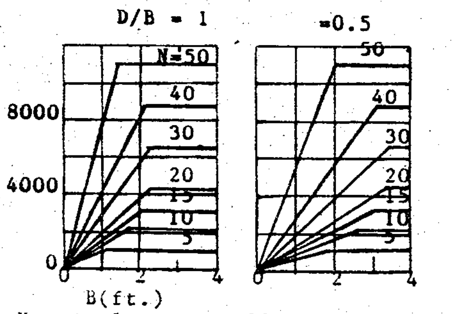


Fig. 9 Overburden correction for N

d) Compute the average N value for each borehole; use the smallest average N for the design.
e) Ultimate, not longterm, average loads should be used for the analysis.
f) First select an allowable bearing pressure for the largest footing using the chart given in Fig.10. The chart accounts for both bearing capacity and settlement. Use the same allowable bearing pressure for smaller footings to select their dimensions and check with the chart.
In presence of a water table the allowable bearing pressure should be multiplied by a correction factor C_w. C_w is equal to unity (i.e. no influence) when $D_w > D + B$, but is equal to
$C_w = 0.5 + 0.5D_w/(D + B)$ in other cases; D_w is the depth to water table from ground surface. This correction is controversial. Some argue that unless the water table at the time of construction is different from that at sampling no correction should be applied as the water table had already influenced the measured SPT values.



Vertical axis ; allowable bearing pressure in lbf/ft^2

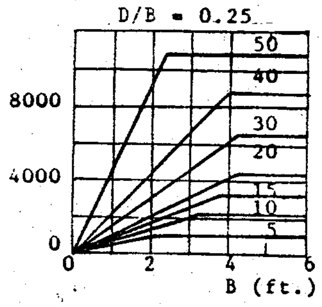


Fig.10 Peck et al design chart using N

3.3.3 Static Cone Penetration Test

In the SCPT first widely used in Holland and Belgium, a 60° apex angle cone with an end area 10 cm^2, is driven into the ground hydraulically. The cone is attached to rods protected by a sleeve. Readings for the thrust on sleeves and cone are taken separately at every 200 mm penetration up to a maximum thrust of 10 tonf.

Meyerhoff (19) suggested the following empirical formulas for allowable bearing pressure to ensure that the maximum settlement is less than 25 mm. His formulas were based on Terzaghi & Peck (17) design curves using SPT values. He proposed the empirical relation $N = 1/4\ C_{kd}$, $(N < 50)$ where, C_{kd} is the cone resistance in kg/cm^2. For square or strip foundations with $B < 4$ ft. allowable bearing pressure q_a is taken as $C_{kd}/30$ $tonf/ft^2$. When $B > 4$ ft.,

$$q_a = C_{kd}\ (1 + 1/B)^2/50\ \text{tonf./ft}^2.$$

q_a should be halved if the water table is at the ground surface. Tomlinson (6) recommends the use of these formulas to find approximate sizes of the foundations and then use either de Beer & Martens (20) or Schmertmann (21) method to determine the actual foundation settlement.

In de Beer and Martens method the constant of compressibility in the classical Terzaghi formula is taken as $C = 1.5\ .\ C_{kd}/p_0$ where p_0 is the effective overburden pressure at the point of measurement. Then the settlement of a layer of given thickness H is equal to $H/C\ \log_e((p_0+\Delta\sigma_z)/p_0)$ where $\Delta\sigma_z$ is the vertical stress increase due to foundation pressure. The total

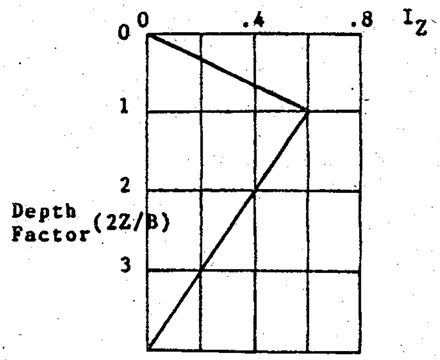


Fig. 11 Vertical Strain Influence Factor

settlement is calculated by dividing the soil into a number of layers and using the average C_{kd} per each layer in the calculations.

Schmertmann (21) proposed a simplified strain distribution curve (see Fig.11) for the strain influence factor I_z at the centre line of a shallow footing. Then the settlement of the foundation is expressed as

$C_1 C_2 q_n\ \Sigma\{I_z/E\ .\ \Delta z\}$ where,

C_1, corrrection for foundation depth given by $1 - 0.5\ p_0/q_n$,

C_2, the correction factor for creep given by $1 + 0.2\ \log_{10}(t/0.1)$; (t - time in years),

q_n net foundation pressure.

E, the Young's modulus of soil for the layer of thickness Δz , is taken to be equal to $2C_{kd}$.

3.3.4 Plate Bearing Test

Plate bearing test was very widely used in early days to determine the bearing capacity of foundations. However, due to the high cost and availability of other techniques it has fallen into disfavour. Still in gravelly or boulderly soils where any other type of testing is extremely difficult it is still the best method. The test is conducted by jacking a plate, normally circular with a diameter of about 0.8 m, into the ground. A typical test arrangement is shown in Fig.12 below.

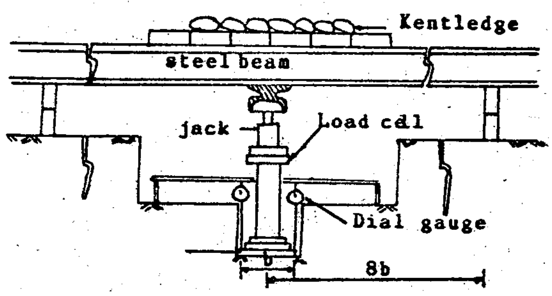


Fig. 12 Plate bearing test

The settlement of the plate is measured with reference to points far away from the plate. The reaction system for the load also should be sufficiently far away from the plate not to influence the measurements. The load is applied in steps allowing settlements to come to a constant in between increments. Refer CP 2001 (22) for full test procedure.
The main problem associated with the plate bearing test is that the area of influence of the loading plate is much less than that of foundation area. Hence, in a layered soils or when water table is present at a moderate depth the test results may be misleading. In a soil with constant Young's modulus E_o the settlement at centre of the plate is given by,

$q_n D(1-\nu^2)/4E_o$ where,

ν is the Poisson's ratio and D, diameter of the plate. This gives a linear variation of settlement with D. However, in most of the soils E_o increases with depth hence the settlement to diameter ratio reduces with depth. Terzaghi & Peck (17) suggested the relation

$\rho_B/\rho_1 = (2B/(B + 1))^2$ where,

ρ_1 is the settlement of a plate of dimensions 1 ft. x 1 ft.; B is also measured in ft.
The results of a plate bearing test can be significantly affected by minor variations in soil compressibility near the base of the plate. Hence, a test should always be carried out at the foundation level. However still, the effect of deeper layers which may influence the actual foundation is not tested. A recent version of the plate load test using a screw plate (Janbu & Seneset (23)) which can be rotated to and tested at several depths in the same location can overcome this problem.

3.4 Allowable Settlement

Settelments can affect the appearence or utility of a structure or may cause damage to it. The extent of allowable settlement depends on the type, importance and function of the structure. Types of settlement which is of interest are uniform settlement, tilt and, differential settlement. The differential settlement is the most important of these three componants as it causes distortion of a structure. The maximum permissible angular distortion for steel framed or reinforced concrete buildings is 1/500 as suggested by Skempton and Macdonald (24). They also suggested a maximum allowable differential settlemnet of 25 mm. for sand and 40 mm. for clay. The total allowable settlement is 40 mm. for sand and 65 mm. for clay. If this criteria cannot be satisfied by a normal design then it is necessary to reduce the settlements using the methods given in the introduction.

3.5 Other Methods

The settlement of a foundation can be calculated by elastic methods if the variation of Young's modulus and Poisson's ratio with depth is known. Similarly the bearing capacity can be determined if the variation of strength with depth is available. These parameters can be determined by laboratory testing of field samples. However, as sampling is difficult in sand and in sensitive clay insitu testing is more suitable. Pressuremeter test in which a cylinderical rubber membrane is inflated in a borehole is gaining increasing popularity in this regard. In this test the relationship between lateral displacement and pressure in the membrane is determined. It is possible to evaluate insitu horizontal effective stress, Poisson's ratio, Young's modulus, and also strength parameters using this method. More sophisticated versions of pressuremeter have pore pressure probes which can be used to determine consolidation parameters as well. There are two popular versions one by Menard and the other called Camkometer pioneered by Soil Mechanics Group of Cambridge University, U.K. Refer Baguelin et al (25) and, Hughes et al (26) for more details. Pressuremeter is normally used with a self boring device and can be used in both sand and clay.

4. CONCLUDING REMARKS

The design methods outlined in this paper are mainly based on research in sand or in sedimentary type clays. However, in this country the majority of soils is of residual nature ; also some organic soils are found in coastal areas. Little research had been carried out on the behaviour of residual soils. Fortunately, settlements are normally not a problem in residual soils; a bearing capacity check using Brinch Hansen method is usually adequate for the design. In peaty soils settlements (especially creep) and, bearing capacity should both be considered in the design. In most of the cases in peat preloading or placing the foundations on a compacted fill above the peat layer may be necessary. There is a great need for better understanding of the behaviour of both residual and organic soils found locally.

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REFERENCES

1. Terzaghi, K. (1943). Theoretical Soil Mechanics. Wiley. New York

2. Prandtl, L. (1920). Uber die Harte plastischer Korper. Nachr. der Ges. der Wiss. Gottingen.

3. Hansen, J. Brinch. (1961). A general formula for bearing capacity. Danish Geotechnical Institute Bulletin No.11.

4. Hansen, J. Brinch. A revised extended formula for bearing capacity. Danish Geotechnical Institute Bulletin No. 28. (1968).

5. Skempton, A.W. (1951). The bearing capacity of clays. Proc. Building Res. Congr. London.

6. Tomlinson, M. J. (1980). Foundation design and construction. Pitman Books Ltd. London.

7. Davis, E. H. & Poulos, H.G. (1968). The use of elastic theory for settlement prediction under three-dimensional conditions. Geotechnique 18. 67-91.

8. Janbu, N., Bjerrum, L. and Kjaernsli, B. (1956). Veildning Vedlosning avfundamenteringsoppgarer, Publication No.16. Nowegian Geotechnical Institute.

9. Christian, J. T. and Carrier, W. D. (1978) Janbu,Bjerrum and Kjaernsli's chart interpreted. Canadian Geotech. J. 15. No.1 123-128.

10. Butler, F. G. (1974) Review paper : Heavily overconsolidated clays. Proc. Conf. on Settelemnt of structures. Cambridge, Pentech Press. 531-578.

11. Simons, N. E. and Menzies, B. K. (1975). A short course in foundation engineering. Butterworths. London.

12. Newmark, N. M. (1935). Simplified computations of vertical pressure in elastic foundations. University of Illinois Engineering Experiment Station Bulletin No. 429.

13. Skempton, A. W. and Bjerrum, L. (1957). A contribution to the settlement analysis of foundations in clay. Geotechnique 7. 168-178.

14. Fox, E. W. (1948) The mean elastic settlement of a uniformly loaded area at a depth below the ground surface. Proc. 2nd Int. Conf. on SMFE. Vol.1 129-132.

15. Mesri, G. (1973) Coefficient of secondary compression. ASCE J. of SMFE V 99. NO.1 123 - 137.

16. Lambe, T. W. (1964). Methods of estimating settlement. Proc. ASCE. Settlement Conf. Northwestern University.

17. Terzaghi, K. and Peck, R. B. (1967) Soil mechanics in engineering practice. 2nd. Ed. Wiley. New York.

18. Peck, R. B., Hansen, W. E. and Thornburn, T. H. (1967). Foundation engineering. 2nd ed. Wiley. New York.

19. Meyerhoff, G. G. (1956) Penetration tests and bearing capacity of cohesionless soils. Proc. ASCE. J. SMFE.

20. de Beer, E. and Martens, A. (1957) Method of computation of an upper limit for the influence of the heterogeneity of sand layers in the settlement of bridges. Proc. 4th Int. Conf. on SMFE. London. Vol.1, 275-282.

21. Schmertmann, J. H. (1970). Static cone to compute static settlement over sand. Proc. ASCE. J of SMFE. 96 SM3. 1011-1043.

22. CP 2001. British Standard Code of Practice for site investigations.

23 Janbu, N. and Senneset, K. (1973). Field Compressometer - principles and applications. 8th. Int. Conf. on SMFE. Moscow. V 1.1,191 - 198.

24. Baguelin, F., Jezequel, J. F. and Le Mehaute, A. (1974). Self-boring placement method of soil characteristics measurement. Proc. Speciality Conf. Subsurface exploration for underground excavation and heavy construction. Henniker. 312-332.

26. Hughes, J. M. O., Wroth, C. P. and Windle, D. (1977) Pressuremeter tests in sands. Geotechnique 27, No.4, 455-477.

SOIL - STRUCTURE INTERACTION

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INTRODUCTION

When attempting to predict analytically the behaviour of a structure or a part of a structure in contact with the earth it is necessary to make an assessment of the loads acting on the surface of contact. These depend on the nature of the interaction of the soil and the structure and generally drastic simplifications become necessary.

Soil - structure interaction for various types of structures have been investigated for about a hundred years. The work of Hertz on the behaviour of "floating" plates in 1884 and that of Zimmermann on the analysis of railway tracks, published in 1888, are perhaps the earliest in this field. Recent publications, e.g. on the behaviour of retaining walls (Potts and Fourie, 1985), negative skin friction on piles (Alonso, Josa and Ledesmai, 1984) and the non-linear behaviour of circular rafts on clay (Hooper, 1983) indicate that there is continuing interest in interaction problems.

An idea of the significance of the study of interaction problems can be obtained by considering the behaviour of the skeletal frame shown in Fig. 1. The conventional analysis considering frame action and flexural effects would give a deflected shape as indicated in Fig. 1(b). The foundations of the columns may thus be considered to be subjected to concentrated axial loads and bending moments. Standard design procedures assuming linear pressure distributions with or without the use of reduction factors, have been found to give satisfactory results.

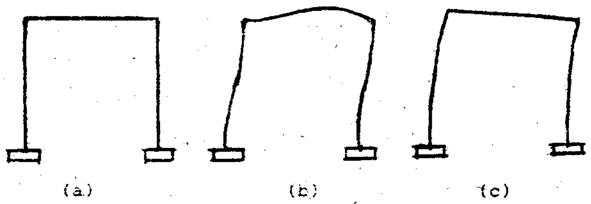

Fig. 1. An example of interactive behaviour

If the frame is infilled by even a thin membrane e.g. a partition, the behaviour is altered considerably, frame action being replaced by cantilever action. This effect can be demonstrated convincingly by using a thin wire frame and a tissue paper infilling. Although not solely a soil-structure interaction problem, this example helps to illustrate that commonly used assumptions may not be quite representative of actual interactive behaviour.

This paper presents some of the assumptions and techniques used in the solution of soil-structure interaction. Early classical approaches and more recent ones are discussed.

The Winkler assumption

Soil-beam interaction

Winkler introduced in 1867 the assumption that the reactive force exerted on a foundation medium was proportional at every point to the deflection of the beam at that point. Zimmermann's work mentioned earlier was based on this assumption.

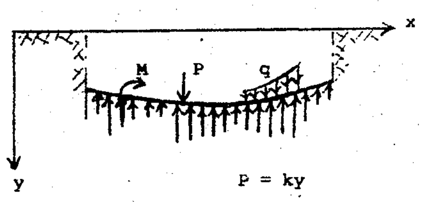

Fig. 2. Beam on Winkler foundation

Consider a beam, initially straight, supported along its entire length and loaded as shown in Fig. 2. The beam will deflect under these loads and give rise to contact pressure of intensity

$$P = ky \qquad \text{..........(1)}$$

according to Winkler's assumption. This implies that the supporting medium is elastic and that it can offer compressive pressures when the beam deflects downwards and tensile stresses when the beam tends to lift off the medium. Other implications are explained in the section continuity effects. k is termed the **modulus of subgrade reaction or foundation modulus.**

The differential equation governing the behaviour of a beam such as that shown in Fig. 2 can be written in the general form

$$EI \frac{d^4y}{dx^4} = -ky + q \qquad \text{..........(2)}$$

In the portion where there is no loading, $q = 0$ and the equation becomes

$$EI \frac{d^4y}{dx^4} = -ky \qquad \text{..........(3)}$$

For constant EI, the general solution to (3) can be expressed in the form

$$y = e^{\lambda x}(c_1\cos\lambda x + c_2\sin\lambda x) + e^{\lambda x}(c_3\cos\lambda x + c_4\sin\lambda x) \quad \text{..(4)}$$

where λ, which is termed the **characteristic** of the system which includes the properties of the beam as well as the foundation medium, is given by

$$\lambda = \sqrt[4]{\frac{k}{4EI}} \qquad \text{.........(5)}$$

In order to obtain solutions to specific cases of loading implied in (2), a particular integral corresponding to q has to be added to the general solution given by (4).

For an infinite beam subjected to simple cases of loading such as a concentrated load or bending moment, concise solutions can be readily obtained and were published by Zimmermann (1888). In a general case of a beam of finite length, the process of obtaining the constants of integration c_1 to c_4 can be tedious. When a beam is subjected to a series of loads, such as the one shown in Fig. 2, it will be necessary to consider the various portions between loads and to use conditions of continuity of these portions. Hetenyi (1946) describes two simplified procedures. In the **method of initial conditions** developed by Russian investigators in the early 1930's, the general solution (4) is expressed in a form in which the constants of integration are replaced by the simple physical quantities deflection, rotation, bending moment and shear force at the end $x = 0$ of the beam. In the **method of superposition** developed by Hetenyi in 1936, the constants of integration are obtained for infinitely long beams. Some of the solutions for deflections are well known standard cases and can be expressed in simple concise forms and have a periodical character. By superposing these solutions, it is possible to obtain solutions for beams of finite length. This procedure is perhaps the simplest to apply to particular problems.

Strain energy methods too can be applied to problems of this nature and are discussed under plate problems.

Soil-plate interaction

Based on the same assumption made earlier, that the intensity of the soil reaction at any point of the bottom of the plate is proportional to the deflection at that point, the differential equation governing the behaviour of the plate can be written as

$$\frac{\partial^4 w}{\partial x^4} + 2\frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^4} = \frac{q}{D} - \frac{kw}{D} \qquad \text{.......(6)}$$

where w is the deflection and D is the plate stiffness.

Classical solutions to plate problems such as those given by Timoshenko & Woinowsky-Krieger (1959) can be adapted to obtain solutions to (6) in some specific cases. Consider for instance the case of the rectangular plate, a x b, resting on a medium having a foundation modulus k and subjected to a concentrated load P at a general point (x^1, y^1) as shown in Fig. 3.

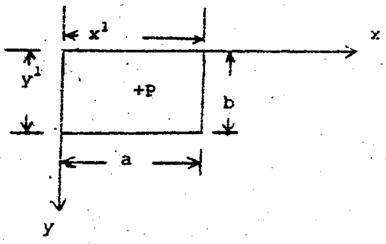

Fig. 3. Rectangular plate under point load

Double trigonometric series of the type first used by Navier in 1820 for the solution of rectangular plates can be used to represent deflections:

$$w = \sum_{m=1}^{\infty}\sum_{n=1}^{\infty} A_{mn} \sin\frac{m\pi x}{a} \sin\frac{n\pi y}{b} \qquad \text{..........(7)}$$

The soil reaction kw too can thus be derived from (7). The applied load function q can be expressed as

$$q = \sum_{m=1}^{\infty}\sum_{n=1}^{\infty} B_{mn} \sin\frac{m\pi x}{a} \sin\frac{n\pi y}{b} \qquad \text{............. (8)}$$

For the case of the concentrated load P at (x^1, y^1), B_{mn} takes the form

$$B_{mn} = \frac{4P}{ab} \sin\frac{m\pi x^1}{a} \sin\frac{m\pi y^1}{b} \qquad \text{..........(9)}$$

For simply supported boundary conditions at the edges, the expression for w takes the form

$$w = \frac{4P}{ab}\sum_{m=1}^{\infty}\sum_{n=1}^{\infty} \frac{\sin\frac{m\pi x^1}{a}\sin\frac{n\pi y^1}{b}}{\pi^4 D\left(\frac{m^2}{a^2}+\frac{n^2}{b^2}\right) + k} \sin\frac{m\pi x}{a} \sin\frac{m\pi y}{b} \quad .(10)$$

The magnitude of w and consequently the bending effects will depend on k and D and it will be seen from (10) that these will decrease with increase of the magnitude of k as well as D. Computations for practical cases will however be tedious.

Use of strain energy methods

Strain energy methods too can be used in the approximate analysis of soil-plate interaction problems. The total energy of the plate-foundation system can be expressed as

$$V = \iint\left(\frac{D}{2}\left[\left(\frac{\partial^2 w}{\partial x^2}+\frac{\partial^2 w}{\partial y^2}\right)^2 - 2(1-\nu)\left[\frac{\partial^2 w}{\partial x^2}\frac{\partial^2 w}{\partial y^2} - \left(\frac{\partial^2 w}{\partial x \partial y}\right)^2\right]\right] - wq + \frac{kw^2}{2}\right) dxdy \qquad \text{..........(11)}$$

The term $\iint \frac{kw^2}{2} dxdy$ represents the energy stored in the foundation while $-\iint wq\,dxdy$ represents the loss in potential energy due to the movement of the applied load. For stable equilibrium, the total energy must be a minimum. The problem is thus reduced to a mathematical one of finding a function $w(x,y)$ which satisfies the boundary conditions and makes V a minimum.

The deflected shape can be assumed to be represented by

$$w = \alpha_1 f_1(x,y) + \alpha_2 f_2(x,y) + \ldots \alpha_n f_n(x,y) \qquad \ldots\ldots(12)$$

where the functions $f_n(x,y)$ are chosen to satisfy the boundary conditions and at the same time represent the shape of the deflected surface. By physical consideration, the probable shape of the deflected plate can be assessed and this could be used as a guide in determining the functions $f_n(x,y)$. The double trigonometric series (7) may be used. For minimum V,

$$\frac{\partial V}{\partial \alpha_1} = 0, \quad \frac{\partial V}{\partial \alpha_2} = 0 .. \frac{\partial V}{\partial \alpha_n} = 0 \qquad \ldots\ldots(13)$$

These give a set of simultaneous linear equations in $\alpha_1, \alpha_2, .. \alpha_n$ which will enable them to be evaluated.

One advantage of this technique is that by choosing one or two terms of (12) some idea of the trend of behaviour can be obtained. As n increases, a more accurate solution can be obtained, but the computation will become more tedious.

Continuity effects in soil

The basic procedure used in the solution of soil-structure interaction problems is to treat the soil as a semi-infinite half space on which the structure rests. Under the action of applied loads, which can be assessed within the limits of commonly used assumptions, and the unknown contact pressure on the soil-structure interface, the structure will undergo deflection. The soil too will in turn undergo deflection under the reactive pressure which will be equal and opposite to the contact pressure on the beam. The deflected shape of the structure must coincide with that of the soil if no portion of the structure lifts off the soil. This general condition can be used to estimate the unknown contact pressure distribution.

The assumption that the contact pressure at any point is proportional to the deflection of the beam or plate at that point implies that this pressure is independent of the pressure or deflection at other parts. As such it represents the case of a floating beam or plate. Such an assumption implies a complete lack of continuity in the soil (see Fig. 4a). This will not generally represent the correct behaviour. At the other extreme, soil may be assumed to have complete continuity and to display the properties of a semi-infinite elastic body (Fig. 4 c). The work of Boussinesq in 1885 is perhaps the earliest classical elasticity solution. The analysis of particular cases presents complex mathematical problems. Actual behaviour will not be represented by this assumption either. Some experimental investigations have shown that in certain soils deformation is localised mainly in the loaded region. For such cases, theoretical calculations based on the Winkler assumption could give good agreement with practical results.

For the solution of problems where the deformation is partly localised and partly continuous (see Fig. 4 b.), a number of simplified soil response models have been proposed.

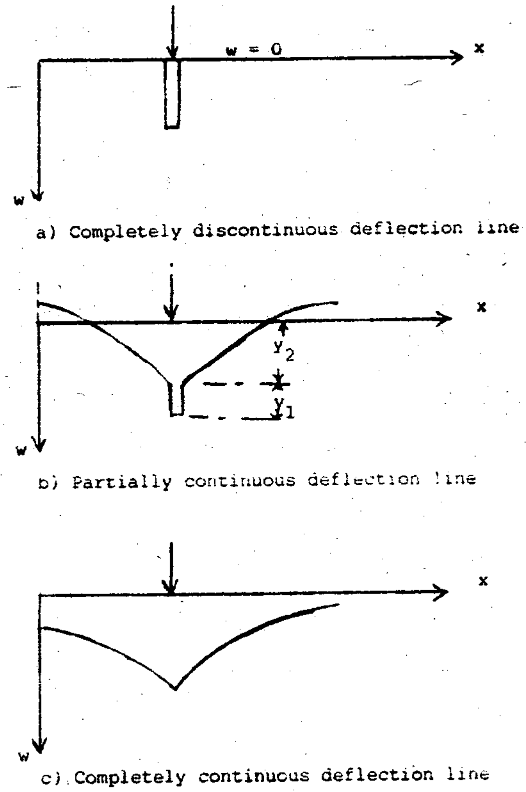

Fig. 4. Deflection under unit load

Cases where partial continuity is assumed are sometimes termed two-parameter systems and may represent actual cases better than the extreme cases.

In the Filonenko-Borodich model, developed in the early 1940s, continuity is realised by incorporating a stretched elastic membrane over the Winkler model (see Fig. 5) which is without continuity.

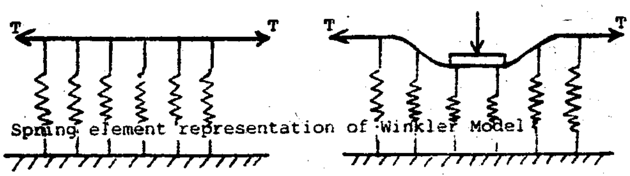

Fig. 5. The Filonenko-Borodich Model

Hetenyi (1946) presented a model for partial continuity by assuming a continuous member imbedded in the soil which itself displays no continuity (see Fig. 6).

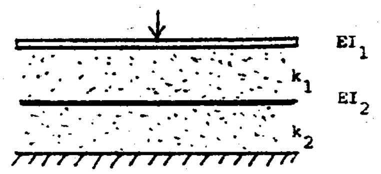

Fig. 6. The Hetenyi Model

By proper selection of the moduli k_1, k_2 and the flexural rigidity of the beam in Fig. 6, any ratio of the continuous and discontinuous components of the deflection (i.e. y_1/y_2 in Fig. 4b) can be attained. Various other models e.g. the Pasternak model presented in 1954 and the Reissner model presented in 1958 are described by Selvadurai (1979).

Classification based on relative flexibility of soil and structure

In the section on soil-beam interaction, the term λ, the characteristic of the system was introduced. The non-dimensional quantity λl, where l is the length of the beam is a measure of the relative flexibility of the soil-beam system. This incorporates the material and geometrical properties of the beam and the deformational characteristics of the soil. The magnitude of this parameter may be used to obtain approximations to the general analysis by neglecting certain quantities which will result in simpler solutions.

Based on the Winkler assumption, a foundation beam may be classified as indicated by Hetenyi (1946) as

Group I: Short beams	$\lambda l < \pi/4$
Group II: Medium length beams ...	$\pi/4 < \lambda l < \pi$
Group III: Long beams	$\lambda l > \pi$

In Group I, flexural deformations can be neglected. The conventional rigid footing analysis with linear contact pressure distributions obtained by direct application of the equations of equilibrium (see Fig. 7) are appropriate.

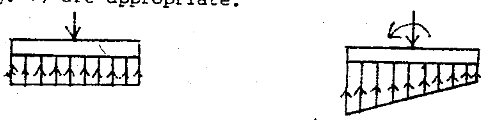

Fig. 7. Linear contact pressure distributions

Group II and III require more accurate analysis. In Group III, however, the analysis is made simpler because solutions for infinite beams can be used.

Similar classifications have been introduced even when partial continuity or complete continuity of the soil is assumed (see for instance, Selvadurai, 1979).

Horizontal contact pressures

Contact pressure consists not only of vertical effects but also horizontal effects. Horizontal displacements are produced even when only vertical applied loads are present and these are resisted by friction and shearing effects. Thus horizontal contact pressures too are developed. Although these are considered to be of secondary importance and are usually neglected, they could affect the behaviour of foundations. The presence of ground water and time-dependent effects can alter frictional forces at the interface and any assumption will therefore have its limitations. Cheung and Nag (1968) describe the use of the finite element technique to take into account horizontal effects too.

Non-linear behaviour

For flexible beams and plates, linear elastic analysis can give compression as well as tensile contact pressures. The contact surface along the soil-structure interface cannot usually transmit tensile forces and there will be a tendency to separate. The actual behaviour will thus be non-linear. Cheung and Nag (1968) discuss an iterative procedure using finite element techniques to deal with such cases.

Even if the contact pressure is entirely compressive some of the high values obtained by elastic analysis cannot be sustained in practice. Appreciable local yielding will take place resulting in non-linear behaviour. The technique of dealing with such a situation is to obtain a solution by elastic analysis and to refine this by an iterative procedure in which the high contact pressures are reduced to a specified limiting value. Excess reaction forces are distributed to other areas of the foundation. Hooper (1983) discusses this procedure in relation to the behaviour of circular rafts.

Numerical techniques of analysis

The classical solutions outlined in the earlier sections are not always suitable for the investigation of practical problems. Various aspects such as foundations of variable thickness, geometrically irregular shapes, variation of soil properties, loss of soil support, time dependent phenomena and the influence of neighbouring structures cannot be easily included in such solutions

The finite difference technique

The differential equations governing the soil-structure interaction and the boundary conditions for a particular problem can be replaced by their corresponding finite difference equations using forward, central or backward difference formulations. A set of simultaneous equations are thus obtained which can be easily solved. Allen and Severn (1960, 1961, 1963) discuss several examples of applications to soil-plate interaction problems using Winkler assumptions. Bowles (1977) too discusses finite difference applications to such problems.

The finite element technique

The soil-structure continuum can be replaced by a discrete system consisting of a number of finite elements, which may be triangular, rectangular etc. These elements are assumed to be connected together at a discrete number of nodal points at their boundaries. The displacements of these points are treated as the basic unknowns. Cheung and Nag (1968) explain the application of this technique which is quite versatile.

An influence coefficient technique

A solution proposed by Barden (1962) appears to be capable of providing practical solutions to a wide variety of soil-structure interaction problems. This can deal with homogeneous isotropy and anisotropy, compressibility decreasing with depth as well as cases of stratifications. The vertical contact pressure distribution is assumed to be made up of a number of steps of uniform values P_1, P_2, ...P_n as shown in Fig. 8. Estimates of deflections based on an assumption such as this are known to be reasonably accurate.

The general procedure is to equate structure and soil deflections. For this purpose, a common datum is required and in the case of

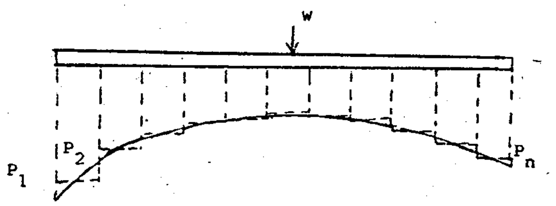

Fig. 8. Approximate representation of contact pressure variation

a beam, the line XY connecting the ends can be taken as shown in Fig. 9.

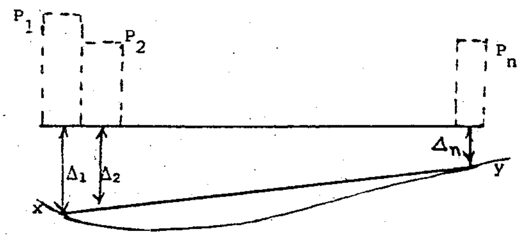

Fig. 9. Deflection of soil due to contact pressure

The deflections in the soil under each step can be evaluated using standard solutions for half-space models. By equating beam and soil deflections, n - 2 simultaneous equations can be obtained for the solution of the n unknowns. Two other equations can be obtained by considering the vertical equilibrium ($\Sigma P_n = w$) and rotational equilibrium ($\Sigma P_n l = 0$). Barden presents the solution in the form of a single set of influence coefficients, governed by a non-dimensional parameter, ϕ, which depends on the properties of the soil-structure system. This corresponds to λl introduced earlier in connection with the classical theory.

The solutions have been demonstrated to correspond closely with those obtained using other techniques.

Experimental and field investigations

The validity of a pattern of distribution of contact pressure derived from analysis, and hence the ability of the structure to behave as expected, will depend on the validity of the idealisations of soil behaviour discussed earlier. The magnitudes of the parameters such as the foundation modulus and the other properties of the soil and the structure have to be assessed by using experimental data. Whether or not any particular assumption of soil behaviour gives results which correspond to reality can only be verified through experimental investigation and field observations.

Experimental investigations had been carried out by various investigators since the late 1920s and several publications are cited by Selvadurai (1979). However there is much scope for more comprehensive experimental and field investigations.

Concluding remarks

Generally, the Winkler model seems to give acceptable results in the case of cohesionless soils while two-parameter models seem to provide better representations of cohesive soils. Classical solutions as well as approximate numerical solutions give similar results.

References

1. Allen, D.N. de G. & Severn, R.T. (1960). The stresses in foundation rafts. I. Proc. Inst.Civ.Eng. 15: 35-48.
(1961). The stresses in foundation rafts. II. Proc. Inst.Civ.Eng. 20: 293-304.
(1963). The stresses in foundation rafts. III. Proc.Inst.Civ.Eng. 25: 257-266.

2. Alonso, E.E., Josa, A. & Ledesmai, A. (1984). Negative skin friction on piles: a simplified analysis and prediction procedure. Geotechnique 34: 341-357.

3. Barden, L. (1962). Distribution of contact pressure under foundations. Geotechnique, 13: 87-88.
(1963). Winkler model and its application to soil, Struct.Eng. 41: 279-280.

4. Bolton, R. (1972). Stresses in circular plates on elastic foundations. J.Eng.Mech.Div. ASCE EM3, 98: 629-640.

5. Bowles, J.E. (1977). Foundation Analysis and Design. McGraw Hill, New York. 2nd Ed.

6. Cheung, Y.K. & Nag, D.K. (1968). Plates and beams on elastic foundations - linear and non-linear behaviour. Geotechnique, 18: 250-260.

7. Hetenyi, M. (1946). Beams on elastic foundation Ann.Arbor: Univ. Michigan Press.

8. Hooper, J.A. (1983). Non-linear analysis of a circular raft on clay. Geotechnique 33: 1-20.

9. Potts, D.M. & Fourie, A.B. (1985). The effect of wall stiffness on the behaviour of a propped retaining wall. Geotechnique 35: 347-352.

10. Selvadurai, A.P.S. (1979). Elastic analysis of soil-foundation interaction - Elsevier, B. Amsterdam.

11. Timoshenko, S. & Woinowsky-Krieger, S. (1959). Theory of Plates and Shells. 2nd ed. London: McGraw-Hill.

12. Vesic, A.B. (1961). Beams on elastic subgrade and the Winkler hypothesis. Proc.5th.Int. Conf.Soil Mech., Paris, pp. 845-850.

DESIGN OF MACHINE FOUNDATIONS

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ABSTRACT

In addition to static loads, the machine foundations are subjected to dynamic forces caused by the working of machine. These dynamic forces are then transmitted to foundations supporting machines. Therefore a satisfactory design can be made only if the designer is well conversant with the method of load transmission from the machine as well as with the problems concerning the dynamic behaviour of the foundation and the soil underneath it. This paper outlines the analysis of a block foundation using linear spring theory.

INTRODUCTION

Foundation supporting machines are subjected to two types of loading i.e static loads and dynamic loads. The characteristic feature, in the case of static loads, is that for a given structure the load carried by the foundation at any given time is constant in magnitude and direction. The dynamic forces are intertia forces produced by the motion of various moving parts in a machine in accordance with Newton's law. In general these forces are periodic and give rise to vibrations. When there are several moving parts, these forces set up a corresponding number of separate vibrations in machine/foundation system. Therefore the design of a machine foundation is more complex than that of a foundation which supports only static loads.

There are three different methods of approach in the design of machine foundations. They are as follows:

(a) Considering soil as an elastic spring
  (i) without weight, and
  (ii) with weight

(b) Considering soil as continuous elastic medium

(c) Empirical or semi-empirical methods.

The method suggested by Barkan which is based on linear spring theory and which neglects the effects of damping and participating soil mass is the most popular among the methods mentioned above. The basis for the analysis of a block foundation using Barkan's method is presented in this paper. Further details of this technique and applications are available in Barkan (1962), Sirinivasulu & Vaidyanathan (1976) and Prakash et.al (1979).

TYPES OF MACHINES AND FOUNDATIONS

Broadly, the machines may be devided into three catagories based on their operating speeds and the nature of unbalanced forces prpduced during operation.

(a) Those producing periodical forces e.g. compressors and reciprocating engines. The speed of operation of such machines is usually smaller than 600 rpm. The unbalanced force varies sinusoidally.

(b) Those producing impact forces e.g forge hammers, presses. Their speed of operation is usually 60 to 150 blows per minute. The dynamic load attains a peak in a very short interval and then dies out completely.

(c) High speed machinery such as turbines and rotary compressors. These machines may have speeds more than 3000 rpm upto 10,000 rpm.

Considering their structural form, machine foundations are generally classified as follows:

(a) Block-type foundations (Fig.1a) consisting of a pedestal of concrete on which the machine rests.

(b) Box or Caisson-type foundations (Fig.1b) consisting of a hollow concrete block supporting the machinery on their top.

(c) Wall type foundations (Fig.1c) consisting of a system of wall-columns and beam slabs. Each element of such a foundation is relatively flexible as opposed to rigid mass of a block and a box type foundation.

GENERAL THEORY

Vibration problems are encountered in most machines due to unbalanced forces of rotating parts. There are occasions when these vibrations are excessive and hence objectionable for the machine as well as the supporting system. In order to analyse any vibration problem one should try to understand and study the mechanicsm of vibration. The ideal model to understand this mechanism is a single-degree freedom system.

Theroy of a Single - Degree Freedom System

Consider a single-degree freedom system (Fig 2) formed by a rigid mass m resting on a linearly elastic spring of stiffness K and viscous damper

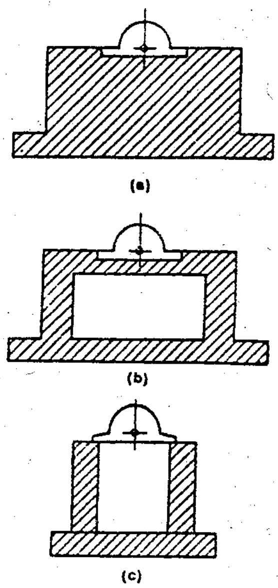


Fig.1 Type of Machine Foundations
(a) Block Type (b) Box Type (c) Wall Type

having damping coefficient C. This system is said to be of a single - degree freedom as it moves only in one direction freely or with some external force.

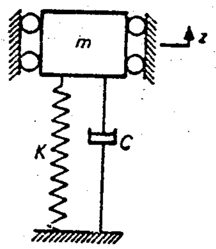


Fig.2 Single Degree Freedom System

Let the system be set in motion by giving an initial velocity V to the mass. The equation of motion for the free vibration of the system is

$$\underset{\text{inertial force}}{m\ddot{z}} + \underset{\text{damping force}}{C\dot{z}} + \underset{\text{spring force}}{Kz} = 0 \quad (1.1)$$

In Eq. 1.1, z denotes the displacement, $\dot{z}$ the velocity and $\ddot{z}$ the acceleration of the mass. The right hand side is zero since there is no external force on the system during vibration.

The solution of Eq 1.1 can be written as

$$z = a_d\, e^{-Ct/2m} \sin\sqrt{\frac{K}{m} - \frac{C^2}{4m^2}}\, t \quad (1.2)$$

where a_d is a constant which represent , the maximum displacement and known as 'free amplitude' of the damped system. The frequency of Oscillation (ω_{nd}) is given by

$$\omega_{nd} = \sqrt{\frac{K}{m} - \frac{C^2}{4m^2}} \quad (1.3)$$

To obtain the free amplitude a_d, the initial conditions at the time when the motion was set in, should be considered, i.e. when $t = 0$, $z = 0$, $\dot{z} = V$.
Substituting

$$a_d = V \Big/ \sqrt{\frac{K}{m} - \frac{C^2}{4m^2}} \quad (1.4)$$

Substituting $C_c = 2\sqrt{km}$, where C_c is called the 'critical damping' and $C/C_c = \zeta$, where ζ is called the 'damping ratio' Eqs. 1.3 and 1.4 can be rewritten as

$$\omega_{nd} = \sqrt{\frac{K}{m}}\,\sqrt{1-\zeta^2} \quad (1.5)$$

$$a_d = V \Big/ \sqrt{\frac{K}{m}(1-\zeta^2)} \quad (1.6)$$

Eqs. 1.5 and 1.6 give the 'damped natural frequency' and the 'damped free amplitude' of a single degree freedom system. If damping is neglected, $C = 0$, or $\zeta = 0$.

Eqs. 1.5 and 1.6 reduce to

$$\omega_n = \sqrt{\frac{K}{m}} \quad (1.7) \qquad a = V\sqrt{\frac{m}{K}} \quad (1.8)$$

The relations given above are used in analysis of foundations for impact type machines. Let the system shown in Fig 2, be subjected to a harmonic exciting force $P_0 \sin \omega_m t$. Depending on the type of excitation, two cases can be considered, One case in which amplitude of excitation is constant and the other in which the amplitude is proportional to the square of the circular operating frequency ω_m. The latter case which occurs in reciprocating or unbalanced rotating mechanisms is of main interest in machine foundations.

The equation of motion of a damped s.d.f system subjected to constant force excitation can be written as

$$\underset{\text{inertial force}}{m\ddot{z}} + \underset{\text{damping force}}{C\dot{z}} + \underset{\text{spring force}}{Kz} = \underset{\text{exciting force}}{P_0 \sin \omega_m t} \quad (1.9)$$

where P_0 is the amplitude of exciting force.

The solution of Eq 1.9 under steady - state conditions may, therefore, be expressed as

$$z = a_d \sin(\omega_m t + \alpha) \quad (1.10)$$

where a_d is amplitude and α is the phase difference between the exciting force and displacement.

Eq 1.9 can now be solved to obtain following expressions for a_d and α .

$$a_d = \frac{P_0}{\sqrt{(K - m\omega_m^2)^2 + C^2 \omega_m^2}} \qquad (1.11a)$$

$$\tan\alpha = \frac{C\omega_m}{K - m\omega_m^2} \qquad (1.11b)$$

Substituting

$$\omega_n^2 = K/m,\ \zeta = C/(2\sqrt{Km}) \text{ and } \eta = \omega_m/\omega_n$$

Eqs. 1.11a and 1.11b can be reduced to:

$$a_d = \frac{P_0}{K\sqrt{(1-\eta^2)^2 + (2\eta\zeta)^2}} \qquad (1.12a)$$

$$\tan\alpha = \frac{2\eta\zeta}{1-\eta^2} \qquad (1.12b)$$

Substituting $P_0/K = z_{st}$, the static displacement Eq. 1.12a can be written as .

$$z = z_{st}\,\mu \qquad (1.13a)$$

$$\mu = \frac{1}{\sqrt{(1-\eta^2)^2 + (2\eta\zeta)^2}} \qquad (1.13b)$$

Here μ is called the 'dynamic magnification factor' Fig. 3 shows the variations of μ with η for various values of ζ.

In the case of reciprocating or unbalanced rotating mass type excitation the exciting force P is of the form

$$P = (m_e e\omega_m^2)\sin\omega_m t \qquad (1.14)$$

where m_e is the reciprocating or unbalanced rotating mass e denotes the displacement in the case of reciprocating type and eccentricity of unbalanced mass in the case of rotating type mechanisms, and ω_m is frequency of motion.

The equation of motion for a s.d.f. system subjected to this type of forced excitation is written as

$$m\ddot{z} + C\dot{z} + Kz = (m_e e\omega_m^2)\sin\omega_m t \qquad (1.15)$$

Substituting Po $P_0 = m_e e\omega_m^2$ in 1.11a, the solution becomes

$$a_d = \frac{m_e e\omega_m^2}{\sqrt{(K - m\omega_m^2)^2 + C^2\omega_m^2}} \qquad (1.16)$$

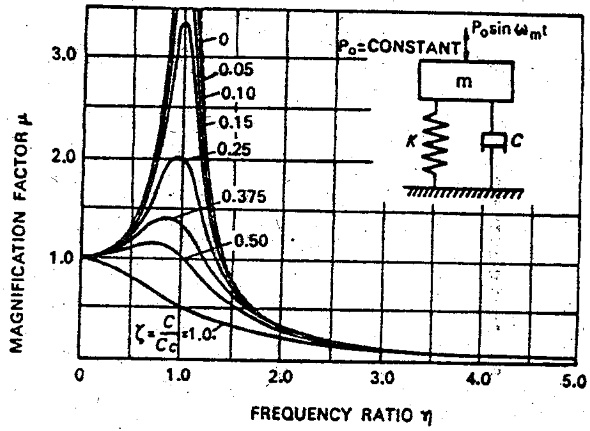
Substituting $\omega_n^2 = K/m;\ \zeta = \dfrac{C}{2\sqrt{Km}}$ and $\eta = \omega_m/\omega_n$

Eq 1.16 gives

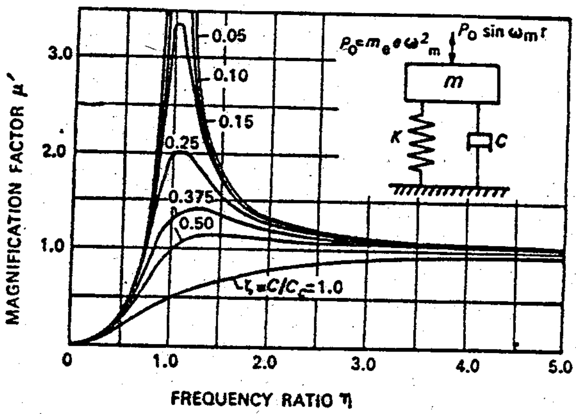
$$\frac{a_d}{m_e e/m} = \eta^2 \frac{1}{\sqrt{(1-\eta^2)^2 + (2\eta\zeta)^2}} \qquad (1.17)$$

or

$$\mu' = \eta^2\mu \qquad (1.18)$$



(a) Constant Force Excitation



(b) Rotating Mass Type Excitation

Fig.4 Response of a Single Degree Damped System where μ' is the 'magnification factor' defined by the left-hand side of Eq. 1.17; μ is the magnification factor for the corresponding case of constant force excitation (Eq 1.13b). Fig 4 shows the variation of μ' with η (Eq 1.18) for various values of ζ. The expression for α is the same as that given in Eq 1.12b.

When damping in the system is neglected, i.e, $C = 0$ or $\zeta = 0$, then

$$\mu = \frac{1}{1-\eta^2} \text{ for constant force excitation} \qquad (1.19a)$$

$$\mu' = \frac{\eta^2}{1-\eta^2} \text{ for rotating-mass type excitation} \qquad (1.19b)$$

Further when $\eta = 1$, both μ and μ' become infinity. This marks the stage of 'resonance'.

In practice,the amplitude at resonance will be finite because of damping which is inherently present in any physical system. It is,however, desirable to ensure in the design of any dynamically loaded structure that the value of frequency ratio η, is far from unity. According to IS: 2974 (Pt.I), the working range for the frequency ratio η is given by the inequality

$$1.4 < \eta < 0.5 \qquad (1.20)$$

Fig. 4 shows the amplitude - frequency relations for damped-forced vibration of a mass-spring system under the action of constant force type and rotating mass type excitatios.

The expressions for resonant frequency and amplitudes for a viscously damped single - degree freedom system for the two cases are given in Table 1 (See Appendix)

The theory of a single mass spring system under forced vibrations is used in the analysis of block foundations for reciprocating or rotating type of machinery.

UNBALANCED FORCES IN MACHINES

The machine manufacturers generally provide data concerning unbalanced forces in them. However for certain basic types of equipment the unbalanced forces can be computed.

Internal combustion engines, piston-type compressors,pumps,steam engines, etc., produce reciprocating forces. Fig. 5 illustrates the main features of a reciprocating mechanism in which a crank mechanism converts the reciprocating motion to a rotary motion, and vice versa.

The piston A and the piston rod B execute an alternating motion, the connection rod C executes a complicated periodic motion and all points on crank D execute a rotational motion around the main axis O.

Designating the total reciprocating mass which moves with the piston as m_{rec} and the rotating mass moving with with the crank as m_{rot} the unbalanced inertial forces P_z (along the direction of piston) and P_x (along the perpendicular direction) can be written as

$$P_z = (m_{rec} + m_{rot})\, r\, \omega_m^2 \cos \omega_m t + m_{rec} \frac{r^2 \omega_m^2}{l} \cos 2\, \omega_m t \qquad (2.1)$$

and

$$P_x = m_{rot}\, r \omega_m^2 \sin \omega_m t \qquad (2.2)$$

where ω_m is the angular speed of rotation and r is the radius of crank.

The reciprocating and rotating masses are given by

$$m_{rec} = m_2 + m_3 \left(\frac{l_1}{l} \right) \qquad (2.3)$$

$$m_{rot} = \frac{r_1 m_1}{r} + \left(1 - \frac{l_1}{l} \right) m_3 \qquad (2.4)$$

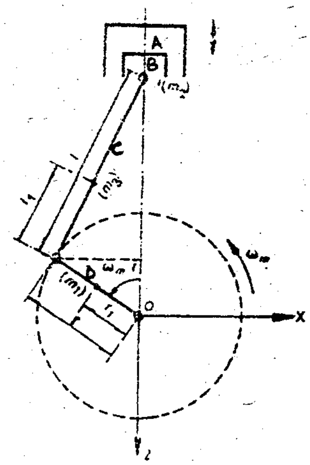


Fig.5 Simple Crank Mechanicsm

Turbines,centrifugal pumps and turbogenerators are examples of rotating machine.The main moving units in these machines are rotors which execute simple rotating movements. Although rotating machinery is balanced before erection, in actual operation some imbalance always exists. This is due to the reason that the centre of gravity of the rotating parts does not exactly coincide with the axis of rotation. Even a minor eccentricity can produce large unbalanced forces in high speed machines with heavy rotors. Consequently their influence should be taken into account in the design of foundations.

Unbalanced vertical force for a rotating mass type oscillator in which a single mass m_e is placed on a rotating shaft at an eccenentricity e from the axis of rotation is given by $P = m_e e\, \omega_m^2 \sin \omega_m t$.

DYNAMIC PROPERTIES OF SOIL

The evaluation of certain soil parameters, is required in the analysis and design of machine foundations using the theory based on the undamped linear spring analogy as proposed by Barkan (1962).

(i) Coefficient of elastic uniform compression, C_z
(ii) Coefficient of elastic uniform shear, C_τ
(iii) Coefficient of elastic non-uniform compression, C_ϕ
(iv) Coefficient of elastic non-uniform shear, C_ψ

The coefficient of elastic uniform compression (C_z) is defined as the ratio of compressive stress applied to a rigid foundation block to the 'elastic' part of the settlement induced consequently. The coefficient of elastic uniform shear (C_ϕ) may likewise be defined as the ratio of average shear, stress at the foundation contact area to the 'elastic' part of the sliding movement of the foundation.

These coefficients are functions of soil type and size and shape of the foundation. The various methods to obtain these parameters i.e. empirical, laboratory and in-situ and the factors affecting them are described in detail in Barkan (1962) Srinivasulu & Vaidyanathan (1976) and in Indian

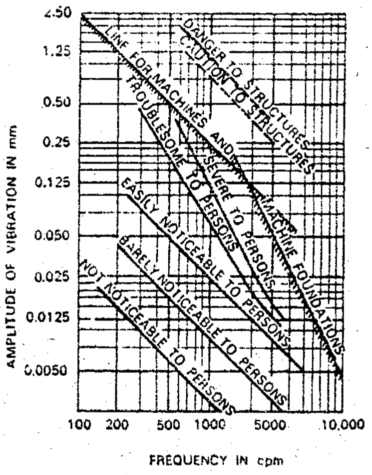


Fig. 6 Allowable Limits for Vertical Vibration Amplitudes

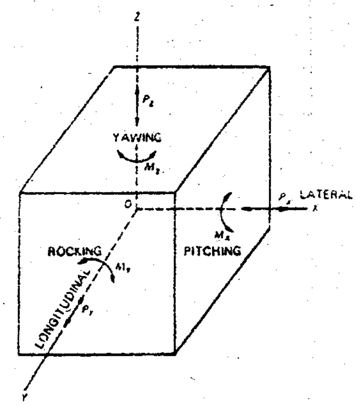


Fig.7 Modes of Vibration of a Block Foundation

Standards.

Using the soil parameters mentioned above the spring coefficients (K) of soil for various modes of vibrations are computed using following expressions.

(i) For vertical motion $K_z = C_z A_f$
(ii) For horizontal (or sliding) motion $= K_\tau = C_\tau A_f$
(iii) For rocking motion $K_\phi = C_\phi I_{x (or\ y)}$
(iv) For torsional motion (rotation about vertical axis) $K_\psi = C_\psi I_z$

where A_f is area of horizontal contact surface between foundation and soil. I is the second moment of contact area about the horizontal axis (x or y) passing through the centroid of the base and normal to the plane of rocking, and I_z is second moment of contact area about the vertical axis passing through the centroid of the base.

GENERAL REQUIREMENTS OF MACHINE FOUNDATIONS

A macine foundation should satisfy the following requirements for satisfactory action.

(i) It should be able to carry the superimposed loads without causing shear or crushing, failure
(ii) It should not settle excessively.
(iii) The combined centre of gravity of machine and foundation should as far as possible be in the same vertical line as the centre of gravity of the base plane.
(iv) No resonance should occur, hence the natural frequency of foundation soil system should be either too large or too small compared to the operating frequency of the machine.
(v) The amplitudes under service conditions should be within permissible limits.
(vi) The vibrations must not be harmful to workers or other machines. The nature of vibrations, whether perceptible, annoying or harmful depends upon the frequency of vibration and amplitude of motion. The variation of frequency and amplitude for vertical vibrations for different conditions is shown in Fig.6 (Richardt, 1960).

From a practical point of view, the following requirements must be fulfilled:

(i) The ground-water table should be as low as possible
(ii) Machine foundations should be separated from adjacent building components by means of expansion joints.
(iii) Machine foundations should be taken to a level lower than the foundations of adjoining buildings.

ANALYSIS OF A BLOCK FOUNDATION

Under the action of unbalanced forces, a rigid block may undergo displacement and oscillations as shown in Fig. 7.

(i) Translation along Z - axis
(ii) Translation along X - axis
(iii) Translation along Y - axis
(iv) Rotation about Z - axis
(v) Rotation about X - axis
(iv) Rotation about Y - axis

Any rigid displacement of the block can be resolved into these six independent displacements. Hence the rigiid block has six degrees of freedom and six natural frequencies (one corresponding to each mode of vibration). The vibratory modes may be 'coupled' or decoupled depending on the centre of gravity of the machine foundation and the centroid of its base area. The natural frequency is determined in a particular mode (coupled or decoupled) and compared with the operating frequency.

The follwong assumptions make it possible to consider the foundation soil system as a mass-spring system in the analysis of block foundations using Barkan's method.

(i) There is a linear relation between the soil reacting on vibrating foundation and displacement of this foundation and it is determined by elastic coefficients C_z and C_θ

(ii) The soil underneath the foundation does not have inertial properties but only elastic properties as described by these coefficients.

The natural frequencies and the amplitudes for various mode of vibrations are given below.

a. Vertical Translation

i. The circular natural frequency (ω_z) for uncoupled vertical translation along the z axis is given by

$$\omega_z = \sqrt{K_z/m} \qquad (3.1a)$$

For foundations resting directly on soils

$$\omega_z = \sqrt{\frac{C_z A_t}{m}} \qquad (3.1b)$$

ii. The vertical amplitude (a_z) under the action of an exciting force $P_z \sin \omega_m t$, ω_m being the circular operating frequency is given by

$$a_z = \frac{P_z}{m(\omega_z^2 - \omega_m^2)} \qquad (3.2)$$

b. Sliding and Rocking Motion in xz Plane

I. Natural frequencies: The two natural frequencies $\omega_{n_1}, \omega_{n_2}$ which represent the coupled motion (sliding along x axis and rocking about y axis) in the xz plane given by the roots of the following quadratic equation in ω_n^2

$$\omega_n^4 - \left(\frac{\omega_{\theta y}^2 + \omega_x^2}{\alpha_y}\right)\omega_n^2 + \frac{\omega_{\theta y}^2 \omega_x^2}{\alpha_y} = 0 \qquad (3.3)$$

where α_y is the ratio of the mass moment of inertia (φ_y) about the y axis passing through centre of gravity to the mass moment of inertia (φ_{oy}) about a parallel axis through the centre of elasticity of the base support.

$$\alpha_y = \varphi_y/\varphi_{oy} \qquad (3.4)$$

$$\omega_{\theta y}^2 = (K_{\theta y} - WS)/\varphi_{oy} \qquad (3.5a)$$

$$\omega_x^2 = K_x/m \qquad (3.5b)$$

For foundations resting on soils

$$\omega_{\theta y}^2 = \frac{C_\theta I_y - WS}{\varphi_{oy}} \qquad (3.6a)$$

and

$$\omega_x^2 = \frac{C_\tau A_t}{m} \qquad (3.6b)$$

The terms ω_x and $\omega_{\theta y}$ are called the 'limiting frequencies' of the coupled motion; ω_x represents the natural circular frequency for 'pure sliding' along the x axis when the foundation is assumed to possess infinite resistance to rocking (about the y axis) and $\omega_{\theta y}$ denotes the natural circular frequency for 'pure rocking' (about the y axis) when the foundation is assumed to possess infinite resistance to sliding (along the x axis).

The two roots ω_{n_1} and ω_{n_2} of Eq.3.3 are given by

$$\omega_{n1}^2 = \frac{1}{2\alpha_y}\left[\omega_{\theta y}^2 + \omega_x^2 + \sqrt{(\omega_{\theta y}^2 + \omega_x^2)^2 - 4\alpha_y \omega_{\theta y}^2 \omega_x^2}\right] \qquad (3.4a)$$

$$\omega_{n2}^2 = \frac{1}{2\alpha_y}\left[\omega_{\theta y}^2 + \omega_x^2 - \sqrt{(\omega_{\theta y}^2 + \omega_x^2)^2 - 4\alpha_y \omega_{\theta y}^2 \omega_x^2}\right] \qquad (3.4b)$$

The foundation vibrates with circular natural frequencies ω_{n_1} and ω_{n_2} (where $\omega_{n_1} > \omega_{n_2}$) about two centres of rotation - O_1 and O_2 (Fig.8) - which are situated at distances

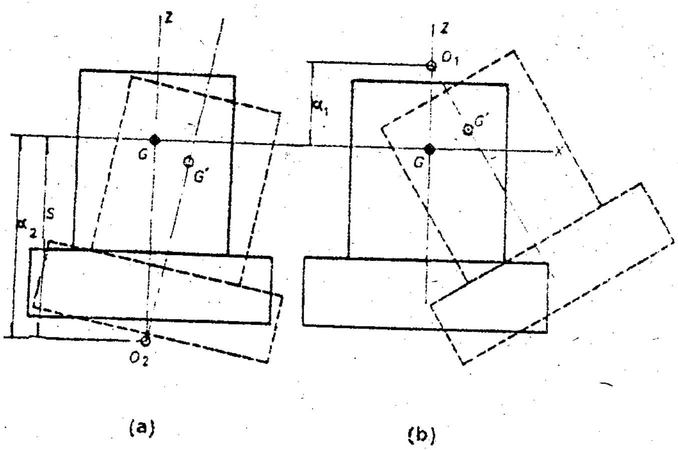


Fig.8 Centres of Rotation for Coupled Sliding and Rocking Motion in x-z Plane
(a) First Mode (b) Second Mode

α_1 and α_2 respectively from the common centre of gravity where

$$\alpha_1 = \frac{\omega_z^2 S}{\omega_x^2 - \omega_{n1}^2} \quad (3.5a)$$

$$\alpha_2 = \frac{\omega_z^2 S}{\omega_x^2 - \omega_{n2}^2} \quad (3.5b)$$

$$\alpha_1 \alpha_2 = \varphi_y / m \quad (3.5c)$$

ii. Amplitudes: The horizontal amplitude (a_x) and rotational amplitude ($a_{\theta y}$) of the foundation subjected to the simultaneous action of an exciting force $P_0 \sin \omega_m t$ and an exciting moment $M_y \sin \omega_m t$ are given by

$$a_x = \left[(K_{\theta y} - WS + K_x S^2 - \varphi_y \omega_m^2) P_x + (K_x S) M_y \right] \frac{1}{f(\omega_m^2)} \quad (3.6a)$$

and

$$a_{\theta y} = \left[(K_x S) P_x + (K_x - m\omega_m^2) M_y \right] \frac{1}{f(\omega_m^2)} \quad (3.6b)$$

where $$f(\omega_m^2) = m \varphi_y (\omega_{n1}^2 - \omega_m^2)(\omega_{n2}^2 - \omega^2 \quad (3.7)$$

The net horizontal displacement (along the x axis) of the upper edge of the foundation is equal to

$$a_x + (H - S) a_{\theta y} \quad (3.8)$$

where H is height of foundation.

c. Sliding and Rocking Motion in yz Plane

The natural frequencies of the coupled sliding (along the y axis) and rocking (about the x axis) motion of the foundation are given by an equation similar to Eq. 3.3 obtained with the suffuxes x and y interchanged in it.

The amplitudes a_y and $a_{\theta x}$ may likewise be obtained from Eqs. 3.9a and 3.9b with the suffixes x and y interchanged. The net horizontal amplitude (along the y axis) of the upper edge of foundation is then

$$a_y + (H - S) a_{\theta x} \quad (3.9)$$

d. Yawing or Twisting Motion about z axis

As mentioned earlier, the yawing motion is uncoupled and the natural frequency (ω_ψ) for twisting mode and the amplitude under the action of a twisting moment $T_0 \sin \omega_m t$ are given by the following expressions

$$\omega_\psi = \sqrt{K_\psi / \varphi_z} \quad (3.10a)$$

$$\omega_\psi = \sqrt{\frac{C_\psi I_z}{\varphi_z}} \quad (3.10b)$$

$$a_\psi = \frac{1}{(\omega_\psi^2 - \omega_m^2)} \times \left\{ \frac{T_0}{\varphi_z} \right\} T_0 \quad (3.11)$$

If the combined centre of gravity of the machine and foundation and the centroid of the foundation base do not lie in the same vertical line, the vertical vibration is not independent of horizontal vibration and rocking. In this case, vertical, horizontal and rocking vibrations in xz (or yz) planes are intercoupled and the three coupled natural frequencies ω_1, ω_2 and ω_3 (three in each plane xz and yz) are given by the roots of the following expression:

$$\omega_n^2 e_x^2 = \frac{\alpha (\omega_z^2 - \omega_n^2)(\omega_{n1}^2 - \omega_n^2)(\omega_{n2}^2 - \omega_n^2)}{\omega_z^2 (\omega_x^2 - \omega_n^2)} \quad (3.12)$$

where $\omega_z, \omega_{n1}, \omega_{n2}$ are given by Eqs. 3.1a and 3.7, e_x is the eccentricity of the centroid of base area of foundation measured along the x axis from the centre of gravity of machine foundation and $\alpha = \varphi_y / m$ (3.13)

FOUNDATIONS FOR RECIPROCATING MACHINES

Foundations for reciprocating machinery are generally of block type with openings provided where necessary for functional reasons. The outline dimensions of the foundation are generally specified by the machine manufacturer. Otherwise the following guidelines may be used to choose the size of foundation block.

(a) The size of foundation block (in plan) should be 150mm larger (all round) than the machine base..
(b) The depth is fixed to ensure stability against rocking vibrations.
(c) The combined centre of gravity of machine and foundation must be well below the top of foundation.
(d) The eccentricity shall not exceed 5 per cent of the least dimension in plan.

The data to be supplied by the machine manufactures include the following:
(a) Normal speed & power of engine
(b) Magnitude and position of static loads of the machine and foundations.
(c) Magnitude and position of dynamic loads which occur during the operation of machine. Alternatively, the manufacturer should supply all necessary data for the computation of exciting forces.

In addition to the above machine data, the necessary soil characteristics must be made available to the designer.

The principal design criteria for foundations subjected to periodical forces are as follows:
(a) The natural frequency should be at least 30 per cent away from the operating frequency of the machine.
(b) The amplitude of foundation should not exceed 0.2 mm.
(c) The stress on soil (or other elastic layers) under the combined influence of static and dynamic loads should be within the respective values

The minimum possible dimensions of the foundation should be selected satisfying the above criteria.

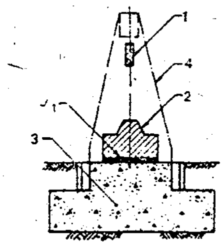
FOUNDATIONS FOR IMPACT-TYPE MACHINES

In Machines such as hammers (see Fig 9) a tup strikes repeatedly on an anvil to forge materials into desired shapes. The impact force is so large that there will be soil failure unless an elastic pad is interspaced between the anvil and foundation block. This elastic pad absorbs the energy produced during impact of tup on the anvil.

The anvil,pad,foundation and soil constitute a two degree freedom system which vibrates freely. So the design criteria is reduced to two conditions shown below.

(a) Maximum amplitudes of vibration of foundation & anvil should be within permissible limits.
(b) Stresses in elastic pad and soil should be less than the permissible limits for the respective materials.

Trial dimensions for the foundation block are assumed and the check for the design criteria. Therefore the design is a trial and error process.



(1) Tup
(2) Anvil
(3) Foundation
(4) Frame

Fig.9 Typical Hammer Foundation

APPENDIX

Table 1:Relations for a Single-Degree of Freedom System

	Constant force excitation (P_0=constant)	Rotating mass type excitation ($P_0=m_e e \omega_m^2$)
Resonant frequency	$f_n \sqrt{1-2\zeta^2}$	$f_n \frac{1}{\sqrt{1-2\zeta^2}}$
Amplitude at frequency f	$\frac{P_0}{K}\left[\frac{1}{(1-\eta^2)^2+(2\eta\zeta)^2}\right]^{\frac{1}{2}}$	$\frac{m_e e}{m}\eta^2\left[\frac{1}{(1-\eta^2)^2+(2\eta\zeta)^2}\right]^{\frac{1}{2}}$
Maximum amplitude of vibration	$\frac{P_0}{K}\frac{1}{2\zeta}\frac{1}{\sqrt{(1-\zeta^2)}}$	$\frac{m_e e}{m}\frac{1}{2\zeta\sqrt{1-\zeta^2}}$

where

Undamped natural frequency $(f_n) = \frac{1}{2\pi}\sqrt{\frac{K}{m}}$

Damping ratio $(\zeta) = C/C_c$

Critical damping $(C_c) = 2\sqrt{Km}$

REFERENCES

BARKAN, D.D.(1962) Dynamics of Bases and Foundations. McGraw Hill Book Co., New York

PRAKASH,S,RANJAN,G and SARAN,S.(1979) Analysis and Design of Foundations and Retaining Walls. Sarita Prakashan, New Delhi.

RICHART.F.E.JR., HALL,J.R.JR and WOODS,R.D. (1970),Vibrations of soils and foundations Prentice Hall, New Jersy, USA

SIRINIVASULU,P. and VAIDYANATHAN,C.V. (1976) - Handbook of Machine Foundations.Tata McGraw - Hill Publishing Co. Ltd., New Delhi.

IS2974 (Part I - 1964) Code of Practice for Design and Construction of Machine foundations for Reciprocating engines.Indian Standards Institution.

DESIGN OF SHALLOW FOUNDATIONS AND CASE STUDIES

P. NAGANATHAR

Department of Buildings

1. *Definition of Foundation*

   It is the part of the structure in direct contact with the ground and which transmits the load of the structure to the ground.

2. *Criteria for Design of Foundations*

   2.1 Foundations have to be in equilibrium between the loads and bending moments from the structure and the soil contact pressures developed. Also the deformations of foundations have to be within the tolerable limits.

   2.2 Soil supporting foundations should have adequate factor of safety against soil shear failure.

   2.3 Acceptance level of excessive total and differential settlements as well as tilt and distresses have to be established. These depend on the structural behaviour as well as the functional and architectural requirements.

3. *Data needed for design of foundations*

   3.1 Loads and bending moments and their locations at foundation level due to dead loads, imposed loads, wind loads etc. have to be estimated.

   3.2 Soil conditions and ground water conditions and soil profiles and parameters up to a depth of soil affected due to foundations have to be obtained by carrying out field and laboratory soil investigations.

   3.3 Limitations in adopting any specific methods of constructions and possible major construction problems encountered in excavation, shoring, dewatering etc. have to be established.

4. *Importance of Soil Investigations*

   4.1 Soil condition and properties and the ground water conditions under and around the foundation are essential to design safe, most suitable type and economical foundations.

   4.2 Informations such as -
   - *(a) general topography*
   - *(b) previous history and use*
   - *(c) general geology*
   - *(d) locations of buried services*
   - *(e) climatic factors (flooding, swelling and strinkage or soil erosion).*
   - *(f) Records of soil strata and ground water conditions within the zones affected by the foundations.*
   - *(g) Results of field and laboratory tests on soil appropriate to design and construction of foundations are generally required.*

   4.3 The soil investigations may range in scope from the simple examination of the surface soil (with trial pits) to the detailed study of soil and ground water condition up to the required depths by means of boreholes as well as in situ and laboratory tests.

   4.4 The extent of soil investigation work depends on the following aspects.
   - *(a) magnitude and importance of the structure*
   - *(b) complexibility of the soil*
   - *(c) type and arrangement of foundation to be selected*
   - *(d) availability of knowledge on behaviour of structures on similar soils.*

5. *Types of shallow Foundations*

   5.1A foundation is usually called shallow if the depth of embedement is less or equal to twice the width of the foundation.

   5.2 Shallow foundations may be classified into the following types.
   - *(a) Pad or Individual Foundations*
   - *(b) Strip Foundations*
   - *(c) Raft Foundations*
   - *(d) Special Foundations such as veirendeel Girder system Foundation, Floating Foundation etc.*

   5.3 Pad Foundations
   This is the simplest type of foundations provided that soil bearing pressures are safe and the settlements are within the tolerable limits. These may consist of circular, square or rectangular slab of uniform thickness or may be stepped or haunched to distribute the loads from heavy columns.

   5.4 Strip Foundations
   This is normally provided for walls and for rows of columns which are spaced so closely and pad foundations nearly overlap.

   5.5 Raft Foundation

   These are required on soils of low bearing capacity and/or where structural columns are so close in both directions that individual pad foundation nearly overlap. Raft foundations are useful in reducing differential settlements.

5.6 *Veirendeel Girder System Foundations*

This foundation system consists of reinforced concrete strip footing and plinth beam connected by vertical reinforced concrete ties at suitable intervals with pannels filled with rubble wall or block/brick wall. The considerable rigidity of the deep beam helps to bridge over any weak pockets of soft soils and also to widthstand large orders of differential settlement without causing serious distresses to the structure,

5.7 *Floating Foundation (Buoyancy Foundations)*

This is a raft foundation and the buoyancy action is achieved by providing a hollow sub structure, such as basement, to a depth so that the weight of soil removed is either nearly equal or little less than the combined weight of the super and sub structures. Adequate care should be taken to prevent heaving of soil and uplift of foundation during construction by considering in design of foundations as well as planning out constructure proceedures and sequences.

6. *Mode of Failure of soil due to foundations*

6.1 *Modes of failures of soil under and around foundations could be classified into three catergories.*
(a) *General shear Failure*
(b) *Local shear Failure*
(c) *Punching shear Failure*

6.2 *General shear Failure*
Large heaving of surface soil around the foundation occurs and the failure of soil is governed by the ultimate bearing capacity of the soil. In dense and stiff soil general shear failure take place.

6.3 *Local shear Failure*

Small heaving of surface soil around the foundation occurs and the failure of soil is governed by the ultimate bearing capacity and/or settlement.

6.4 *Punching Shear Failure*
No heaving of surface soil around the foundations occurs and the failure of soil is governed by excessive settlement. In loose and soft soil punching shear failures take place,

7. *Settlements of Foundations*

7.1 *Soil supporting are subjected to settlements due to the loads transmitted from the structures.*

7.2 *Settlements of soils supporting foundation may be classified into following.*

(a) *Immediate or Elastic settlement*
(b) *Consolidation settlement (i) primary*
(ii) secondary

7.3 *The total settlements, differential settlements, and the rate of settlement as well as the tilt of the structures are more important in clayey soils.*

7.4 *Differential settlements are influenced by the following factors,*
(a) *Variation in soil strata.*
(b) *Variation in foundation loads*
(c) *Sequence of Construction of different (adjacent) parts of the structure,*
(d) *Variation in ground water condition and surcharge condition.*

7.5 *The following methods may be adopted either to avoid or to accommodate excessive differential settlements.*

(a) *Provision of rigid raft with large thickness or with beams in two right angled directions.*
(b) *Provision of deep basements*
(c) *Transfer loads to less compressible soil below by deep foundations.*
(d) *Foundations may be designed for equal settlements.*
(e) *Provision of simply supported connections (facility to jack up the upper floor beams relative to columns subsequently if necessary).*
(f) *Adjust the locations the loads of structure (modifying architectural detailing).*

8. *Soil Contact Pressure*

8.1 *The soil contact pressures due to loads on foundations are usually highly complex. These depend on -*

(a) *Nature of Foundation (Flexible or rigid)*
(b) *Type and Properties of soil (cohessive or noncohessive).*
(c) *Depth of embedment of Foundation into the ground.*

8.2 *In designing rigid foundations, the soil contact pressure is assumed to be uniform if the load is concretric and to be uniformly varying if the load is eccentric.*

8.3 *In designing flexible foundations, the soil contact pressure at any location is assumed to be proportional to the settlement at that location.*

Soil contact pressure = (soil sub grade Reaction) (Ks)x (settlement) (δ)
where soil sub grade reaction is constant for a particular soil condition.

8.4 *In practice, large variation between theory and practice may be allowed due to the following reasons.*

(a) *Foundations are neither absolutely rigid nor flexible.*

(b) *More embedment of foundation into the ground makes soil contact pressure more uniform.*
(c) *Settlements and bearing capacity for foundations are more governed by stress distribution in soil zone below the base.*

9. *Design Procedures*

9.1 *The loads and bending moments on foundations from the structure are estimated. Preliminary design of suitable types of foundations (if applicable) are to be tried and cost comparision is to be done,*

9.2 *Properties of soil of different strata, ground water conditions and the recommendations given by the soil investigation have to be studied and clarified if necessary.*

9.3 *Depths of foundations are to be decided in addition to be bearing capacity requirements depending on the actual depth of sub structure such as basement and also to avoid the effects due to the seasonal moisture and temperature variations and erosion of soil.*

9.4 The types and dimensions of suitable foundation s are to be worked out. If the soil at deeper layer provides greater bearing capacity the feasibility of design foundation at this depth are to be studied considering the practical construction difficulties and the additional cost and time involved in shoring and dewatering of foundation excavations.

9.5 The structural adequacy of foundation structures are to be established at the ultimate limit state and also at the serviceability limit state.

9.6 The total and differential settlements as well as the tilt of the structure are to be estimated and verified that they are within the acceptable limits to suit the structural, functional and architectural requirements.

10. Foundation on difficult soil condition.

Design of shallow foundation on filled up low lying land, expansive soil and steep slope hill are considered.

10.1 Foundation of Fill up Low lying land.

(a) The important factors involved in design of foundation are the properties of the fill material and of the original sub soil as well as the degree of control of filling procedures.

(b) When the fill is controlled, it may be possible to place the foundations of light or low rise structures within the fill provided that the soil pressure developed in the fill and in the original sub soil as well as the total and differential settlements are within the allowable limits. Also structural joints may be provided in order to accommodate settlements without causing distress to the structures within each unit.

(c) If the fill is uncontrolled, the foundations may be placed either on the well compacted hard soil fill replacing the uncontrolled fill and or original sub soil, Depth and width of compacted fill are to be decided depending on the properties and compaction state of the soil below and around the fill.

10.2 Foundation on Expansive soil.

(a) Strink-swell characteristics in the expansive soil, due to variation in moisture content, develope swell pressure to the foundations and distresses to the structures.

(b) One of the following methods is generally adopted to avoid distress in the structure.

(i) Extending foundations into the level where no moisture variation exists.

(ii) Replace expansive soil of satisfactory depth with non expansive well compact hard soil to support the foundations.

(iii) Raft or strip foundations are adopted to resist the swell pressures.

10.3 Foundation on steep sloped hills.

(a) Stability of slopes are to be ensured against the disturbance caused to the equilibrium by excavations, additions loads provided by new structures and the excess pore water pressure caused by reducing or preventing natural drainage system.

(b) Depth of adjacent foundation are to be decided so that overstressing could be avoided due to the loads from the adjacent foundations.

(c) The surface water and rain water drainage systems are to be satisfactorily designed, constructed and maintained permanently around and under the foundations in order to avoid any earth slip due to inadequate drainage facilities.

11. Design of Foundations - Case studies.

11.1 Locations, soil profile classifications, and basic strength parameters, recommended types of foundations, adopted type of foundation, reasons for variation in foundation and the practical difficulties encountered are included in the case studies.

11.2 The following foundations structures are considered for case studies.

(a) Four storeyed building with Floating Foundation at Colombo 7.
(b) Three storeyed building with raft foundations on well compacted fill soil at Ratmalana.
(c) Two storeyed buildings with pad foundations with continuous plinth beams at Galle.
(d) Two storeyed buildings with strip and pad foundations with ground continuous beams at Kalmunai.
(e) Single storeyed building with veirendeel girder system foundations on well compacted gravel fill at Kotte.
(f) Single storeyed building with veirendeel girder system foundations on well compacted river sand at Piliyandala.
(g) Five storeyed building with pad foundations and plinth beams on improved soil at Colombo 7.
(h) Two storeyed building with pad foundations and ground tie beams on steep sloped hill at Kandy.
(i) Single and two storeyed buildings with veirendeel girder system of foundation (on well compacted river sand for single storeyed building only)at Minuwangoda.

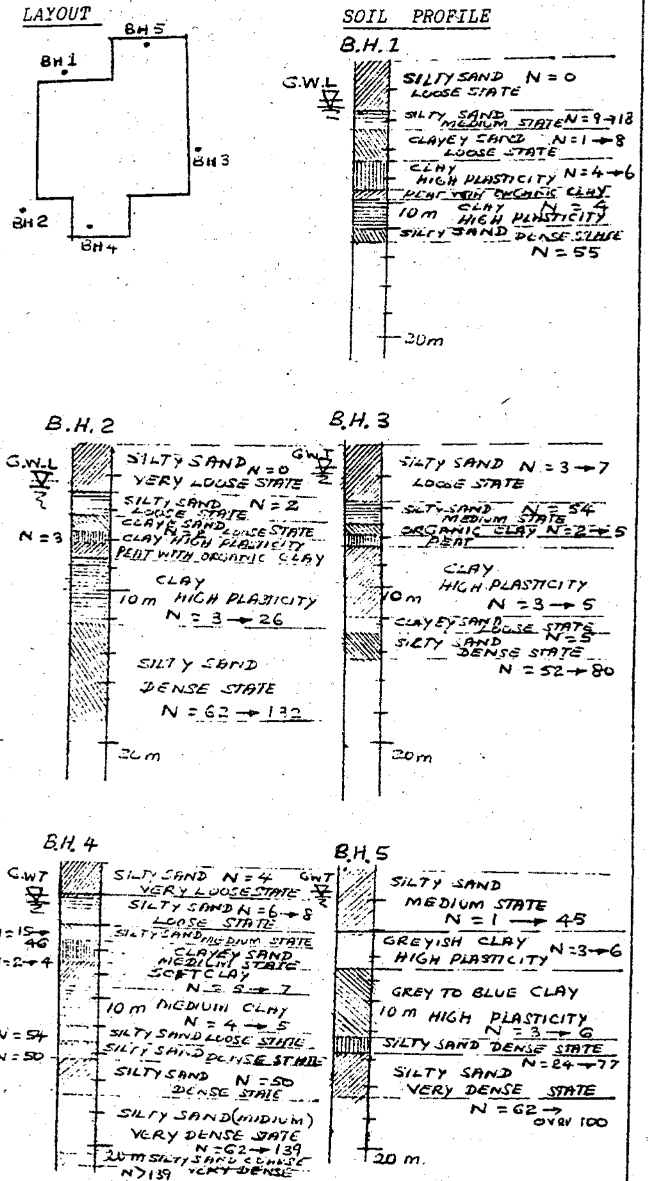
Acknowledgement
The writer is grateful to the Director Buildings for granting permission to present the case studies.

DEPARTMENT OF BUILDINGS

DESIGN OF SHALLOW FOUNDATIONS - CASE STUDY (1)

FOUR STOREYED BUILDING AT COLOMBO 7

LAYOUT SOIL PROFILE



RECOMMENDED FOUNDATIONS

(a) Raft Foundation
Soil Bearing capacity = $40 kN/m^2$
Depth = 2.5m

(b) Pile Foundation
in Dense sand

ADOPTED FOUNDATION

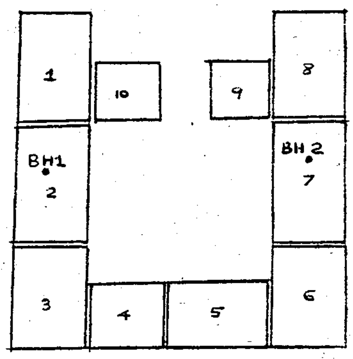
Raft Foundation as basement "Floating Foundation" at 3.0m depth.

DEPARTMENT OF BUILDINGS

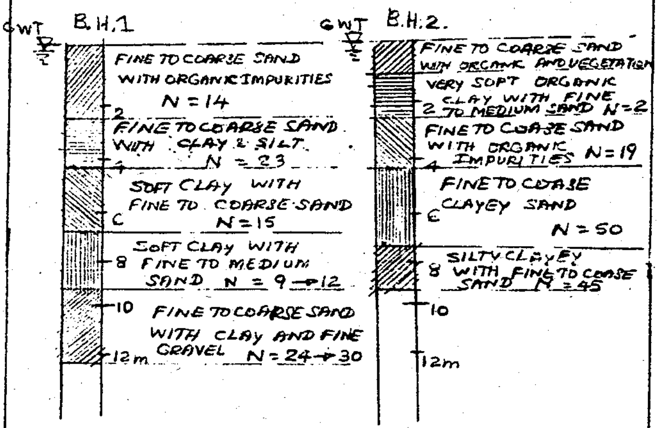
DESIGN OF SHALLOW FOUNDATIONS - CASE STUDY (2)

THREE STOREYED BUILDING AT RATMALANA

LAYOUT



SOIL PROFILE



RECOMMENDED FOUNDATION

(a) Pile or cylinder at 3.0m depth

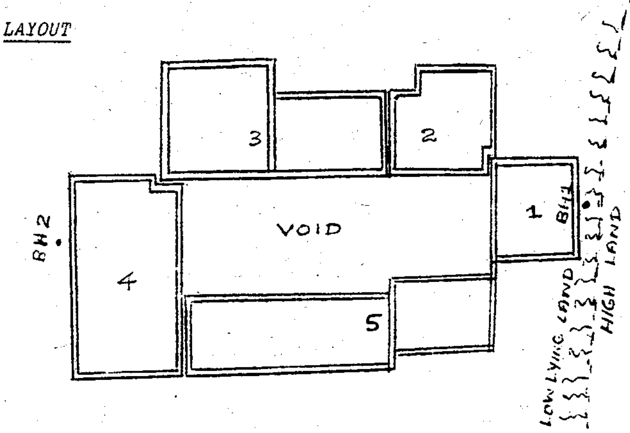
(b) Raft on well compacted fill at 1.75m above ground level

FOUNDATION ADOPTED

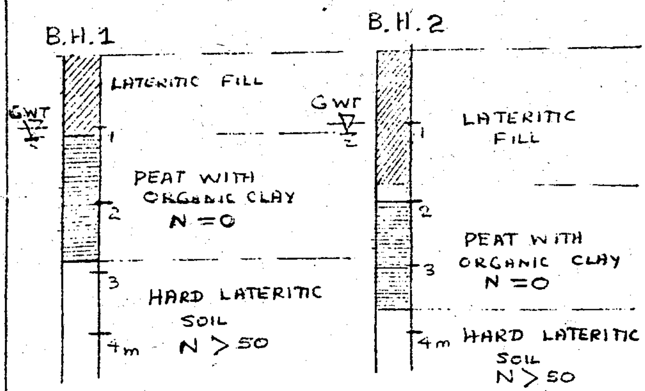
Raft Foundation at ground floor level at 1.75m above ground level with nine structural joints (Ten different structural units).
1.2m deep R.C.C. vertical protective walls at end of Raft monolithic with raft provided along all peripheries including structural joints for each unit.
Also 3.0m wide well compacted fill maintained around the building with Toe Wall.

DEPARTMENT OF BUILDINGS
DESIGN OF SHALLOW FOUNDATIONS - CASE STUDY (3)
SINGLE STOREYED BUILDING AT KOTTE

LAYOUT



SOIL PROFILE



RECOMMENDED FOUNDATION

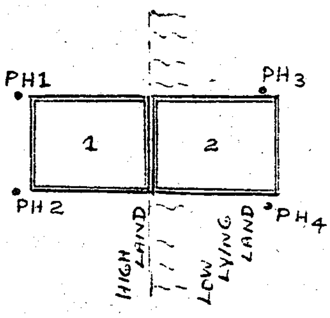
Veirendeel Girder System Foundation

ADOPTED FOUNDATION

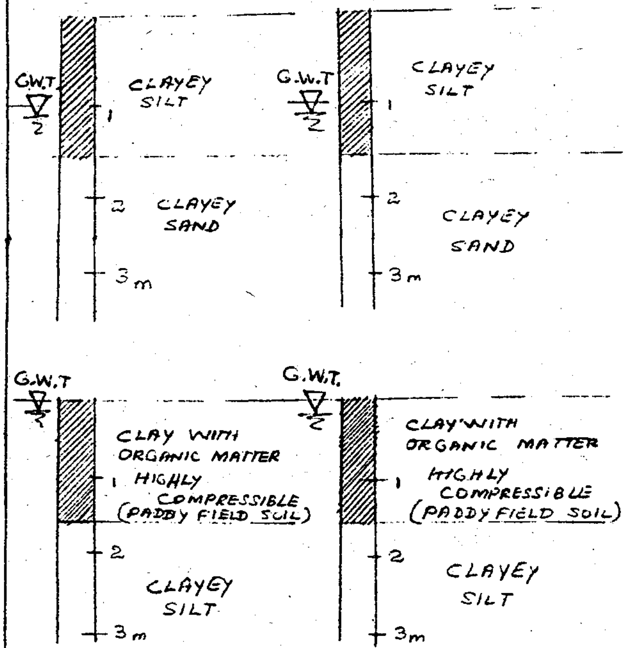
Reinforced concrete and Rubble work veirendeel Girder System Foundations (Overall depth = 750mm) with five structural units constructed on well compacted gravel fill.

DEPARTMENT OF BUILDINGS
DESIGN OF SHALLOW FOUNDATION - CASE STUDY (4)
SINGLE STOREYED BUILDING AT PILIYANDALE

LAYOUT



SOIL PROFILE
(PRELIMINARY SOIL INVESTIGATION)



RECOMMENDED FOUNDATION

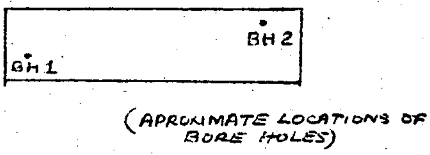
(a) Strip Foundation on well compacted fill
(b) Veirendeel Girder System Foundation (Reinforced concrete and Rubble work) With one structural joint.

ADOPTED FOUNDATION

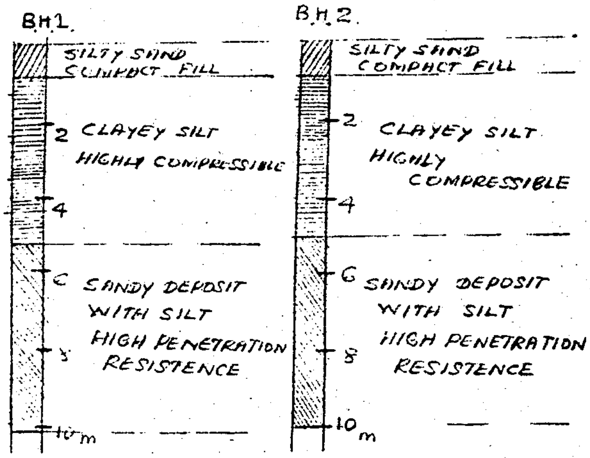
Reinforced concrete and Rubble work Veirendeel Girder System Foundation (Two structural units) on well compacted replaced fill. Overall depth of Foundation = 750mm.

DEPARTMENT OF BUILDINGS
DESIGN OF SHALLOW FOUNDATIONS - CASE STUDY (5)
FIVE STOREYED BUILDING AT COLOMBO 7

LAYOUT



SOIL PROFILE



RECOMMENDED FOUNDATION

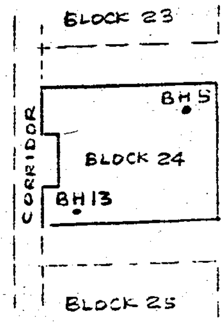
Pad Foundation
(Width not exceeding 3.5m)
Soil bearing capacity = 115kN/m²
Depth of foundation = 1.5m

ADOPTED FOUNDATION

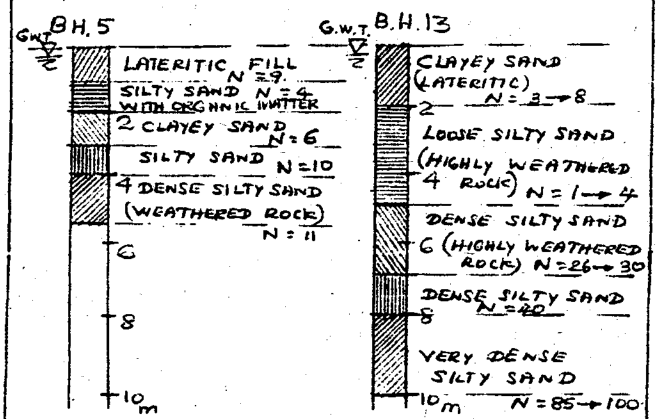
Pad and connected foundations
(Width = 3.5 m, rectangular shape)
Founded on 150 mm → 225 mm thick well
Compacted river sand fill at 1.5m depth.
Plinth level beams provided.

DEPARTMENT OF BUILDINGS
DESIGN OF SHALLOW FOUNDATIONS - CASE STUDY (6)
TWO STOREYED BUILDING AT GALLE

LAYOUT



SOIL PROFILE



RECOMENDED FOUNDATIONS

(a) Pile foundation
8 → 10 m depth

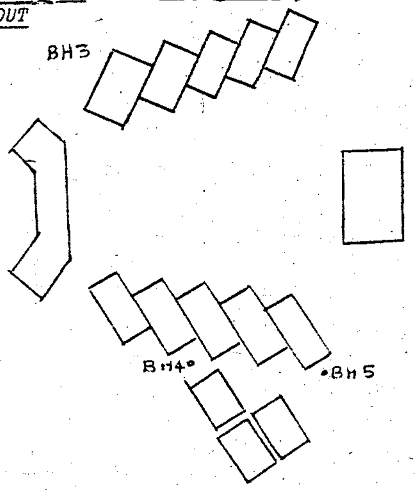
(b) Strip footing
Soil bearing capacity = 50kN/m²
at 1.5m depth

(c) Veirendeel Girder System Foundation

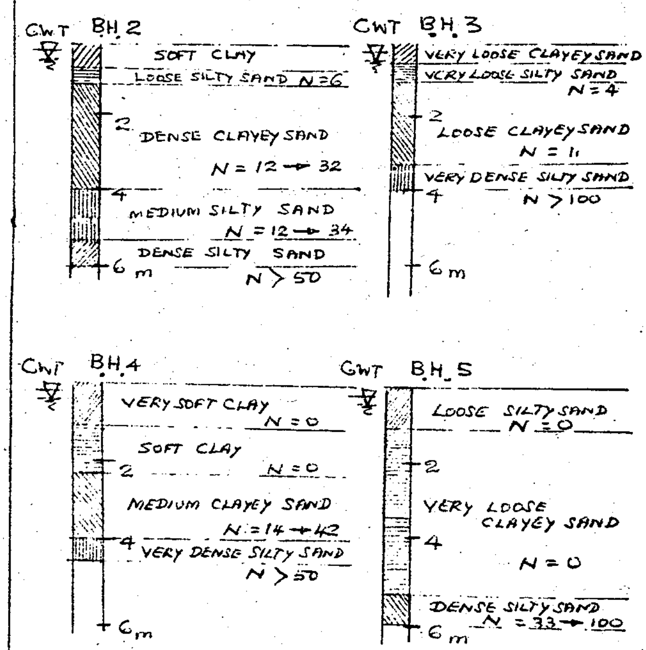
ADOPTED FOUNDATION

Pad and connected foundations with plinth level beams, with improved soil condition.

DEPARTMENT OF BUILDINGS
DESIGN OF SHALLOW FOUNDATIONS - CASE STUDY (7)
TWO STOREYED BUILDINGS AT KALMUNAI
LAYOUT



SOIL PROFILE



RECOMENDED FOUNDATION

Pad Foundations
Soil bearing capacity = 200kN/m²
Depth = 2m

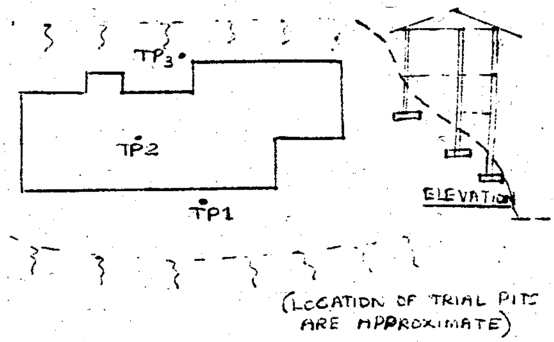
ADOPTED FOUNDATIONS

(a) Inverted Tee Beam Strip Foundation
(At two right angles)

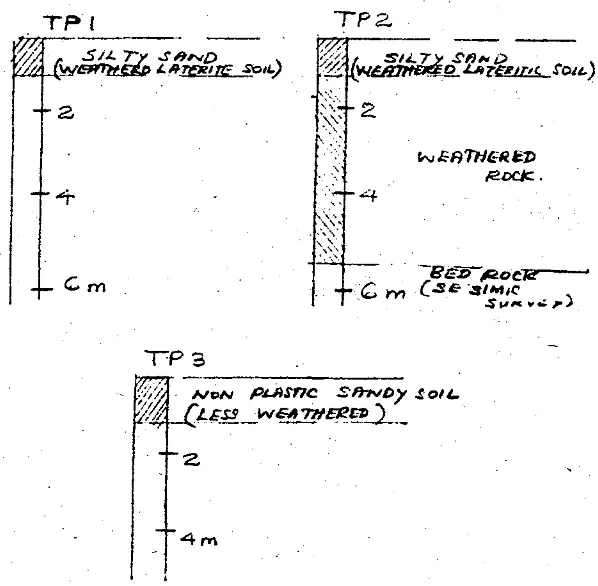
(b) Pad Footings
Also beams constructed at original ground level and at plinth level.

DEPARTMENT OF BUILDINGS
DESIGN OF SHALLOW FOUNDATIONS - CASE STUDY (8)
TWO STOREYED BUILDING AT KANDY

LAYOUT



SOIL PROFILE



RECOMMENDED FOUNDATION

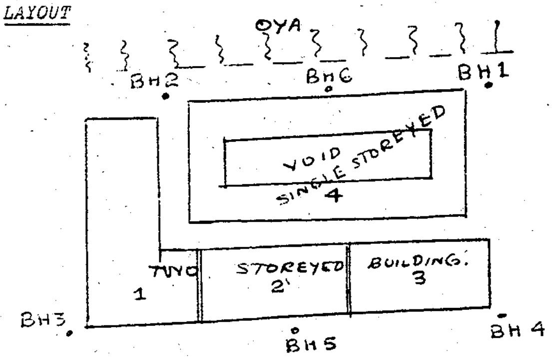
Pad foundations
Soil bearing capacity = 200 kN/m²
Depth = 1.0m
Proper surface water and rain water drainage system to be provided.

ADOPTED FOUNDATION

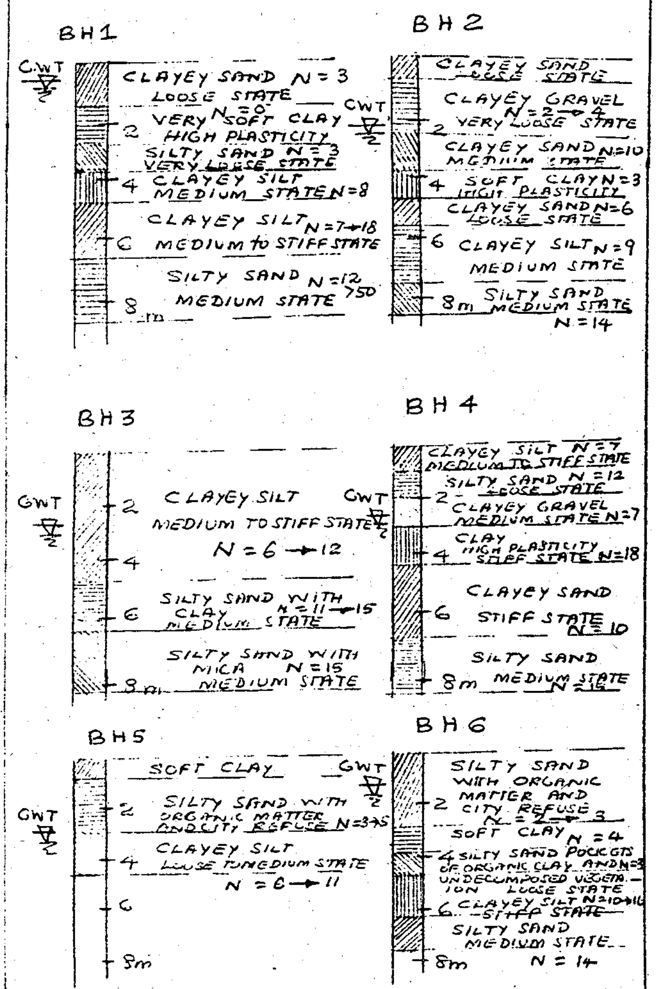
Pad foundations
At depth 1.0m → 1.5m
Ground level beams provided.
Proper surface and rain water drainage system provided.

DEPARTMENT OF BUILDINGS
DESIGN OF SHALLOW FOUNDATIONS - CASE STUDY (9)
SINGLE STOREYED AND TWO STOREYED BUILDINGS AT MINUWANGODA

LAYOUT



SOIL PROFILE



RECOMMENDED FOUNDATIONS

(a) SINGLE STOREYED BUILDING

Reinforced concrete strip foundations

Soil bearing capacity = 30kN/m²

Depth = 1.5m

(b) TWO STOREYED BUILDING

Reinforced concrete strip foundation

Soil bearing capacity = 60kN/m²

Depth = 1.5m

ADOPTED FOUNDATIONS

(a) SINGLE STOREYED BUILDING

Reinforced concrete/Rubble wall Veirendeel Girder System Foundation on one meter well compacted river sand fill (Soil between 1.5m depth to 2.5m depth replaced) at 1.5m depth.

(b) TWO STOREYED BUILDING

Reinforced concrete/Rubble wall Verendeel Girder System. Foundation with two structural joints (Three structural units) at three different levels. Due to actual variation in depths of soft and organic soil near surface.

UNIT 1

Foundation at 1.5m below ground level (≏ 1.8m below D.P.C.)

UNIT 2

Foundation at 1.5m below ground level (≏ 2.75m below D.P.C.)

UNIT 3

Foundation at 2.5m below ground level (≏ 3.6m below D.P.C.)

ACKNOWLEDGEMENTS

The writer is grateful to THE DIRECTOR of Buildings for granting permission to present the case studies.

FOUNDATIONS ON DIFFICULT GROUND CONDITIONS

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1.0 INTRODUCTION

Difficult ground conditions are generally implied when a general weakness or a high degree of variability or a combination of these two is encountered in the ground affected by the foundation pressures and/or by the construction operations.

Man had always tried to avoid difficult ground situations whenever it was possible to select an alternative site with less or practically no problems. But, when the easily buildable lands with sound ground conditions are exhausted there would be no other choice than to build on lands which had been hitherto considered unsuitable for construction purposes either due to high cost of development or due to inherent geotechnical problems. In tackling difficult ground situations, the geotechnical engineer may have to adopt either foundation treatment or ground treatment or a combination of these and in a worse case perhaps to consider revision of the design to make possible alterations in the structures to be constructed to suit the given ground conditions.

In Sri Lanka, with a rapid development programme formulated a decade ago, urbanisation has intensified, thereby generating an increased demand of the use of lands, which were hitherto considered unsuitable for development. The geotechnical engineer today, is therefore faced with the challenge of constructing on difficult grounds which may cause serious distress to structures, unless special precautions with foundation treatment or ground treatment is adopted.

The highly compressible peat and other organic deposits found in the urban and coastal low-lying lands, highly variable laterite formations found in most parts of the wet zone, expansive soils occasionally observed in some dry zone areas, the residual soils found in the hill country and the man made fills now found everywhere, often present difficult ground conditions. However, certainly very little is known about the engineering behaviour of such ground conditions.

Difficult ground situations may fall in widely varying categories, however, difficult ground may be broadly classified into:

(a) Variable ground with irregular soil profile

- (i) Laterite formations often found in the wet zone of the island consists of materials in varying stages of decomposition and in different soil composition due to varying rates of leaching of minerals.
- (ii) Residual soils and rocks with different degrees of decomposition and weathering.
- (iii) Filled up grounds where filling has been carried out under controlled or uncontrolled conditions using inorganic or organic materials.
- (iv) Irregular soil deposits that are generally associated with estuary and delta deposits and sometimes with shore deposits.
- (v) Ground with faulty structures.

(b) Grounds with inherent weakness which could be attributed to high compressibility, high void ratio or occurrence of deep extents of compressible or loose layers.

(c) Special ground conditions attributed to unique soil properties; for example expansive soils found in some parts of the dry zone.

(d) Difficult ground conditions which are attributed to environmental factors such as high ground water table.

Selection of the appropriate type of foundation and the method of construction to meet difficult ground conditions require careful assessment of subsoil characteristics based on proper investigations and laboratory testing. However, due to lack of knowledge of the correlation between predicted and actual performances of foundation and in the absence of adequate understanding on difficult ground behaviour, the foundation engineer may not always be successful in his pioneering efforts in finding solutions under such unpredictable ground conditions. While such knowledge has to be acquired through research and experience, successful attempts have been made in Sri Lanka by adopting special foundation types such as stiffened RC strip foundations, Vierendeel girder type foundations or raft foundations in situations where the conventional shallow foundations were found not satisfactory.

Where pile foundations are prohibitively costly, but where again direct foundations has to be supported in "too deep" layers, underreamed piles have enormous potential as a more appropriate choice of foundations, specially in the case of small to medium scale buildings.

2.0 FOUNDATIONS ON LATERITE FORMATIONS

Laterite soils are widely distributed over the wet zone and most of the early built-up urban lands around the capital are constructed over hillocks composed of the so called "lateritic" residual soils locally known as kabook. A significant feature of the laterite profile is the presence of a vesicular hard crust of ferricrete often seen as outcrops, followed by a hard cellular skeleton of iron oxide with clay filled cavities. This is followed by highly weathered material generally fall under classifications MH & ML but CH, CL and SM materials are not uncommon. The boundaries of distribution of these different materials in both vertical and lateral directions are not clear and the properties, both physical and mechanical, often vary widely, thus presenting a variable ground condition. It is not unusual to encounter large and isolated pockets of highly 'kaolinized" soft clays among the silty material or even to encounter highly porous, ironstone cells devoid of clays. Hence the interpretation of Standard Penetration Tests (SPT) results too need careful attention. The SPT N-values observed in laterite clays generally vary from about 5 to 10 while in somewhat harder materials average values around 15-20 could be observed. Owing to the peculiar and heterogeneous structure of this soil formation, the strength and deformation characteristics and the behaviour of lateritic soils are not well understood and may not be easily determined by the conventional testing procedures.

Light foundations for single to three storeyed buildings have been successfully supported over strong lateritic strata using conventional foundations such as strip footings of rubble or brick masonry for load bearing walls and pad footings for columns. However, heavier buildings are generally supported on wide based strip footings, stiffened footings, raft or pile foundations, when weak or variable soil conditions are encountered. Where severe variations in soil type are observed or anticipated, attention may be necessary to investigate for isolated soft or compressible clay pockets which should either be by-passed with deep foundations or be bridged over using rigid foundations.

3.0 FOUNDATIONS ON EXPANSIVE SOILS

Expansive soils occur in many parts of the world and according to their parent materials, they fall into two groups. In the first group, the feldspar and pyroxene minerals in the basic igneous rock have decomposed to form montmorillonite and other secondary minerals. In the second group montmorillonite contained in certain sedementary rocks breaks down physically to form expansive soils.

Of the three most important clay minerals; montmorillonite, illite and kaolinite, montmorillonite is the clay mineral that presents most of the expansive soil problems. These clay minerals are formed by a complicated alteration process that include disintegration, oxidation, hydration and leaching. Formation of montmorillonite minerals is aided in an environment of extreme disintegration, strong hydration and restricted leaching where magneseum,calcium, sodium and iron cations may accumulate in the system. Such conditions are favourable in semi-arid zones having low or seasonal moderate rainfall, with evaporation exceeding precipatation, where enough water is available for the alteration process but without sufficient flush rain to remove the accumulated cations.

Expansive soils do exist in Sri Lanka and these have been identified in various parts of the dry zone; in (Murunkan) Mannar, Anuradhapura, Puttalam, Dambulla, and Kataragama, when subsoil investigations were carried out for specific development projects in these areas. However, the parent rocks and the process of alteration etc. are yet to be understood and research need to be initiated in this direction.

Buildings, especially those with lighter loadings, founded on expansive soils could undergo severe distress due to uplift forces caused by the swelling of soils unless special precautions are taken in the design, construction and also in maintenance. Cracks on floor slabs due to heaving, diagonal cracks that develop below windows and above doors are a strong indication of swelling movement. However, such distress should not be unduly blamed on expansive soils as similar symptoms could be observed under other circumstances too.

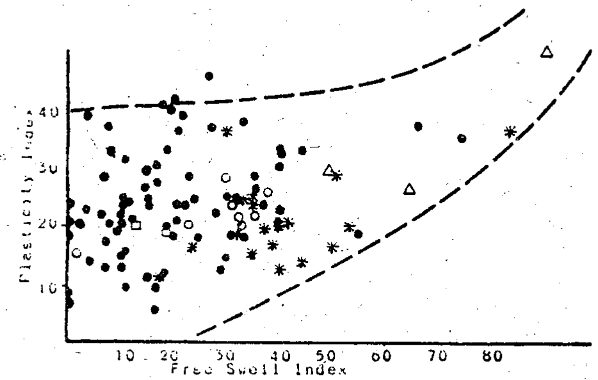
For the recognition of potentially expansive soils, three different methods of classification are available. The first, mineralogical identification can be done using techniques such as X-ray Defraction, Differential Thermal Analysis, Dye Absorption, Chemical Analysis or Electron Microscope Resolution. These methods would be useful in the evaluation of the material and are important in exploring basic properties of clays. However, this is not sufficient in dealing with natural soils and moreover, is impractical and uneconomical. In the second, or in the indirect method, Index Properties, Potential Volume Change (PVC), Activity, Soil Suction are valuable tools in evaluating swelling properties (Chen, 1975). However, none of these should be used independently without direct tests in order to avoid errorneous conclusions.

The third method or the direct measurements of Swelling Pressure offer most useful data for a practising engineer and these tests can be easily performed in the laboratory using a conventional one-dimensional consolidometer.

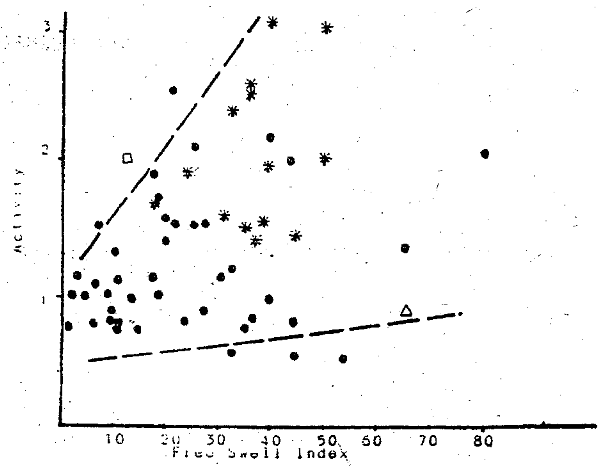
In the field it may not be an easy task to identify expansive soils. However, soils containing a high content of high plastic clays and in a very dry state with a moisture content below 15%, possessing high dry strength and high dry densities in excess of 1.75 gm/cm^3 are suspected to have expansive properties and therefore should be subjected to confirmative tests. Field penetration resistance exceeding SPT N-value of 15 is often associated with expansive soils.

One practical method of identifying expansive soils in a suspected area is to examine the actual behaviour of existing light structures which had experienced a few dry and wet cycles. Observation of cracks in walls, sidewalks, floors, etc., shrinkage cracks appearing in the soil during the dry season, high dry strength of soil lumps and high stickiness of soil when wet etc. would be helpful to identify expansive soils.

In the laboratory, testing for Free Swell Index (i.e. the difference between final and initial volume, expressed as a percentage of initial volume, when a known volume of dry soil is allowed to swell in water inside a graduated cylinder for 24 hours without a surcharge) is a very crude yet an useful guide before proceeding with classification based on parameters such as Colloid Content, ($<$ 0.001 mm.), Clay Content ($<$ 0.002 mm.), Plasticity Index which are considered to offer more reliable data. Results of laboratory tests conducted for swelling properties of some soils in the dry zone of Sri Lanka are shown in Fig. 1.



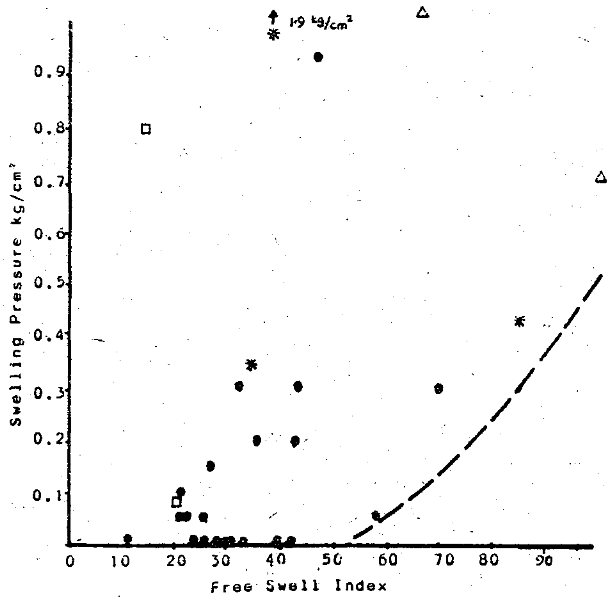
VARIATION OF PLASTICITY WITH FSI



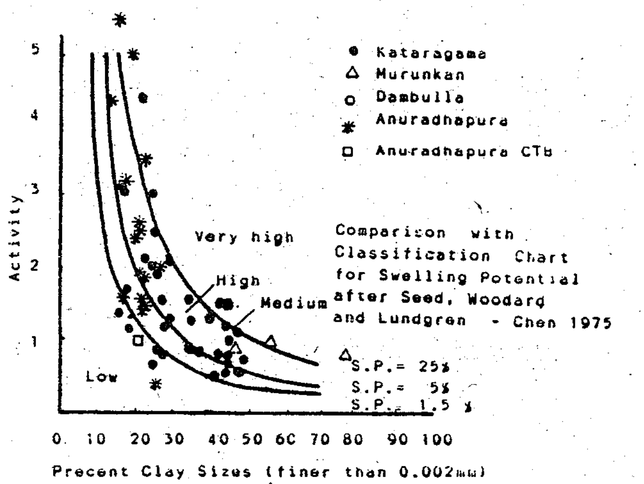
VARIATION OF ACTIVITY (PI/(C-10)) WITH FSI

Table –. *Data for making estimates of probable volume changes for expansive soils*

Laboratory and field data			Probable expansion, percent total volume change	Swelling pressure, ksf	Degree of expansion
Percentage passing No. 200 sieve	Liquid limit, percent	Standard penetration resistance, blows/ft			
>95	>60	>30	>10	>20	Very high
60-95	40-60	20-30	3-10	5-20	High
30-60	30-40	10-20	1-5	3-5	Medium
<30	<30	<30	< 1	1	low



VARIATION OF SWELLING PRESSURE WITH FSI



VARIATION OF ACTIVITY WITH CLAY CONTENT

FIG 1. SOME PROPERTIES OF EXPANSIVE SOILS IN SRI LANKA

Slight changes of moisture content in expansive soils are sufficient to cause detrimental swelling and therefore stable, uniform soil moisture conditions should be ensured beneath and around the foundation. To achieve this, it would be necessary to prevent ingress of water, by proper guidance of surface drainage away from the structure with the provision of moisture barriers with cut off walls around the building, by exclusion of deep roots intruding into building area, and by taking adequate precautions to protect against undetected leakage from underground piping or backup through poorly backfilled trenches, etc. Where practical, the foundations may be taken to a depth sufficiently below the perenial ground water table so that moisture variation does not cause any further swelling of the soil that supports the structure. However, maintaining constant soil moisture conditions seems to be very difficult in practice, at least in the case of dwellings in this country. In such cases, replacement of expansive soil with granular material works as a satisfactory solution in cushioning out or nullifying the swelling effects.

Expansive soils being stiff, offers high ground bearing capacity adequate for most lightly to moderately loaded structures, However, swelling pressures may often exceed the bearing pressures, resulting in an upward movement of the structure. In such circumstances, shallow foundations may be designed to exert high bearing pressures which could withstand the swelling pressures, but at the same time ensuring rigidity of the foundation.

Special waffle or raft foundations (Fig 2) which can act as a unit to minimize differential action due to swelling may be considered where shallow spread footings or slab-on-grade construction are required. Shallow foundations may also be adopted after stabilizing the soil with cement/lime or chemicals.

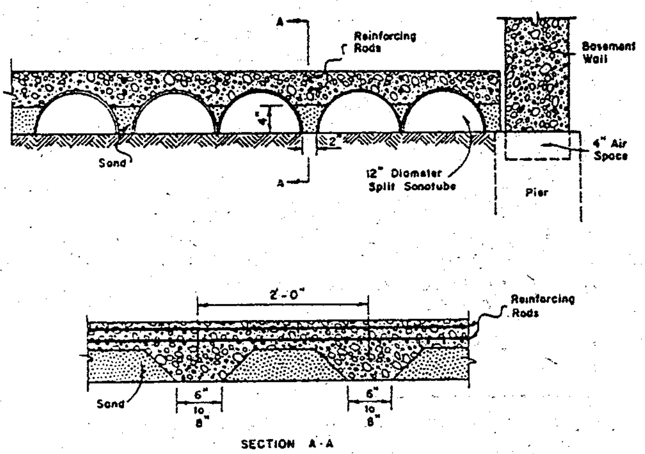


FIG 2. SLAB FOUNDATION WITH HONEYCOMB STRUCTURE

If the expansive soil layer is not very thick, the foundations may be supported on a deeper, non-expansive soil layer with adequate care taken against the swelling forces acting on floor slabs, for example, by designing them as suspended floors. Where the expansive soils extend to greater depths, underreamed-piles (Fig 3) which can resist upward swelling forces can be adopted (CBRI, 1978). The underream should be anchored in a non-expansive layer or below the perenial ground water table in a zone of no-moisture change.

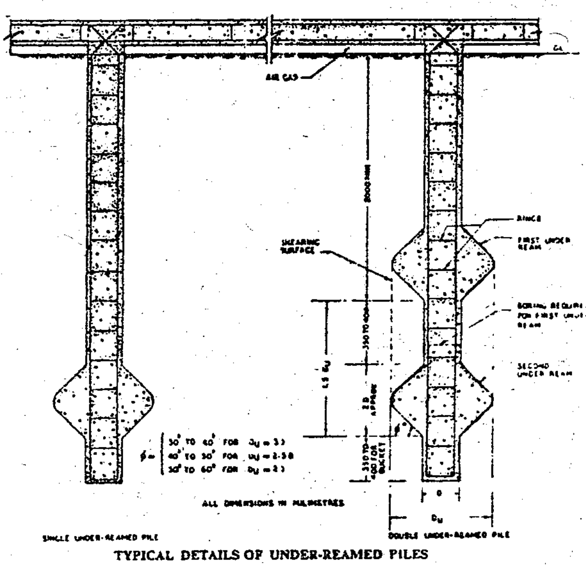


FIG 3. SINGLE AND DOUBLE BULBED UNDER-REAMED PILES

4.0 FOUNDATIONS ON FILLS

A fill may be either a controlled fill or an uncontrolled fill and may consist of norganic or organic materials or of a mixture of the two. Compaction Controlled filling may be used beneath structures for one or more of the following purposes;

i) to raise the general grade of the structure
ii) to replace unsuitable foundation soil that must be removed
iii) to provide a relatively stiff mat over soft subsoils to spread bearing pressure from foundation loads and to decrease differential settlement
iv) to bridge over subsoil with hard and soft spots or small cavities in erratic profiles
v) as preloading to accelerate subsoil consolidation and eliminate all or part of settlement of the completed structure.

Rigidity, strength and homogeneity of a fill can be largely increased by controlled compaction with appropriate equipment and properly selected material. Foundations resting on controlled fills can be designed in a similar manner as in the case of foundations supported on natural soils. Foundations supported on filled ground may undergo settlements due to;

i) Consolidation of compressible fill under foundation loads transmitted

ii) Consolidation of the fill under its own weight and

iii) Consolidation of natural ground beneath the fill under the combined loads from the fill and the structure and also under the weight of soil in the natural ground.

If the fill had been recently placed, compressible subsoils may still be in a sub-consolidated condition and may undergo settlement under the fill load with time and this fact should be considered in the design.

For lightly loaded structures, settlement caused by the foundation load and the own weight of fill in a properly compacted fill would generally be insignificant and may not pose problems to the engineer. Settlement due to consolidation of the fill under its own weight depends on the depth and the degree of compaction of the fill layer and the conditions under which it had been placed (Table 1).

Unfortunately in most of the filled lands in and around Colombo, except where special attention had been given under the guidance of an engineer, filling had not been done with any compaction control whatsoever. Often, things are made more difficult to the engineer where soils, construction debris and refuse, etc. had been dumped and dressed with a thin layer of "acceptable" earth material which is either simply deposited or compacted before the "developed" land is disposed to a client.

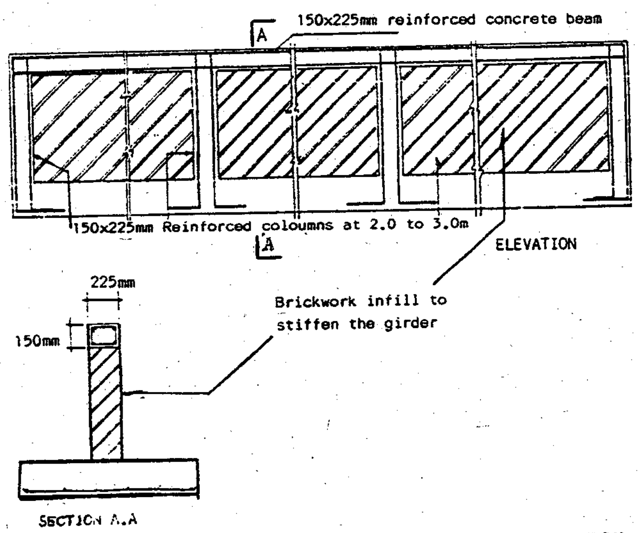


FIG 4. VIERENDEEL GIRDER SYSTEM OF FOUNDATION

Table 1. Total settlement of different fill materials under their own weight.

Type of fill		Total Settlement under its own weight expressed as % of the fill thickness.
Well compacted inorganic fill		0.5 %
Poorly compacted inorganic fill	up to	1 %
Well compacted clay fills		0.5 %
Highly compacted clay fills placed in deep layers		1 % - 2 %
Domestic refuse fill (with controlled topping layer)	about	10 %

Uncontrolled fills may offer irregular profiles and may not be suitable for foundation support. Especially those containing compressible and decomposable organic matter, hard objects like construction debris, etc. with large cavities and loose pockets will exhibit complicated and erratic soil profile and offer misleading information even when subsoil investigations are carried out. In such cases, special foundation treatment would be required and it is advisable to support the foundation on a natural hard stratum using deep foundations. A raft foundation with stiffened slabs or a Vierendel girder system of foundation (Fig 4), which can take care of large order or differential settlement, may be an alternative solution to deep foundation. Footings may also be adopted over a well compacted granular fill which replaces the underlying weak soils to a depth equivalent to more than the width of the footing.

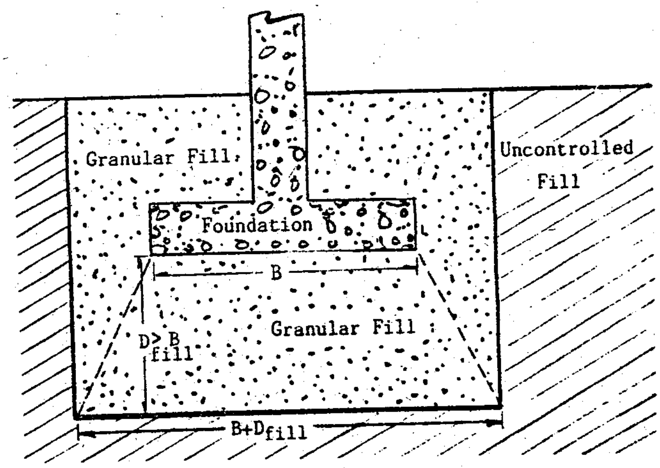


FIG 5. FOUNDATION ON COMPACTED GRANULAR FILL OVER UNCONTROLLED FILL

5.0 FOUNDATIONS ON IRREGULAR SOIL DEPOSITS

Irregular soil deposits are generally associated with lacustrine and deltaic and also sometimes with shore deposits. The continually shifting shore currents, seasonal floods, oscillation of the sea in geological times, etc. cause a wide variation in the soil properties within a short expanse.

Subsoil in the estuary and delta deposits is mostly composed of compressible fine grained sediments with mixed or sandwiched organic matter. In composite shore deposits, sand may alternate in erratic manner with organic silts and peats. Such deposits can therefore have a widely varying compressibility. In such situations sub-surface soundings supplemented by a few exploratory drill holes would be more appropriate and practical than to proceed with a thorough investigation involving undisturbed samples which could lead to contradictory and confusing information. Foundations may be designed to withstand the maximum differential settlement between the hardest and softest ground conditions met at the site that is roughly estimated based on empirical relationships.

6.0 FOUNDATIONS ON HIGHLY COMPRESSIBLE SOILS

Difficult ground conditions are faced with where the ground has inherent weakness due to high compresibility, high void ratio and occurrence of deep extents of compressible layers. High compressibility may be attributed to the presence of peat, organic clay or soft inorganic clay in the subsoil. Very loose silts and sandy deposits may also lead to high compressibility in the ground.

In the vast expanse of low lying areas in and around Colombo City that has been earmarked for development, a significant feature is the predominant presence of highly compressible organic deposits mainly consisting of peat, organic clay and organic silt.

Peat is generally formed by the death and partial decay of plants in the swampy areas and with the accumulation of such deposits over the years. Texturely, peats in Colombo area vary from highly permeable fibrous material with high to very high order of compressibility to a fine amorphous material which resembles a soft organic clay. Combined thickness of peat and organic clay deposits is found to vary from a few meters to nearly 15 meters (Senanayake, 1986). As such, in the process of development of low lying areas, the loads due to earth filling and structures supported on the fill can cause significant settlement in the underlying compressible layers, often excessive and intolerable in terms of both ultimate and differential settlements. Mechanism of settlement and engineering behaviour of peats in Sri Lanka need research. Settlement occurs in peat deposits due to immediate compression of peat mass with the expulsion of air and gases, gradual expulsion of water from the macro voids in the peat mass and from micro pores in fibrous peat, lateral spreading towards adjacent unloaded peat which has low lateral support, creep and of peaty soils or in an extreme case by rotational slip and upheaval of the ground.

If the peat deposit is within replaceable reach, it is always perferable to construct the foundations over a well compacted quality material which replaces the peat deposit. Very light structures may be supported on suitably stiffened reinforced strip foundations over adequately thick layer of compacted fill when the thickness of peat layer is not very large. Vierendeel girder system of foundation may be used as an alternative. To avoid cracking due to ground settlement, the floors may be suspended. Rafts with stiffened beams may be used for light structure with up to about two storeys.

A sufficiently thick well compacted fill serves as a mat in distributing the foundation loads. However, it should also be noted that often, the settlement due to fill load is significantly larger than the settlement due only to the load of structure that is to be supported over the fill. For medium to heavy structures and when the peat deposits are thick, pile foundations supported on a suitable bearing stratum need to be provided. On the other hand, when a suitable bearing stratum underlying the peat is available within easy reach, say up to about 7 m, under-reamed piles may prove to be economical.

Foundations on soft clays and peat deposits largely depend upon the order of compressibility, and the thickness of the deposits. When the deposit is not very thick; say up to about 5 m to 7 m, it is worthy to explore possibility of ground improvement by preloading as the first step. Preloading technique involves loading of the ground to pre-induce before laying of the foundation, a grater part or whole of the ultimate settlement that is anticipated to take place under the foundation load. This method is most suitable where borrow material for surcharge is economically available and if the area is large so that surcharge material can be progressively provided in bulk and moved across using earth machinery. Duration of preloading is crucial since on the one hand it should be sufficient to pre-induce the desired settlement and on the other hand it should not be too long to delay construction activities unduly. Sand drains or other type of vertical drains may be used to accelerate the preloading process. Weak ground conditions due to loose sandy soils can be remedied by ground improvement techniques such as vibroflotation, dynamic consolidation (heavy tamping) etc.

7.0 FOUNDATIONS ON HILL SLOPES

Central and south west regions of the island comprise of mountaneous terrain with hill peaks rising up in a wide range up to about 2500m above mean sea level. The hill slopes are mostly utilized for the major plantations of tea and rubber, and for the cultivation of sugar cane, tobacco and vegetables while the upper areas had once been reserved forests which have now begun to degrade. As a result of growth of habitation on the hill slopes, development activities have gradually increased to provide the necessary infra structural facilities. Provision of dwellings for the increased population and for the families dislocated each year by landslide disasters has become an urgent need.

Geotechnical conditions of the hill terrain are somewhat complicated and may not be possible to generalise over large areas. Geomorphology, geotechnical structure, degree of weathering, topography, type of formation and strength soils combined with many other factors affect the stability of hill slopes. By nature, some of the hilly areas are vulnerable to landslides and ground creep and this situation is aggravated by human interference through development and by negligence. Hence, the foundation engineer who deals with hilly areas, has to firstly consider the stability of terrain before he thinks of the stability of the structures that are to be constructed on the hill slopes. The fact that human habitation had been there for centuries suggests that ample experience is available in building technology, to be used and improved by the engineers of today.

Soil-structure (ie. foundation) interaction and structure-foundation interaction become very important in the case of buildings constructed on ground where vertical and lateral movements are anticipated. Flexible steel framed structures supported over independant footings can withstand large differential movements, while structures with load bearing stone masonary walls would show signs of warning with progressive development of cracks and distress when appreciable movements have taken place. On slopes which are creeping at a very slow rate, small units of rigid structures may be considered.

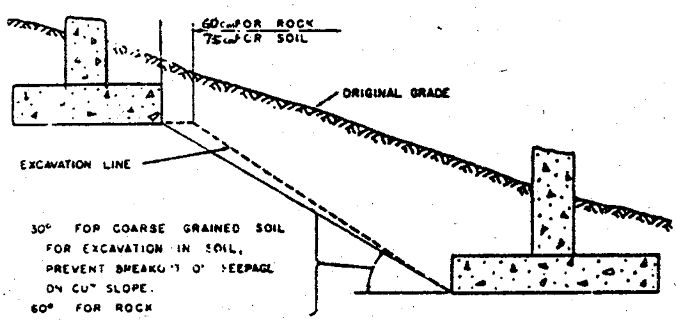


FIG 6. ADJACENT FOUNDATION ON SLOPES

For buildings on slopes it becomes often necessary to place adjacent footings at different bearing levels. In such cases, the elivations of footings shall be subjected to following limitations;

i) The sloping ground surface shall not encroach upon a frustum of bearing material below the footing (Fig. 6.)

ii) In the case of footings on granular o clayey soil, a line drawn between the lower adjacent edges of adjacent footings shall not be steeper than two horizontal to one vertical.

These shall not apply when adequate provision is made for the lateral support of the material supporting higher footing or when the factor of safety of the foundation soil against shearing is not less than four, (NAVFAC DM-7, 1971).

On hill slopes, rock or boulders may be encountered at shallow depths. Though, rocks offer relatively higher bearing capacities than soils, attention should be paid where;

i) Sound rock overlying unsound, weak or compressible materials.

ii) Planes of weakness, such as bedding planes, dikes, faults and jointing

iii) Deep pits or crevices in the surface, cavities and caverns etc., are observed.

Foundations supported over sloping rock may need to be anchored with steel dowels etc. to prevent sliding when the bond between the foundation and the rock is not sufficient.

Foundation of the same structure, unless structurally seperated by expansion joints, should not be supported partly on rock and partly on soil. In such satuations it is recommended to provide a cushion of yielding material for example a 50 cm thick layer of uncompacted sand beneath the footing on the rock.

ACKNOWLEDGEMENTS

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REFERENCES

Chen (1975) Foundations on Expansive Soils, Chapter 1. Elsevier, 1975.

CBRI (1978) Handbook on Under-reamed and Bored Compaction Pile Foundation: CBRI, INDIA 1978.

Senanayake (1986) Geotechnical Mapping of Low-Lying Areas in and around Colombo City - Asian Regional Symposium on Geotechnical Problems and Practices in Foundation Engineering Colombo 1986.

NAVFAC (1971) Design Manual NAVFAC-DM-7, Department of the Navy, USA, 1971.

CONSTRUCTION OF SHALLOW FOUNDATIONS WITH SPECIAL REFERENCE TO POOR SOIL CONDITIONS - A CASE STUDY

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Built Environment Consultants

Soils susceptible to expansion and shrinkage with changes of moisture content are a constant source of trouble in the design and construction of foundations. Such soils, popularly known as peats and black cotton soils, occur extensively in many parts of the world, especially the Asian region including Sri Lanka. Several deposits of peat and peaty soils are found in and around Colombo particularly in low lying areas liable to flooding. They are generally found in poorly-drained topography and are usually associated with a high water table.

Buildings constructed on such soils, adopting the types of foundations commonly employed for other types of soil strata, are observed to crack extensively within a short period of their construction in spite of every reasonable precaution.

This is mainly because of differential movements caused by alternate swelling and shrinkage of soil with seasonal variation of moisture content. The construction of small buildings such as houses on such soils is often impossible because the cost of normal piled foundations would be uneconomic, and other methods may be ineffective. The design and construction of inexpensive and safe foundations in such soils poses a challenge to the Engineers. Often the Engineer finds himself between two opposing factors, the principles of soil mechanics to contend with and economic considerations limiting him, so that very often the solution is a compromise formula based on sound and intelligent engineering judgement and perhaps a little imagination!

Listed below are several examples of the use of different types of shallow and other types of foundations in poor soil conditions and each case is discussed in terms of its suitability, effectiveness, ease of construction and economy. The case studies include in particular:-

(i) Shallow foundation types for the low-cost Housing Schemes in Colombo constructed generally in peat and peaty soil deposits.
(ii) Houses constructed on inverted T-beam and slab and beam raft foundations.

(iii) Municipal-Housing Scheme constructed using 1"-2" diameter steel piles in Nigeria.
(iv) Foundations using under-reamed piles.

1. <u>Strip footings on fill for low-cost housing.</u>

The low-cost housing schemes constructed in the Maligawatte, Dematagoda and Wanathamulla areas are examples of building constructions adopting shallow type foundations on peat and peaty soils. Refer figure 1.
These are basically single storey dwelling type structures with brick or block walls founded on shallow strip footings. These foundations are constructed on a fill, gravel-sand-clay soil has been found to be suitable. The depth of fill generally is about 3 to 3½ ft., so that with the foundations constructed 1 to 1½ ft. below the fill level, there will be about 2 ft. thickness of fill below the foundation. This effectively reduces the stresses at the peat layers to about 0.1 Tons/Sq. ft. which has been found to be an acceptable value as suggested by values for unconsolidated undrained strength determined from laboratory tests for unconfined compression strength of peaty soils in and around Colombo.

The fill, which is an inevitable necessity to make the marshy land habitable in the first place, also has other beneficial effects in addition to reducing the stresses caused on the peaty soil layers and allowing the designer to use the higher strength of the fill material.

In general it is found that peaty soils in and around Colombo normally consolidate when loaded. In this process, the moisture content is reduced with a consequent increase in the strength of the peat. The weight of the fill material super-imposed on the peaty soil helps to consolidate the compressible peaty soil. This effect is sometimes used to advantage by the use of surcharging to accelerate consolidation and minimise settlements.

Further a gravel-sand-clay fill placed over a weak deposit like a peaty soil tends to act as a natural raft due to its stiffness. This raft action helps the loads from the strip footing super-imposed on the fill to distribute evenly causing lesser shear stresses on the peat, and to minimise the adverse effects of differential settlements.

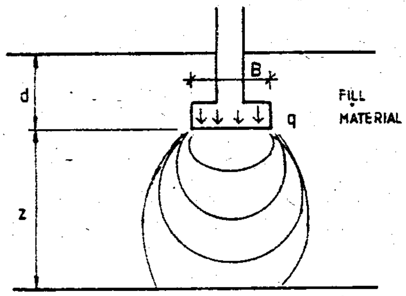


FIG. 1

2. (i) Raft Foundation.

Raft foundations are generally used on soils of low bearing capacity where the foundation pressures have to be distributed over as wide an area as possible, in order to work within the allowable bearing capacities. In soils of varying compressibility, such as soft peaty clays or loose recently deposited fill material, the partial rigidity given by the stiff slab and beam construction helps to bridge over areas of more compressible soil and thus reduce differential settlements.

The slab and beam raft (refer fig. 2 (i) is a shallow foundation generally adopted for buildings where stiffness is the principal requirement to avoid excessive distortion of the super-structure as a result of variations in the load distribution over the raft or in the compressibility of the supporting soil.

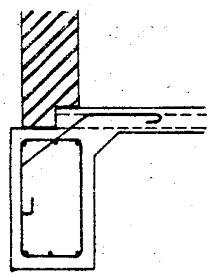


FIG. 2(I)

These stiff rafts are generally designed as "downstand beam" type which provides a level surface slab which can form the ground floor of the structure.

(ii) Inverted T-beam foundations - Refer Fig. 2 (ii).

This type of foundation has been used with much success for storeyed - housing, of load bearing wall construction, built on soils of low bearing capacity. While the flange slab of the T-beam helps to distribute the load on the soil and to reduce the bearing pressures to acceptable values, the beam portion, because of its stiffness, gives rigidity to the foundation and helps to span over any localised pockets of bad soil areas in the site and also helps to minimise differential settlements. This type of foundation has proved to be much more economical and amenable to construction than the alternative of a conventional raft foundation.

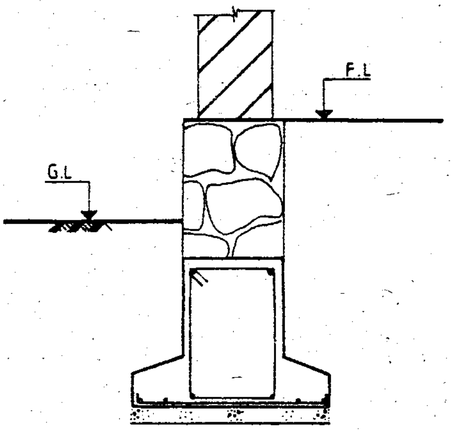


FIG. 2(II)

3. Use of Small-diameter piles in expansive soil.

The Construction of small buildings such as houses on expansive soils is often impossible because the cost of normal piled foundations could be uneconomic, and other methods such as the use of strip footings on fill may be ineffective in that the useful life span of the structures may be greatly reduced due to the ill effects of settlements. The use of small-diameter piles in such circumstances has been considered and practical applications have been reported.

Small-diameter piles are subject to smaller resulting uplift forces, and these could be overcome by the application of a suitably large load to each pile, if the bearing capacity is sufficient. Such a system of small-diameter piles would be especially suitable on a site when the expansive soil overlies a hard stratum of high bearing capacity. If this hard layer is at a shallow depth, it will be possible to use small-diameter piles effectively and economically.

In one practical application, in a municipal housing complex in Nigeria, small-diameter piles had been used at the site where the soil profile was found to be a very soft clay 7 to 8 ft. deep overlying a hard stratum. There were 23 or 24 piles per house, mainly 2" and 1½" in diameter, with a few 1" diameter piles. The 2" piles were scaffold tubes, the 1½" piles were black piping and the 1" piles were solid mild steel rods. The piles were driven with a drop weight to a required set. All the piles were coated with epoxy-tar before driving to protect against corrosion. A later improvement had been to provide cathodic protection. Load carrying capacities of the piles had been confirmed by load tests. Level observations had been commenced immediately after completion of construction and continued through two seasonal cycles of wetting and drying. The settlements observed had been of very small magnitude and the buildings had not suffered any damage due to differential settlements.

4. Foundations using under-reamed piles.

In cases where there is not an adequate bearing stratum at a reasonable depth for the piles, there is a possibility of ensuring adequate bearing capacity by under-reaming. The principle of this type of foundation is to anchor the structure at a depth where ground movement due to changes of moisture content is negligible. Depending on the loads to be carried, either a single under-reamed bulb or double/multiple bulbs may be provided. Refer fig. 4. The use of under-reamed piles has proved to give a cost-saving compared with traditional foundations in poor soils, particularly in expansive clays and the Indian Standards Institution has included the method in their Code of Practice for Design and Construction of Pile Foundations.

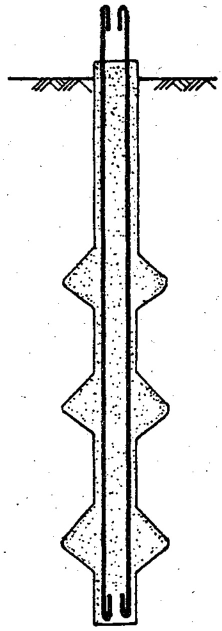


FIG. 4

"CONSTRUCTION MANAGEMENT WITH EMPHASIS ON SHALLOW FOUNDATIONS"

C H DE TISSERA

1.0 INTRODUCTION

Any exercise in Construction Management will have as its primary objective the achievement of the following:-

* the cost target;
* the quality target and
* the time target.

A Construction Management procedure to achieve the above objective will consist of an integrated management planning and control systems package.

Under management, issues relating to the organisation required to undertake the construction, the structure of the organisation, the specialisations necessary for the staff and the key tasks to be performed by the staff have to be determined. In addition such aspects as site layout, stores management and materials procurement procedures, manpower recruitment and personnel management systems, etc. have to be formulated. The Planning Sub System will have planning of construction operations, planning for the materials requirement and their procurement, planning of manpower requirement, planning for the plant and equipment requirements and their procurement, planning for the financial requirements. Under Control Systems, procedures for progress control, cost control and quality control have to be established.

Now a foundation, in its complete sense, includes both the structural member consisting of either the footing or raft etc. that transmit the superstructure loads to the ground and the interacting soil or rock below. It is therefore seen that foundations defer from other parts of the structure. Its behaviour is more dependent on the soil and rock below, which are materials of uncertain and non-uniform engineering properties. The most disturbing feature of an unsatisfactory foundation is that the difficulties and faults seldom appear immediately, and most of them are not obvious until the building is in use, when it is very expensive to remedy them.

It is therefore very important that foundation design and construction should be done with special care and attention. It is necessary to have comprehensive site investigation information, adequate design theories, and properly managed construction programme to ensure safeguards against above.

It is common in foundation construction to encounter variations due to:

(a) setting out errors;
(b) inadequate site investigations
(c) artificial obstructions.

The construction management procedure should have provision to take due note of variations arising from above and to have necessary documentation for establishing the variation claims. The construction manager, generally has to work within the relevant provision in the conditions of contract for this purpose.

2.0 MANAGEMENT ASPECTS OF A SITE INVESTIGATIONS PROGRAMME

An adequate site investigation programme is a pre-requisite to a safe and economic design and a successful foundation construction. The management aspects of site investigation has to take into account the following:

A manager who requires the relevant information from site investigations, should first of all specify clearly the particular objects of the site investigation programme. They may be to:-

(a) assess the general suitability of the site for the proposed works.

(b) enable an adequate and economic design to be prepared.

(c) foresee and provide against difficulties that may arise during construction, due to ground and other local conditions.

As the site investigation is done well ahead of the design and construction stages, it is very important that the investigations should be properly documented. The manager should specify in broad terms what should be included in the report. This report may among other things include essentially the following :

(a) An estimate of the competence of the underlying soil to support the proposed structure, stability of the site against slips, availability of services, building materials, manpower skills in the locality, information on access to site, factors such as incidence of flooding, ground water condition and some general aspects as ownership of land, limitation of use, leases, etc.

(b) Characteristics of the sub-soil and recommendations on probable behaviour under stress and interpretation of the data and soil parameters for design.

(c) Recommendations on appropriate construction techniques in relation to the locality, need for site protection, drainage, disposal of debris, refuse and surplus excavation materials, de-watering systems wherever applicable.

(d) Guidelines for preparation of cost estimates in relation to the prices prevailing in the locality including information on materials that are available at site, which can be recovered and used for both temporary and permanent works.

As the techniques available for site investigations today are very extensive, the Manager has to choose the particular technique in relation to the size of the job in hand to have the most cost effective site investigations programme.

In the planning of a site investigations programme, the Manager has to at least ensure that 3 basic steps are gone through, viz:

(a) A desk study
(b) A reconnaissance survey or walkover survey
(c) Detailed investigation.

In setting up a management structure for detailed site investigation programme, it is necessary to firstly examine the key tasks that have to be performed. Initially a plan must be prepared for the field work and the laboratory work including the analysis of data, based on the scope of the job in hand, and the information available from the desk study and the reconnaissance survey.

After the analysis of initial field and lab work, plans for the detailed field and lab work are prepared and ammended as and when necessary. A report is thereafter prepared on the basis of all the information and the results of the investigation. This report may consist of two parts viz:

(a) Data and Information
(b) Analysis and recommendations.

The first part may be available as general information, while the second part would be classified information for limited use as determined by the manager.

A bar chart showing the key tasks and the sequence in a site investigation programme is annexed.

In terms of personnel, the three main responsibilities are:

(a) Geotechnical Engineer - responsible for planning and directing lab and field work and prepare a report.

(b) Field Technicians - to carry out field investigations in accordance with specifications and instructions of the geotechnical engineer.

(c) Laboratory Technicians - to carry out lab testing in accordance with specifications and instructions of the geotechnical engineer.

During the Designs Stage, the necessary management considerations include decisions regarding:-

(a) appropriate design criteria, design theories,

(b) interpretation of site investigation data, and site conditions.

(c) Provisions for obstructions and other utilities that might be encountered during excavations,

(d) Providing guidelines for excavation, shoring, de-watering,etc.

(e) Preparation of a set of comprehensive working drawings to facilitate estimation and construction.

3.0 CONSIDERATIONS DURING TENDER STAGE

Emphasis has to be given during the tender stage for the following:

(a) Study of plans, bills of quantitites and specifications to determine the appropriate conditions of tender, form of tender, pre-qualification for contractors and preparation of budget estimates.

(b) The preparation of a pre-tender method statement.

(c) Preparation of a macro level pre-tender construction programme.

(d) Identification of relevant factors influencing pricing giving due consideration to construction techniques, type of plant and equipment to be used, norms for labour, de-watering systems, planking and strutting, materials to be encountered during excavation and such other aspects as site office overheads, head office overheads and the mark up necessary for risk factors, profits, etc.

4.0 MATTERS RELATING TO THE CONSTRUCTION STAGE

During the construction stage, it is necessary to review among others, the following in the construction management exercise:

(a) Preliminaries and setting out stage
(b) Planning of construction processes
(c) Planning for resource inputs
(d) Quality Control
(e) Progress Control
(f) Cost Control.

During the preliminaries and setting out stage, a proper layout plan has to be prepared. It is often noted that setting out variations between working drawings and the actuals arise due to errors of the consultants or due to change orders. This results in changes in scope of work as well as cost. In construction management, consideration should be given for proper documentation of such variations for subsequent variation claims.

Many variations to the original scope of work arise due to inadequate site investigation and encountering artificial obstructions. The conditions of contract applicable to the particular situation will determine the liability and responsibility in respect of these variations between the client and the contractor.

Construction management procedure should have provision to document the circumstances requiring the particular variations, the remedial action undertaken and the consequential changes in use of men, materials, machinery and money to establish variation claims.

The structure of the construction organisation and the staff to be deployed should be able to provide the following specialised knowledge input:

(a) Geotechnical
(b) Contract administration
(c) Construction planning and progress control
(d) Construction materials and technology
(e) Surveying and levelling
(f) Quality control
(g) Financial planning and cost control
(h) Job co-ordination and documentation.

These expertise may be provided either through in-house staff or from visiting staff.

The structure of the organisation and the key functions to be assigned to respective personnel should specify the scope, responsibility and accountability in respect of each of the above areas.

In laying out the site, provision should be made for the usual security fencing, site offices and stores, workmen's accommodation, outdoor storage depots for bulk materials, internal roads and material movement paths, crane tracks, carpentry and barbending sheds, concrete mixing etc. In addition, temporary bench marks and grid co-ordination, references should be indicated in the layout plan. Special attention at this stage should be paid to the movement of excavation and earth-moving machinery, allowance for barricades around trenches, allowance for de-watering system and leadaway drains, allowance for spoil heaps for back filling, disposal and dump sites for debris and surplus excavation materials. Depending on the extent of the site and the number of sub-structures, it would be advantageous to layout a grid pattern on the site plan and establish the grid co-ordinates and temporary bench marks at suitable locations by an accurate land survey. Thereafter, all setting out and levelling should be done in relation to them to minimise errors.

During the foundation construction as in the rest of the work, stores management and material procurement procedures should be sound. This is important to have an un-interrupted supply of materials to maintain the construction programme. Foundation construction often is done under difficult conditions by coping with ground water and weather changes. A proper requisitioning system based on a material requirement schedule, purchase procedure, is necessary to minimise delays and over-expenditures in materials. A satisfactory stock control and an issue procedure will minimise wastage and pilferage.

Personnel management should ensure timely recruitment of necessary skilled staff, delegation of tasks, responsibility and authority, procedures for communication and decision making, motivation and welfare of workers, and provision of on-the-job training.

Planning of foundation construction would also fall into the usual pattern of, planning of the construction operations, planning of material requirements, planning of manpower requirement, planning for plant and equipment and planning for the financial requirement.

Initially, a "process analysis" will identify the different activities that have to be performed, their logical sequence and the allowable time durations. Based on this information, the construction operations are scheduled either in the form of a bar chart or a similar method. When the foundation construction is a non-repetitive type, extensive in nature and involving a large number of activities to be carried out, and also when the circumstances require the completion on a critical target, a planning technique such as the CPM is appropriate. If the foundation construction has a repetitive content such as in the case of a construction of a large number of similar pad foundations, a technique like the line of balance or serial production method would pay handsome dividends. The concept of economy of scale in mass production in relation to unit price is being applied in this case. The repetitive activities performed on many foundation activities by specific groups of workers can increase productivity, reduce wastage and achieve efficiency, thereby reducing unit cost. Once the appropriate planning for the construction operations has been done, it would be possible to determine the schedules of material requirement, manpower and plant requirement to undertake the construction. It is advisable to prepare histograms for the resources, which are considered to be critical for the construction.

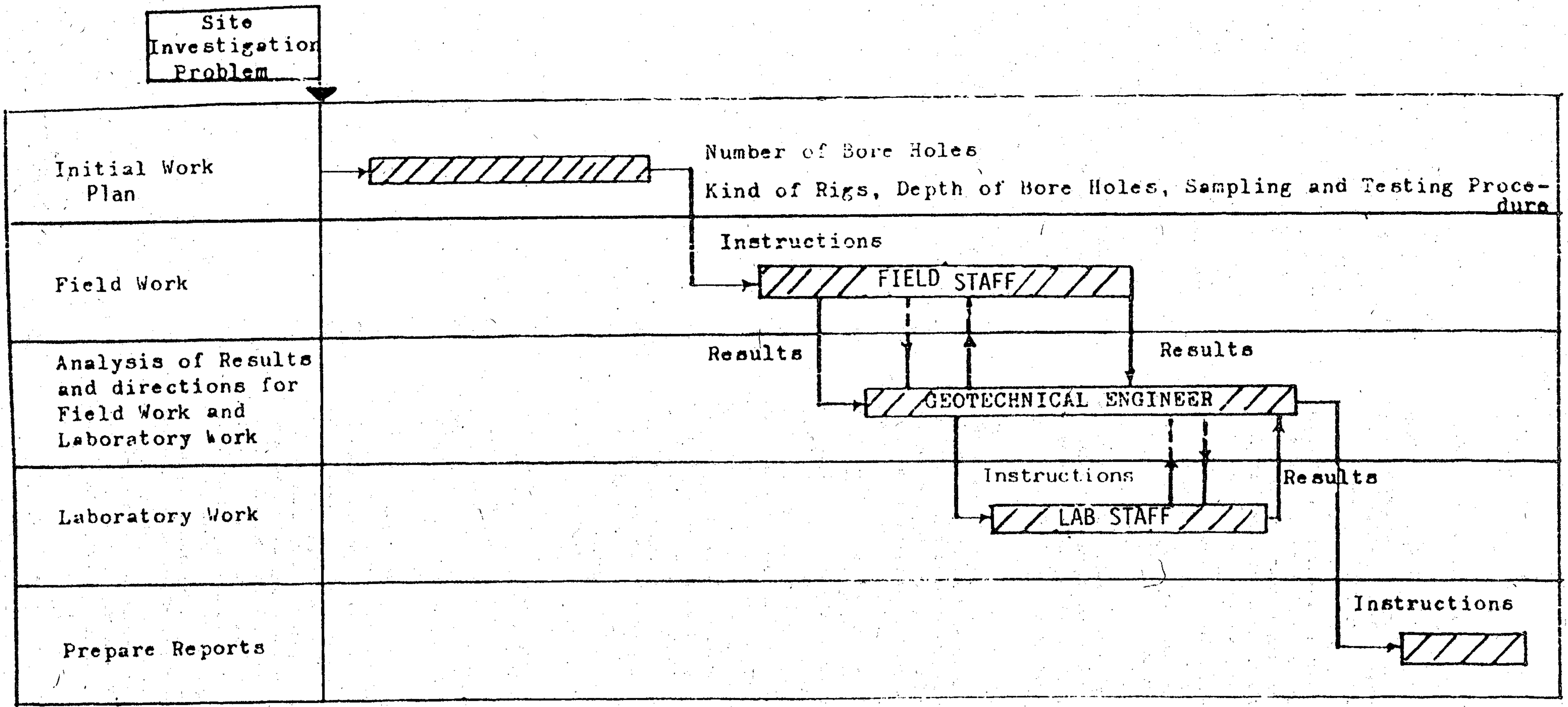
Under the control systems, the progress control activity shall ensure that the time targets in the construction management objective will be achieved. It is recommended that a periodical progress review to determine the actual performance against the given targets in terms of activities and the deployment of resources are monitored and remedial measures taken on any short falls.

The quality control procedure involves procedures to ensure conformity with specifications, conformity with design criteria and conformity with working drawings. With regard to specifications, due care in relation to excavation, de-watering, instrumentation and monitoring of neighbouring structures as necessary, inspection of cuts, fills and foundation levels, compaction, quality of materials used and the workmenship are being ensured. The conformity with design criteria will necessitate the supervision staff using their geotechnical knowledge to take appropriate decisions as construction proceeds. It will be appreciated that in foundation construction, the design guidelines and the working drawings would not be able to specify all the site conditions that may be encountered, and innovative decision making of the site engineer in conformity with design theories will be essential. The quality control procedure should also ensure that the construction shall conform where possible to the dimensions, depths and sizes specified in the working drawings and the B.O.QQ. Proper record keeping in this respect is essential to document any variations as well as extra work orders, where information on man hours, de-watering, nature of the excavated soils, use of any shoring and equipment, etc. are provided, to establish variation claims. It is also very essential that records are maintained to compile a set of "as-built" drawings, so that once the structures are buried under ground, information will be available for future use. Under qaulity control procedure, inspection and testing for finished works including load testing as necessary have to be carried out.

In cost control, it is necessary to have a costing system and a review procedure. The costing system may include data on resource consumption, the determination of unit costs of such resources consumed, and actual costs incurred in completing the different construction activities. These actual costs are compared with the estimated costs, and a variance analysis is carried out on the changes. Independently reconciliating reporting system, will ensure that the data being reviewed in cost control are reliable and accurate. The review procedure will facilitate the comparing of actual costs, against estimated costs, and determine what follow up action is necessary.

This overview of construction management issues in foundation construction is presented to stimulate a discussion. Detail procedure in respect of most of the methods suggested has to be looked up in relevant Construction Management text.

Bar Chart showing Key Tasks in a Site Investigations Programme



Ideally interaction between geotechnical engineer
and site and laboratory throughout most of the site investigation desirable.