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SITE INVESTIGATIONS

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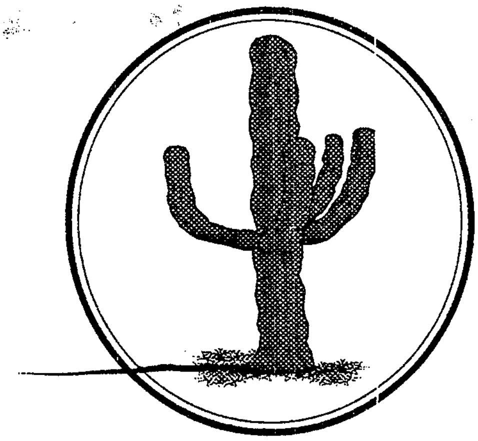

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Programme of the 10th Anniversary-SLGS Conference on Site Investigations - 1996

Inaugural Session

8.45 - 9.15	Registration of Participants
9.15 - 9.20	Inauguration & Lighting of Oil Lamp
9.20 - 9.30	Welcome Address
9.30 -10.15	Keynote Lecturer "Site Investigation Using In-situ Testing Techniques" -by Prof. H Nimal Senevirathne,

Session I

Chairman Mr. K W Perera

10.15 -10.35	Site Investigations for Projects in Reclaimed Marshy Lands by Dr. Sunil de Silva
10.35 -11.00	Tea
11.00 -11.20	Determination of Piezometric Pressure Fluctuations in Construction by Mr. A A Virajh Dias, Mr. P P D H Pallewela, Mr. V S Jayarathna.
11.20 -11.40	A Critical Assessment of Site Investigation Practices in Building in Low Laying Areas by Prof. B L Tennekoon, Miss. H A Y Chamindani and Miss. K A Madunishanthi
11.40 -12.00	Experimental Findings of Shear Strength Parameters of some Problematic Soil deposits in Sri Lanka by Mr. A A Virajh Dias & Mr. M Abeysinhge
12.00 -12.45	Discussion
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Session - II

Chairman Mr. D P Mallawarachchi

13.45 -14.05	Some Hard Realities Faced in Planning Measures for Landslide Disaster Mitigation through Geotechnical Investigation Resorvoirs by Mr. N M S I Arambepola and Mr. A A Virajh Dias
14.05 -14.25	Geotechnical Investigations in Medium Scale Irrigation Projects by Eng. E A C Ekanayake Eng. K W Perera,
14.25 -14.45	A Review of Site Investigation Practices in Lateritic Soils for Buildings by Prof. B L Tennekoon, Miss H A Y Chamindani & Miss. K A Madunishanthi
14.45 -15.05	Investigation of Kabaragala Rockfall cum Debrisflow Mr. N M S I Arambepola and P P D H Pallewela
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Keynote
Lecture

Site Investigation Using In-situ Testing Techniques

by

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SITE INVESTIGATION USING IN-SITU TESTING TECHNIQUES

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1. INTRODUCTION

The practice of site investigation is undoubtedly one of the most important objectives of a geotechnical engineer's work. The main objective of a site investigation is to accumulate all relevant data relating to stratigraphy and physical properties of the ground required for the successful realisation of a project (Johnston (1983)). A complete geotechnical investigation of a project should necessarily establish,

a. the stratification of the materials within the volume which is influenced by the proposed project/construction;
b. the engineering properties of these materials;
c. a prediction of the optimum process by which a project may be realised and,
d. a prediction of the performance of the project and it's influence on surroundings.

These four components are highly inter-dependent and all must be carried out at a competent level if the final result is to be meaningful. The data required for these components should be derived by whatever means is appropriate and relevant, to the characteristics of the site and to the proposed development, using the most economic and technically accurate methods available. The geotechnical engineer should possess a sound knowledge of all available techniques, their limitations, reliability, and relevancy to the geotechnical problem under consideration to achieve this aim.

In-situ and laboratory testing techniques represent the two principal approaches for the determination of engineering properties required for geotechnical investigation. However, it is often a combination of the two techniques which will provide the best information. Unlike laboratory testing, in-situ testing on many instances provide information on stratigraphy thereby reducing the amount of drilling or other direct sounding which may be necessary. This paper describes in-situ testing techniques available for site investigation, their relative merits in different soils under various ground conditions, and their usefulness as sounding techniques.

2. APPLICABILITY OF IN-SITU TESTING METHODS

Mitchell et al (1978) lists the following main reasons for the growing popularity of in-situ testing.

a. their ability to determine properties of soils such as sands and offshore deposits that cannot be easily sampled in the undisturbed state.
b. their ability to test a large volume of soil than can conveniently tested in the laboratory.

c. their ability to avoid some of the difficulties of laboratory testing such as sample disturbance and the proper simulation of in-situ stresses, temperature, and chemical and biological environments.
d. the increased cost effectiveness of an exploration and testing program using in-situ methods.

However, there are significant limitations in in-situ testing, an understanding and awareness of which are essential to the correct usage of in-situ testing. Robertson (1986) lists them as,

a. stress direction and stress path cannot be independently varied in most cases. Principal stress directions and stress path in the tests may differ from those in the real problem. Rotation of principal stresses may occur in the test but not in the real problem.
b. Drainage conditions cannot be controlled independently.
c. The possible effect of future changes in environmental conditions cannot be readily determined.

In-situ test methods can be divided into two basic groups, namely, logging or stratigraphic profiling methods, and specific test methods. The logging methods are often either penetration type or geophysical type tests and, are usually fast and economical; they often provide qualitative estimates of various geotechnical parameters. They are best suited for stratigraphic logging and for preliminary evaluation of soil parameters. The specific test methods are used mainly for the measurement of physical properties at specific locations; they are usually more specialised, and, therefore often slower and more time consuming to perform than logging tests. They provide specific soil parameters such as shear strength, and used when more detailed assessment of soil properties are required. These two basic groups of in-situ testing are generally complementary in their usage.

Table 1 illustrates a partial list of major in-situ techniques available, and their perceived applicability as in June 1986, given by Robertson (1986). The methods are listed in the table in approximate order of cost and complexity therefore logging methods appear towards the top of the list. The grades (A, B, etc.) are based on experience and a qualitative evaluation of the confidence level assessed for each method. The perceived applicability changes with the sophistication of the equipment; a self boring pressuremeter with a pore pressure probe would have a high applicability in measuring piezometric pressure while one without a probe would have little or no applicability.

Looking at Table 1, it is clear that most test methods have at least reasonable applicability in softer and finer soils (sand, silt, and clay); the best overall ratings in these soils are for self boring pressuremeters (SBPMT) and for electronic piezo-friction cones (SCPTU and CPTU). Most of the testing methods have low applicability in rocks and gravel which can be a problem in the Sri Lankan context due to the predominant presence of residual soils in the country. Prebored pressuremeter seems to be the best testing method under these conditions. However, a combination of geophysical methods (resistivity and seismic) will have high to moderate applicability in all terrains except in finding piezometric pressures, permeability, consolidation characteristics, in-situ stresses, and stress history. The author believes that geophysical testing techniques have improved considerably over the last decade; their application when supplemented with a certain amount of direct drilling and testing (laboratory or field) may be the most economical and suitable in large to medium scale projects.

The remainder of this paper is confined to a discussion of major logging test methods, major specific test methods and a brief introduction of combined test methods. Major logging methods discussed are penetration techniques (SPT, CPT, and DMT) and geophysical methods. Specific test methods included are pressuremeter test (PMT), SBPMT, and screw plate test (SPLT). In combined test methods logging equipment or methods are modified so that soil parameters at specific points may be obtained like in special test methods.

3. LOGGING TEST METHODS

3.1 STANDARD PENETRATION TEST (SPT)

The standard penetration test developed in the United States in the early part of this century is by far the most popular logging test method. The SPT parameter (N) is the number of blows required from a standard hammer to drive a predefined split spoon sampler at the bottom of a borehole by 300 mm. The test which is relatively simple can be easily carried out at frequent depth intervals using rugged equipment. It is possible to conduct the test in most of the soil types and the soil sample obtained from the split spoon sampler would enable the direct inspection of the material. Though the test is dynamic and is not amenable to analysis, many useful empirical correlations have been developed between soil parameters and SPT value.

SPT has the best use in sand and very limited use in clays; the test, sometimes with a variation of having a solid cone for the open ended tube has been used in gravels and cobbles. According to Douglas (1983) unsound field procedures may cause serious errors in N-value; problems may occur in sands and gravels below watertable due to caving in of uncased boreholes, running sand condition with shell and auger or hollow flight augers or from, compaction from driven casings. According to several researchers (e.g. Kovacs and Salomone (1982)) the most significant factor affecting the measured N-value is the amount of energy delivered to the drill rods which may vary between 30% to 80% of the theoretical maximum for the traditional rope cat-head type testing equipment. The introduction of automatic release trip hammer has considerably reduced the variation in energy levels still there is scope for better standardised equipment which can reduce the variation further. However, the problem here is that most of the earlier correlations are based on old equipment and therefore may not be applicable with new equipment. In countries like Sri Lanka where no local correlations do exist, there is opportunity for leap-frogging into new technology and developing correlations based on them.

SPT is the most popular method of investigation and design of routine foundation work in the United States, used in 80% - 90% of the total projects. However, the method of design is highly dependent on the local SPT procedures and therefore will continue be based on local experience. According to a study by Talbot (1981) the ratio of predicted settlement to observed settlement using various design methods available in the literature varied from 23.8 to 0.11 with an average of 1.92. This should serve as a warning to our designers using SPT based methods without any local verification of accuracy. Out of the available methods in general, direct methods (i.e. no corrections to measured N-value) like that of Parry(1977) seem to be the most consistent with a small margin of conservatism.

The generally accepted correlations between N-value with friction angle and relative density for sand (as given by Mitchell and Katti (1981)) are given in Table 2.

STATE	VERY LOOSE	LOOSE	MEDIUM DENSE	DENSE	VERY DENSE
N-VALUE	<4	4-10	10-30	30-50	>50
RELATIVE DENSITY	<15	15-35	35-65	65-85	85-100
DRY UNIT WEIGHT (kN/m^3)	<14	14-16	16-18	18-20	>20
FRICTION ANGLE (°)	<30	30-32	32-35	35-38	>38

TABLE 2 N-VALUE AND SAND PROPERTIES

3.2 DYNAMIC CONE PENETRATION TEST

The dynamic cone penetration test is a continuous test in which a rod usually equipped with a conical tip is driven into the ground by repeated blows from a falling weight (like in SPT test). Even though considerable variations exist, of the equipment and of the method of testing, at least in some countries the rods and the impact weight are similar to those used with SPT. Usually the cone tip has a cross-sectional area of 10 cm^2 and the blows required for a penetration of 300 mm or 200 mm is recorded.

Comparing with SPT, the test is relatively simple, inexpensive and would give a almost continuous record of driving resistance with depth. However, the main disadvantage of DCPT is the lack of standardisation of equipment which makes the development of empirical relations between the DCPT results and soil properties difficult. The main factors affecting the DCPT test results are cone size and geometry, energy transfer from hammer (same problem as in SPT) and rod friction. Rod friction may be reduced or quantified by using enlarged cone tips, rod rotation, rod redrive resistance, casings, and mud wash.

DCPT may be a useful logging tool in stratigraphic surveys though unlike in SPT visual identification of the soil is not possible. However, it may be used to supplement direct drilling in a site investigation. Interpretation using DCPT data is generally qualitative, based on local experience, and is used to evaluate soil profile and apparent variations in density. Some correlations do exist in the literature between DCPT results and soil properties though they must be used with extreme caution due to the factors mentioned above.

3.3 STATIC CONE PENETRATION TEST (CPT)

In the static cone penetration test, a cone - friction sleeve device is pushed continuously into the ground and the resistance by the cone and the sleeve are separately recorded. CPT has the advantages of simplicity of testing, continuous record, reproducibility of results, and greater amenability of data for rational analysis. It has strong application in soft soils where depth of penetration may even exceed 100 m provided that verticality is maintained. However, gravel layers, boulders, or heavily cemented soils can restrict the penetration and may damage the cones and rods. Sanglerat (1972) is a good reference on the use of cone penetrometers for soil exploration though many developments have taken place since then.

There are electrical and mechanical types of cone devices. The accepted European standard is a cone with 10 cm^2 base area (diameter 35.7 mm) and a tip apex angle of 60°. The friction sleeve, located above the conical tip at a distance not more than 5 mm from the cone must have the same diameter as the cone base and a standard surface area of 150 cm^2. The stroke of the device should be at least 1 m. If a friction reducer (to reduce friction due to rods) is incorporated into the device it should be at least 1 m above the cone base. The cone should be pushed into the soil at a constant rate of 2 cm per second. The vertical interval between the readings should not be more than 20 cm and penetration depth should be measured to an accuracy of at least 10 cm. The measurements may be taken electrically (e.g. using strain gauges etc.) or by using mechanical means such as hydraulic or pneumatic systems or using separate inner and outer rods.

Fig. 1 illustrates the Begemann cone, a well-known mechanical penetrometer which does not exactly confirm with European standards. When using this penetrometer, the cone tip is advanced first by 4 cm and tip resistance measured. Then the cone tip is advanced by further 4 cm along with the sleeve and total resistance obtained. The sleeve frictional force is calculated by subtracting the earlier determined cone tip resistance from the total resistance. Fig. 2 illustrates the electrical cone manufactured by Fugro limited. Unlike in the mechanical version, in the electric version the cone and friction sleeve do not move relative to each other; the cone resistance and sleeve friction are continuously monitored during penetration via separate strain gauged load cells. Mechanical penetrometers, though labour intensive, offer the advantage of an initial low cost of equipment and simplicity of operation. However, they have the disadvantage of a rather slow incremental procedure, ineffectiveness in soft soils, requirement of moving parts and generally poor accuracy. In contrast, electric penetrometers are costly and require skilled operators, but offer obvious advantages like, rapid testing, continuous recording, easy data handling, higher accuracy, and repeatability.

The results of CPT tests are reported as variations with depth of q_c, which is the total force acting on the cone tip divided by the base area of the cone, usually measured in kgf/cm^2, f_s , the sleeve friction force per unit surface area of the sleeve and, the dimensionless friction ratio (*FR*) defined as 100 $f_s/\ q_c$ at the same depth. Stratigraphic information is interpreted using correlation charts of the above three parameters q_c, f_s , and *FR* (e.g. see Fig. 3). Sandy soils generally have high cone bearing resistance and low friction ratios where as clayey soils have low cone bearing resistance and high friction ratios. Unlike SPT, CPT is ideally suited for fine grained soils and is an excellent tool for stratigraphic surveys (see e.g. Fig. 4). The measurement of sleeve friction is somewhat less accurate than that of cone tip resistance; cones of different designs may produce variable friction sleeve measurements. Therefore, it is prudent to use a correlation based on the same cone for interpretation of CPT data. Even though the quantitative use of the CPT is generally complicated and not all that reliable there are many empirical as well as analytical methods of design for shallow foundations and deep foundations (piles etc.) based on CPT.

3.4 FLAT PLATE DILATOMETER TEST (DMT)

The flat plate dilatometer developed in ITALY by Marchetti (Marchetti (1980)) consists of a flat plate with a flexible circular stainless steel membrane located on one face of the blade (see Fig. 5). The membrane is inflated by high pressure gas and readings are taken of the gas pressure, when the membrane just lifts off the sensing disc (reading A), and when the central deflection of the membrane is 1 mm (reading B). The dilatometer is pushed into the ground at a rate of 2 cm/s using equipment like in CPT, and readings are taken at depth intervals. Readings A and B are corrected to obtain pressures P_0 and P_1 after allowing for membrane stiffness and for offset in the measuring gauge. The dilatometer results are interpreted using three parameters, material index, I_d given by $(P_1 - P_0)/(P_0 - u_0)$; horizontal stress index, K_d given by $(P_0 - u_0)/\sigma'_{vo}$; and dilatometer modulus, E_d given by $34.6(P_1 - P_0)$ where, u_0 is the in-situ hydrostatic water pressure and σ'_{vo} the in-situ vertical effective stress. An example of dilatometer test results is shown in Fig. 6. The DMT is a simple inexpensive test, which, when used with local correlations, can rapidly provide details of soil stratigraphy, stress history and deformation modulus. The correlations between the DMT indices and engineering properties are strictly empirical; correlations are found in the literature relating the DMT indices to friction angle, coefficient of lateral earth pressure and constrained modulus.

3.5 GEOPHYSICAL METHODS

In geophysical methods of site investigation, the spatial distribution of a physical property of sub-surface strata is determined, and the contrast of the magnitude of physical property is used to obtain sub-surface stratigraphy. Resistivity, and wave velocity (seismic (pressure or shear), radar, acoustic, etc.), are normally used physical properties. The accuracy of the material distribution determined from these surveys will depend on the contrast of the used physical property of different soil groups present in the ground tested.

In resistivity surveys a direct or low frequency alternating current is transmitted through a pair of electrodes placed on earth surface and the voltages arising from this current flow is measured between another pair of electrodes. The most frequently used electrode arrangements are Wenner and Schlumberger arrays (Fig. 7). The geophysical parameter used in interpretation of resistivity survey is apparent resistivity which is equal to the true resistivity if the earth is homogeneous. Resistivity surveys may be carried out in a profiling or sounding manner. In sounding the distance between electrodes is increased in a regular manner about a fixed centre and a graph of apparent resistivity versus electrode spacing is drawn. By comparing this graph with standard curves prepared for layered models thickness of strata may be determined.

In profiling, the electrode spacing is kept constant whilst the centre point of the array of electrodes is varied in a grid pattern; the electrode spacing represents a specific depth at which the distribution of different material is given by the variation of measured apparent resistivity. All resistivity surveys may be interpreted using computer programs available which are based on direct inversion, i.e. calculation of number of layers and their thickness directly from resistivity data. If lateral resistivity variations are substantial resistivity sounding may not give accurate results. If the layer thickness is too small in comparison to its depth, regardless of its resistivity contrast, it may not be detected by a resistivity survey. The above problems may be overcome by using downhole techniques which are described later in this section.

Seismic refraction method is the most widely used method of geophysical investigation for the determination of depth to bedrock. The method involves timing first arrival of seismic energy from a source to lines of geophones located on the ground surface. The energy source is a hammer blow or in large scale investigations an explosive charge. At the geophones close to the source direct waves travelling on the surface arrive first. In contrast, the waves critically refracted from layers with high velocity will arrive first at geophones which are sufficiently far away from the energy source. By plotting the time of arrival of first wave versus the distance to geophone from the energy source, the thickness and wave velocity of the layers may be calculated. The presence of layers of low shear velocity closer to the surface, irregular ground features, and irregular bedrock topography may present problems in seismic investigations. Like in the case of resistivity surveys, downhole methods may be helpful in overcoming these problems.

In downhole method of seismic surveys, a seismic source is placed at the ground surface and the first arrival of waves down the depth of a borehole is obtained by monitoring the response of geophones placed along the borehole at depth intervals. In the crosshole method (see Fig. 8), seismic source and geophones are placed in different boreholes at the same depth and the time required for the wave to travel horizontally from one borehole to the other is determined at different depths. Crosshole and downhole methods are more sophisticated and expensive to use than the normal geophysical techniques but give more accurate results. The downhole method can be used with resistivity surveys by placing current electrodes at depth intervals down a borehole and measuring the surface voltage variations. There are empirical relations available in the literature between resistivity or seismic wave velocity and other material properties. In addition, seismic velocity of elastic soils are directly related to the elastic stiffness parameters. This is particularly useful in the design of machine foundations and in earthquake resistant designs where the stiffness parameters of the material at low strain amplitudes are required.

Ground probing radar or GPR systems uses electro-magnetic waves for exploration. Even though GPR is applicable only for shallow depths due to high ground absorption, it provides high resolution images. Worsfold et al (1986) gives an example of the use of GPR in depth profiling of peat deposits. Geotomography is a new area of geophysical exploration where the seismic waves, radar, or electrical current is used to obtain the variation of material in a specific objective area. In seismic tomography, seismic sources and receivers are placed in boreholes, and on the ground surface to cover the objective area as much as possible. The seismic waves generated by small explosives or by other seismic sources travel through the project area in various directions and reach the receivers. The first time of arrival of these waves at different receivers is fed into a computer program which carries out a tomographic analysis to reconstruct the velocity distribution within the objective area. Similarly in resistivity tomography, instead of seismic sources current electrodes are used, and geophones replaced by voltage electrodes; the measurements are converted to a resistivity distribution using tomographic techniques. Fig. 9 illustrates how seismic and electrical tomography may be used to obtain the material distribution of a project area.

The resistivity and seismic methods may give complementary results and therefore combined use of them is often more powerful. Geophysical methods are frequently used to supplement a direct investigation or to optimise the drilling program by identifying the field profile thereby facilitating the strategic placing of drillholes on a specific site. How ever, they usually require more space for probing and are difficult to use in small sites.

4. SPECIAL TEST METHODS

4.1 PREBORED PRESSUREMETER (PMT)

The prebored pressuremeter of the type developed by Menard (1956) is widely used in in-situ soil testing. This pressuremeter illustrated in Fig. 10, consists of a flexible measuring cell which is expanded inside a prebored hole by applying air pressure to the liquid inside the cell. To ensure nearly right cylinder deformation of the cell, it is constrained in between two guard cells which were supplied with the same air pressure as the measuring cell. The volume change of the measuring cell due to the applied pressure is measured using a volumeter at the ground surface. Pressure and volume change readings taken are corrected for membrane stiffness, expansion of pressure lines, and for head difference between the measuring cell and ground surface.
PMT results are usually presented in a graphical plot of applied pressure versus volume increase (see Fig. 11); this graph shows three distinct characteristic phases, namely, linear recompression of disturbed soil up to a pressure of P_0, linear increase in pressure from P_0 to yield pressure P_f, and rapid volume increase indicating plastic deformation of the soil up to the limit pressure P_L. The limit pressure can be directly related to the shear strength of the soil. There are methods of calculating the coefficient of lateral earth pressure at rest and elastic moduli of the soil using the pressuremeter results though disturbance during boring may make them not all that reliable; local experience and locally derived empirical formula are used in interpretation of results. The elastic moduli of the soil are more accurately measured by carrying out small loading-unloading cycle during the test. PMBT has strong applicability in stiff soils, soft rocks and in residual soils even when the gravel content of the soil is appreciable. In sands, a flexible spiral metal casing is used to enclose the membrane thereby preventing the collapse of the borehole. The applicability of PMBT in sands (particularly below watertable) and in soft soils is limited due to severe disturbance which may occur during preboring.

4.2 SELF-BORING PRESSUREMETER (SBPMT)

Self-boring pressuremeters are developed considering the extent of disturbance during boring which may occur with prebored pressuremeters. One version of this equipment developed at University of Cambridge, U.K. is known as Camkometer (see Fig. 12). The camkometer is slowly jacked into the ground; the material displaced is forced into the cutting shoe inside which a central cutting shoe operates. Drilling fluid, either water or mud, is passed down the rotating shaft and this fluid, with the action of the cutter, reduces the soil into a slurry which is taken to the surface through the casing. The rubber membrane which is behind the boring part of the main instrument is pressurised using a gas cylinder like in the Menard system. Unlike in the Menard pressuremeter, instead of the volume change, the central radial deformation of the membrane is measured, thus eliminating the need for guard cells. The pore pressure is measured using a transducer located at the central plane of the membrane.

There are many empirical, semi-empirical and analytical methods available for determination of soil properties using SBPMT results. Because of the minimal disturbance caused to the surrounding soil during boring, self-boring pressuremeters may be used to obtain coefficient of lateral earth pressure, and elastic moduli of the soil reliably. By interpreting the pore pressure transducer response, it is possible to obtain consolidation parameters of the soil. The instrument may be used in stiff or soft clays and also in sands but the use in very stiff soils and soft rocks is limited. In sands below the watertable the pressuremeter is one of the few instruments which can give reasonable results though interpretation of results in this case may need lot of experience and expertise.

4.3 SCREW PLATE TEST (SPLT)

The screw plate is a pitch auger device that can be screwed to a desired depth (sometimes in a borehole) in a soil and loaded as in a traditional plate load test (see Fig. 13). The bearing area of the plate is taken as the horizontal projection of the single 360° auger flight. A variety of equipment and test procedures has been developed for screw plate by various researchers(e.g. Kay and Avalle (1982)). Usually a constant rate of loading or deformation is applied on to the plate and the load-deformation characteristics obtained, may be used to determine the stiffness and strength parameters of the soil. It is also possible to obtain consolidation characteristics by applying load increments and maintaining constant load in each increment until excess pore pressures are dissipated. The analysis of screw plate test results are difficult and complicated and the theories used involves various assumptions regarding the degree of binding or adhesion at auger-soil interface. Nevertheless SPLT is a useful test for determining vertical stiffness parameters for sands and undrained parameters for stiff clays.

4. COMBINED TEST METHODS

In combined test methods logging instruments are modified to obtain properties of the soils more directly; only modification of CPT is described here. In advanced versions of cone penetrometers extra-sensors for measuring variables such as pore pressure, shear wave velocity, electrical conductivity, temperature are incorporated. CPT is one of the most highly rated logging method for soft soils and the introduction of extra sensors make the determination of soil parameters more accurate. The most useful addition to an electric cone penetrometer is a quick response pore pressure element; the resulting instrument called piezocone can be used to measure pore pressure during cone penetration and pore pressure dissipation following cessation of penetration. The simultaneous measurement of cone tip resistance, sleeve friction and pore pressure greatly enhance the prospect of correct layer identification. The pore pressure ratio obtained by dividing pore pressure by the cone tip resistance is used for soil identification and may also give information on state of normal/overconsolidation of the soil. In the seismic cone a seismic sensor is installed in the cone and its response to an impact caused on the ground surface is monitored. This enable the determination of shear velocity and therefore dynamic elastic moduli of soils which can be very useful in machine foundation design and in problems involving earthquakes.

5. CONCLUDING REMARKS

There are substantial differences in the quality of site investigation practices in developed countries and in Sri Lanka. Sri Lankan practice of site investigation is still concentrated around the use of SPT in the simplest manner possible. Lack of local correlations between material properties and SPT values, and numerous varieties of drilling and testing equipment and methods used by the local contractors, make any such correlation a difficult task. One of the consequences of such practice is the properties of weaker soils are not properly assessed, and therefore are ignored in the design, leading to the frequent use of rigid rafts, or deeper foundations on strong layers such as end bearing piles, in cases where flexible rafts or foundations on weaker soils such as friction piles would have been more economical. With increasing degree of urbanisation in Colombo and other cities in Sri Lanka, effects of new constructions on nearby structures need to be more accurately evaluated. All these factors show the need for use of new or improved methods of site investigation by using good sampling and laboratory testing or by in-situ testing. Therefore, time has come to consider the use of new testing methods and to improve the quality of prevailing site investigation procedures in Sri Lanka, taking into account the experiences from the rest of the world, so that the gap can be eliminated in by the early next century.

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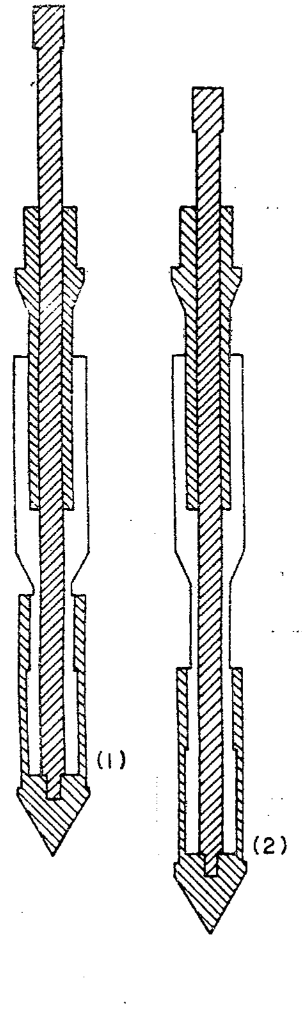
TEST METHOD	GEOTECHNICAL INFORMATION													GROUND CONDITION						
	SOIL TYPE	PROFILE	PIEZOMETRIC PRESSURE	ANGLE OF FRICTION	UNDRAINED SHEAR STRENGTH	RELATIVE DENSITY	COMPRESSIBILITY	RATE OF CONSOLIDATION	PERMEABILITY	SHEAR AND YOUNG MODULUS	IN-SITU HORIZONTAL STRESS	STRESS HISTORY AND O.C.R.	STRESS - STRAIN CURVE	HARD ROCK	SOFT ROCK - TILL ETC.	GRAVEL	SAND	SILT	CLAY	PEAT - ORGANICS
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	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
K0 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
PRESSUREMETER :																				
PREBORED (PMT)	B	B	---	C	B	C	C	C	---	A	C	C	C	A	A	B	B	B	A	B
PUSH-IN (PPMT)	A	B	B	C	B	C	C	A	B	A	C	C	C	---	---	---	B	A	A	B
FULL DISPLACEMENT (FDPMT)	C	B	B	C	B	C	C	A	B	A	C	C	C	---	---	---	A	A	A	A
SELF-BORING (SBPMT)	B	B	A	A	B	B	B	A	B	A	A	A	A	---	C	---	B	A	A	A
OTHER SELF BORING DEVICES :																				
K0 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

TABLE 1 IN-SITU TEST METHODS AND THEIR PERCEIVED APPLICABILITY (AFTER ROBERTSON (1986))

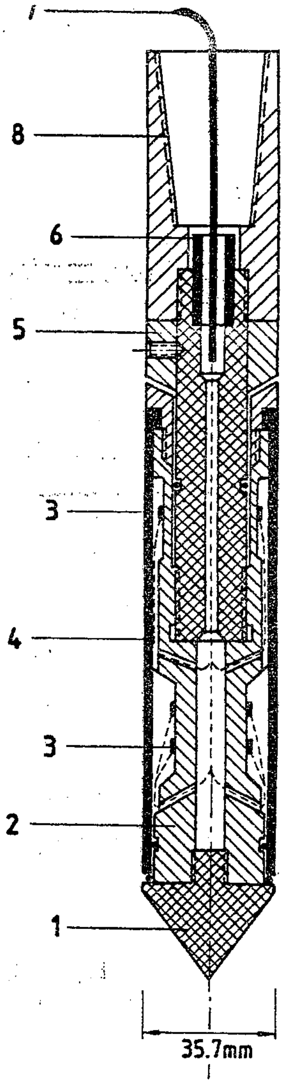


(1) Cone and friction sleeve retracted

(2) Cone in extended position

(3) Cone and friction sleeve both advanced

Figure 1. Mechanical Friction -cone Penetrometer (Begemann Cone)



1 Conical point ($10cm^2$)
2 Load cell
3 Strain gauges
4 Friction sleeve
5 Adjustment ring
6 Waterproof bushing
7 Cable
8 Connection with rods

Figure 2. Electrical Friction-Cone Penetrometer (Fugro-type)

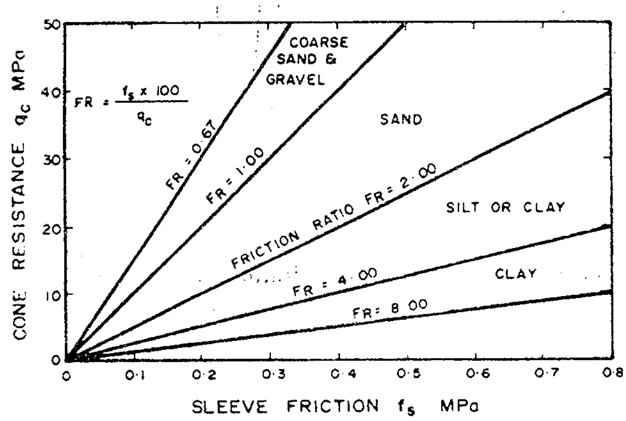


Figure 3. Profile Interpretation from q_c and FR (Marr 1981)

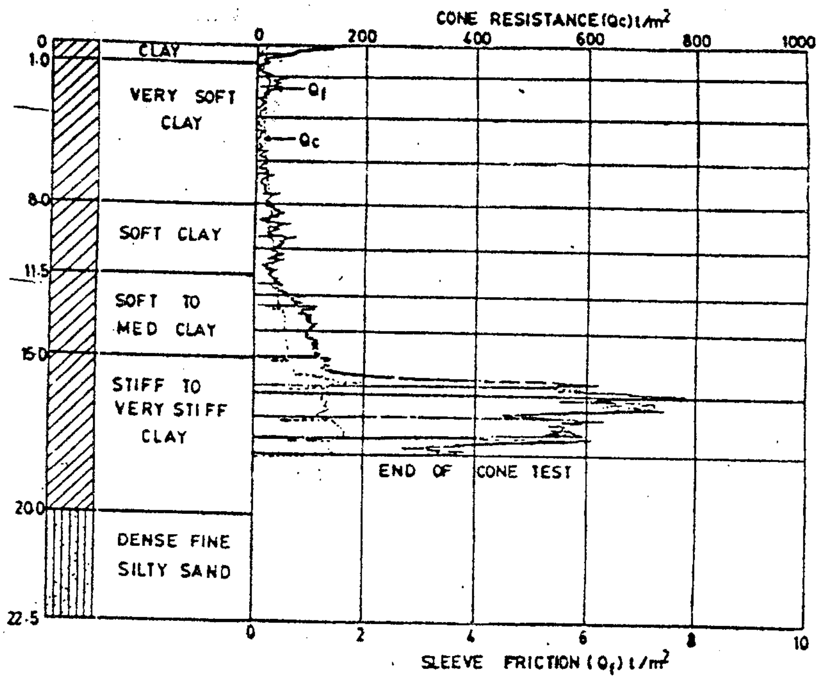


Figure 4. Soil Profile and CPT Results in Soft Clay Deposit-Nong Ngu Hao, Thailand.

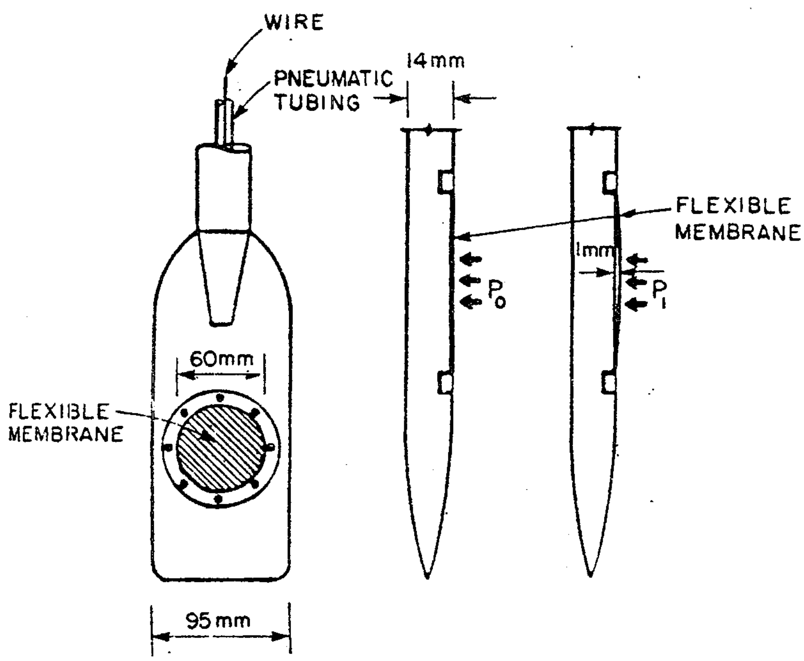


Figure 5. Marchetti flat plate dilatometer.

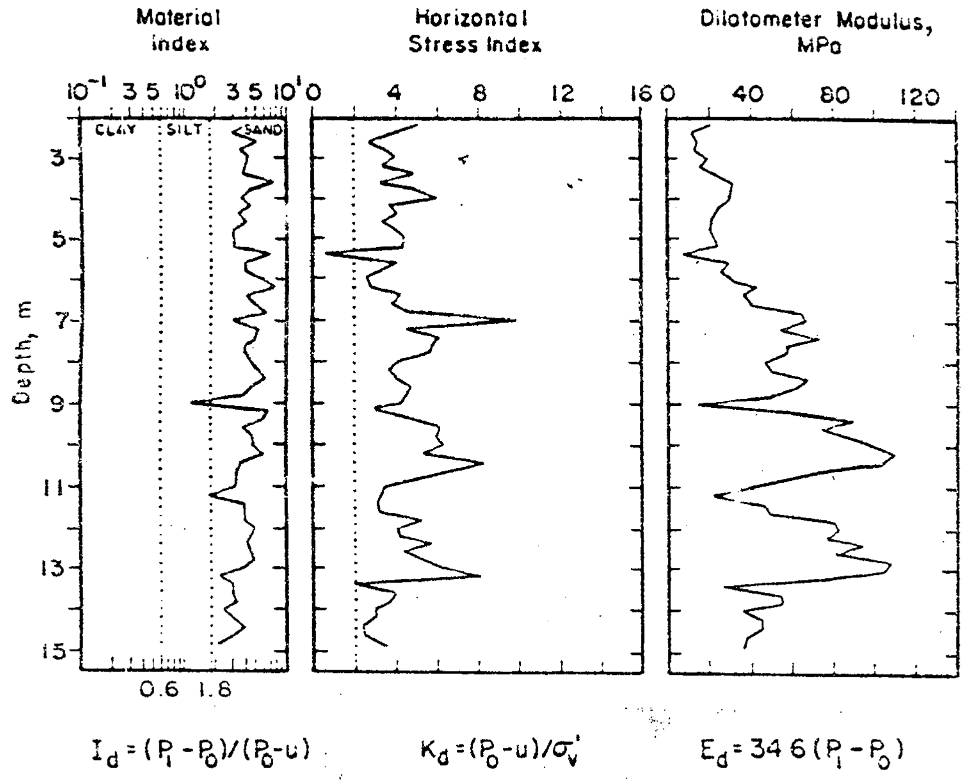


Figure 6. Example of dilatometer index parameter presentation.

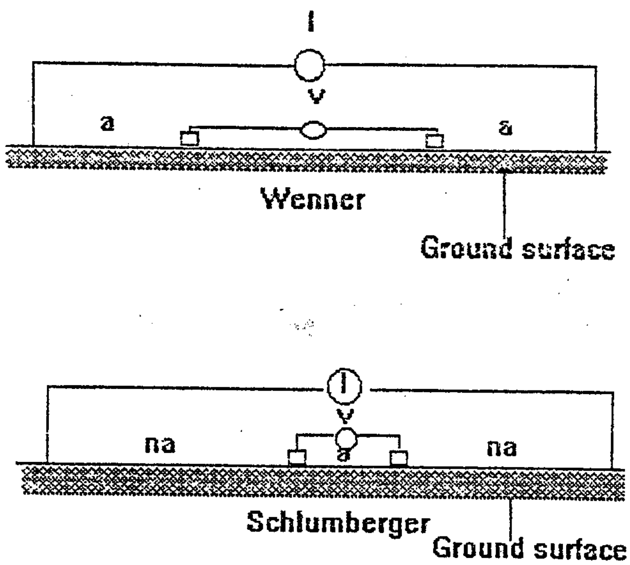


Figure 7. Electrode arrays used in resistivity surveying

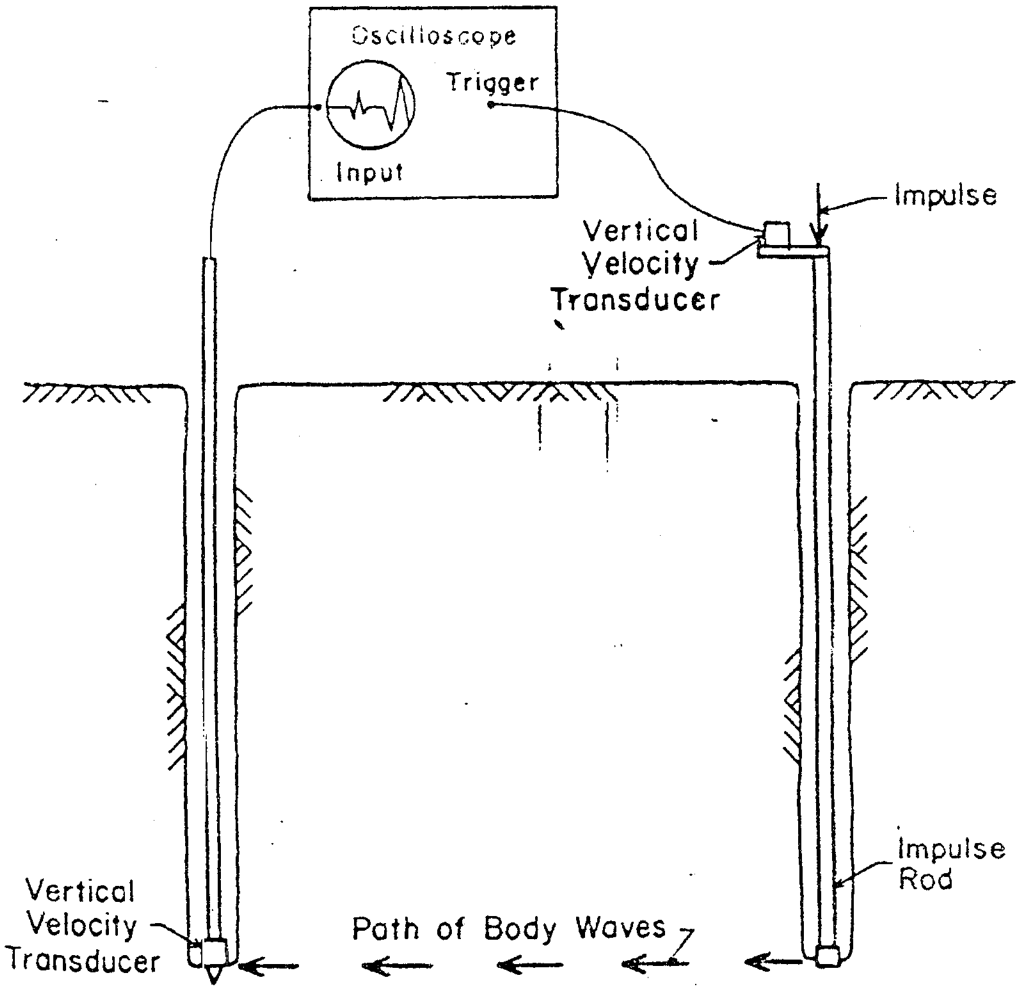


Figure 8. Schematic of Cross-Hole Seismic Survey Technique

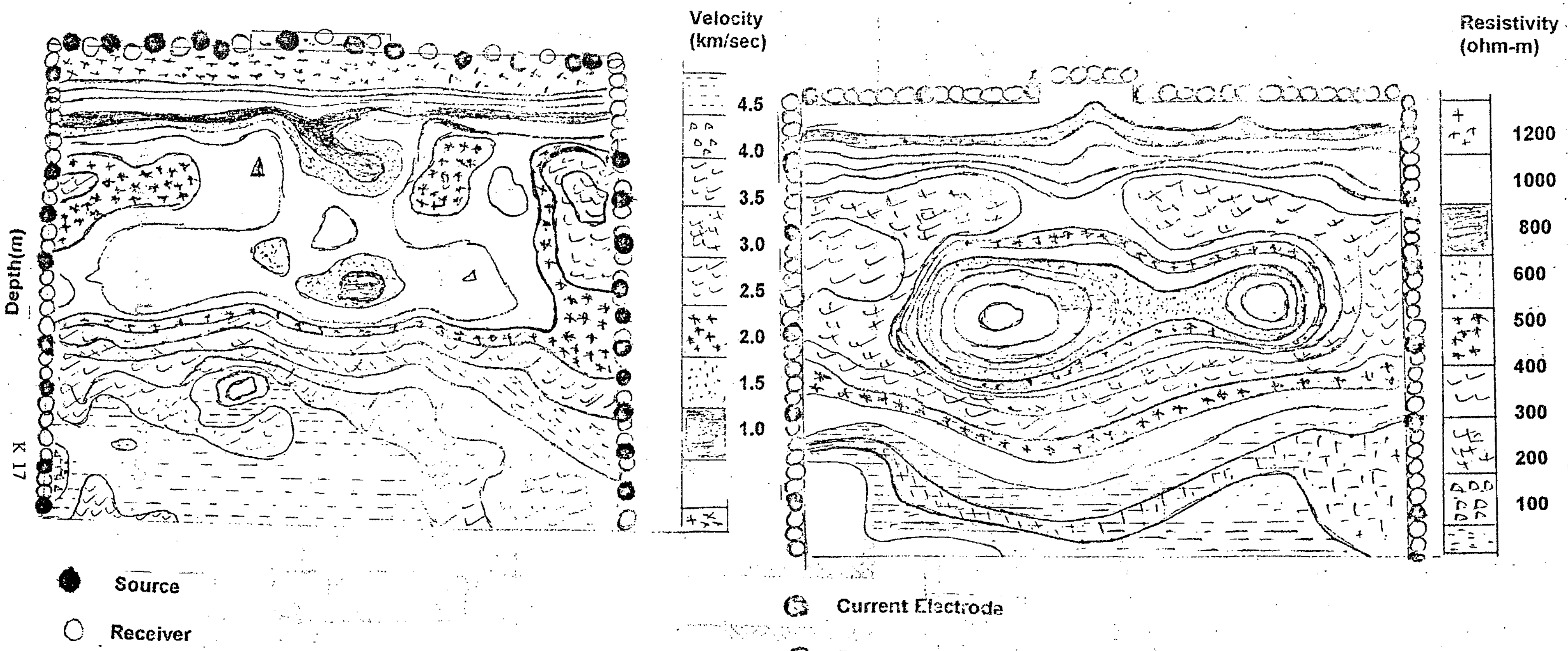


Figure 9. Resistivity and Seismic tomography result of a site

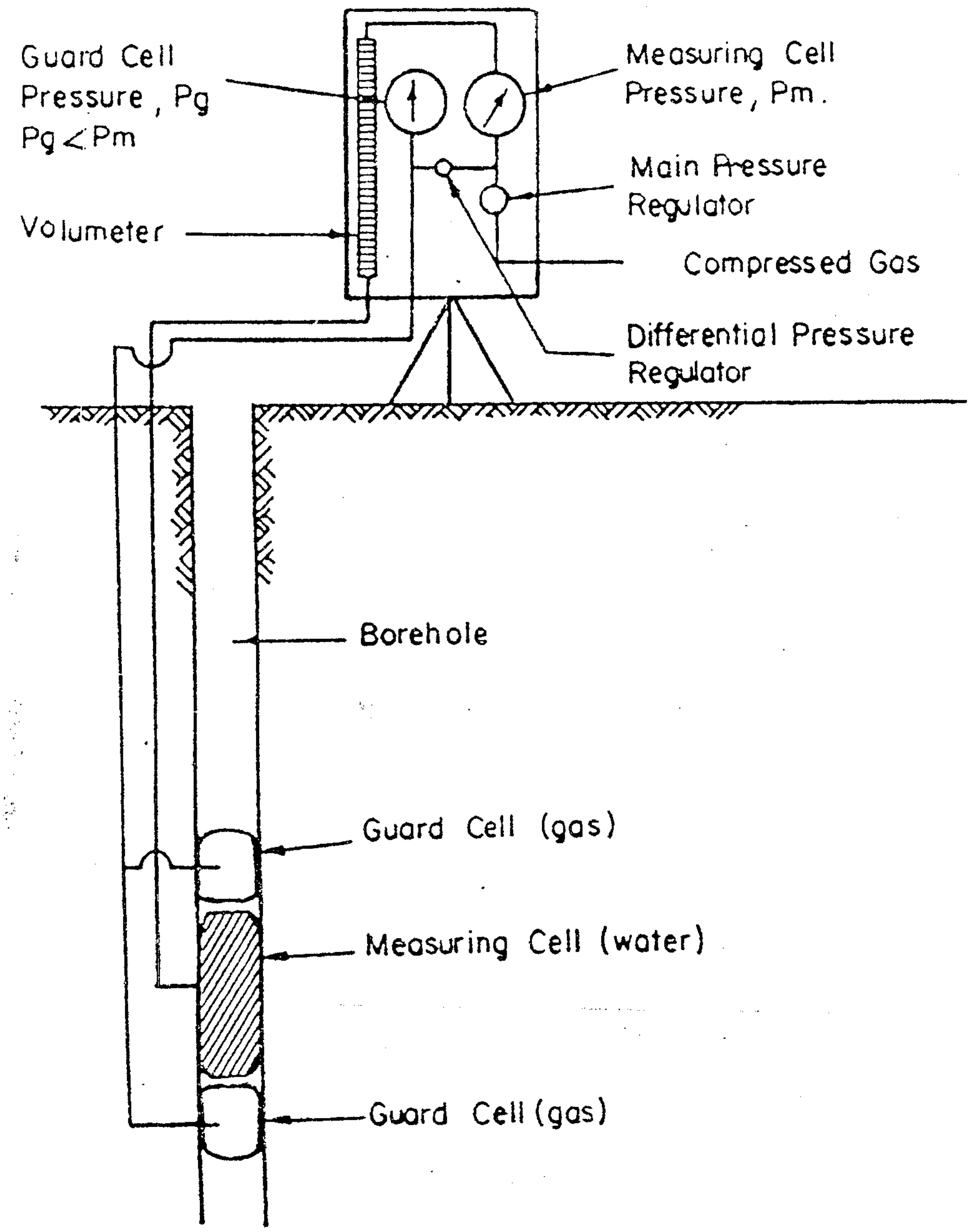


Figure 10. Menard Pressuremeter

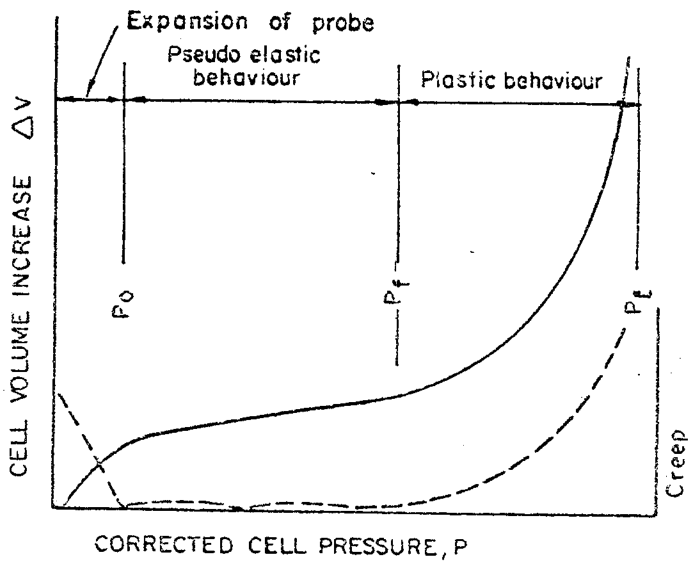


Figure 11. Typical Menard Test Result

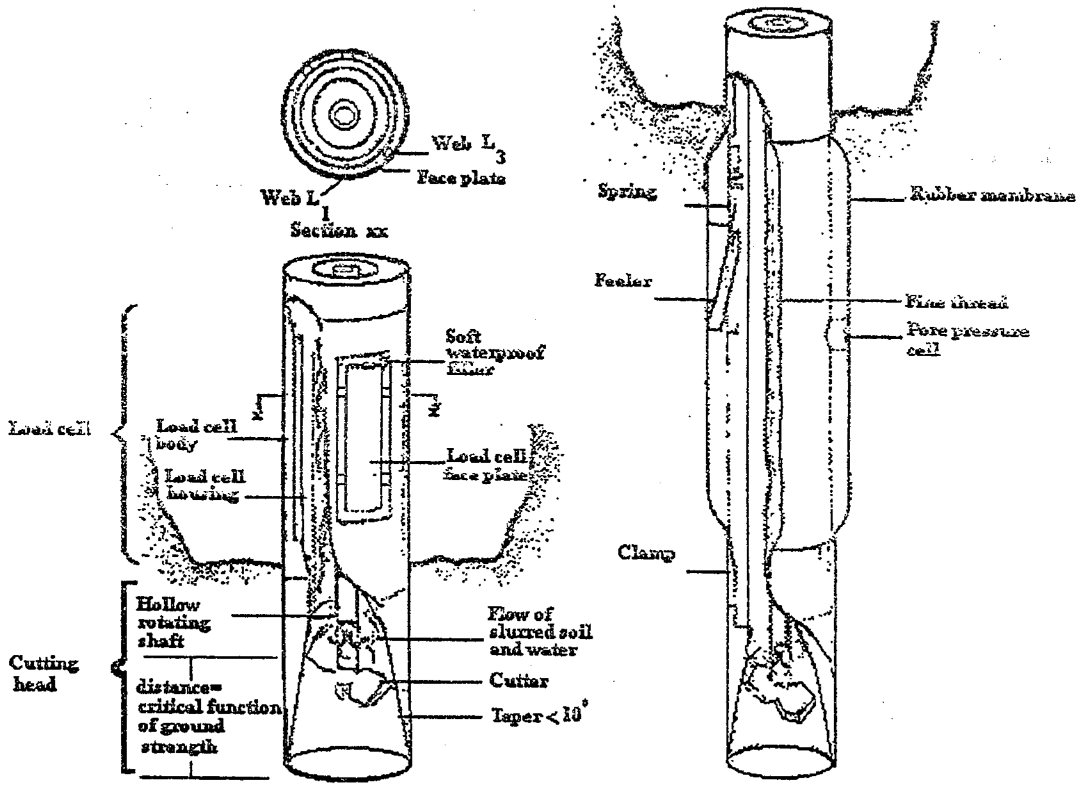


Figure 12. Schematic Diagram os Self Boring Pressuremeter

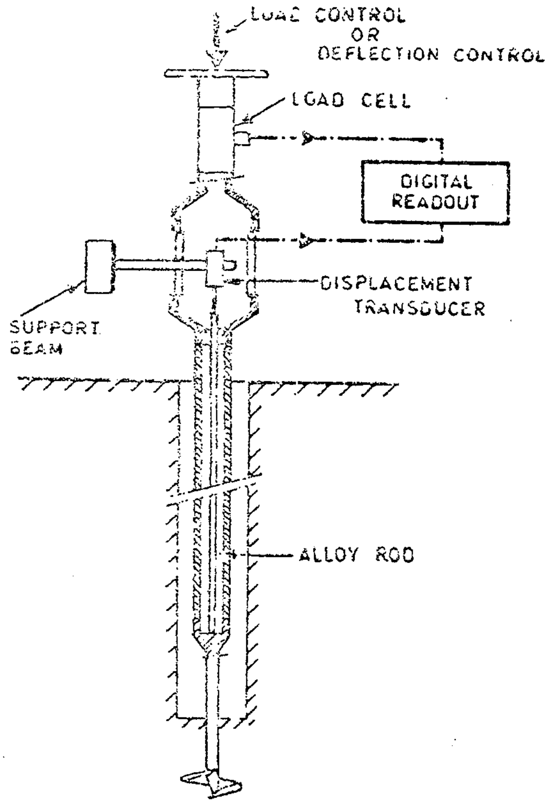


Figure 13(a). Instrumentation Details for test

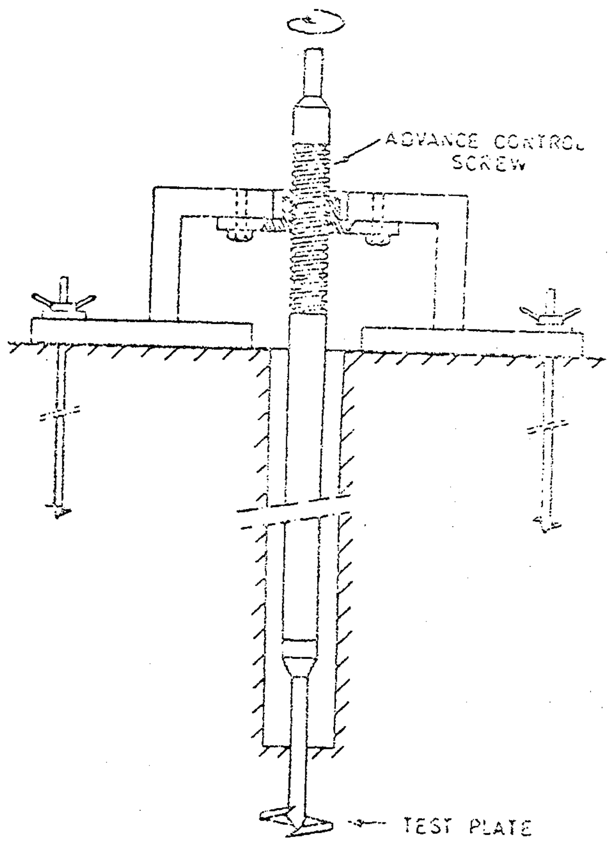


Figure 13(b). Priciple of Installation of Screw Plate in Borehole

SITE INVESTIGATIONS FOR PROJECTS IN RECLAIMED MARSHY LANDS

Dr. Sunil de Silva *Soil Engineering and Deepwells*

1. INTRODUCTION

The proper program of soil investigation for a given project depends on the type of project, the importance of the project and the nature of the sub soils involved. The detection of the soil profile is an essential step in almost all soil mechanics problems. The Engineer must give thought to how the properties of the soil might change, and predict how the foundation would behave during the life of his structure.

There are many parts of the world where good building sites are no longer available close to major cities where other infrastructure facilities are readily available. This is particularly true of harbour and terminal facilities which obviously need to be on the water front.

Similarly Colombo was becoming congested and therefore low lying lands on the outskirts of the City of Colombo had to be reclaimed systematically with the public sector involvement. Once reclaimed these lands were made available for the private sector for development. The author had the opportunity of being involved with two such major projects in Sri Lanka, largest in the recent history of Sri Lanka in soil engineering point of view.

First one was the construction of a warehouse complex at Peliyagoda developed by Clough Engineering group of Australia, for the Urban Development Authority , Sri Lanka in 1992/1993 and the second one was the Shell LPG Terminal Site at Muthurajawela, for shell Terminal Lanka Limited, Sri Lanka, in 1996.

Both these projects can be considered as very important in geotechnical engineering point of view. The Author through this paper attempts to share the experience he had gathered in formulating detailed site investigations, laboratory testing, and instrumentation programmes to cater different needs of these clients utilising limited facilities available in Sri Lanka.

2. SITE INVESTIGATION FOR WAREHOUSE COMPLEX AT PELIYAGODA

As far as the warehouse complex at Peliyagoda was concerned a preliminary soils investigation had already been carried out prior to our involvement. According to the preliminary soils investigation, the sub soil conditions were reported to be fairly heterogeneous. Further, the Client was committed to improve the ground conditions by preloading in order to minimise post construction settlement of the ground.

The Clients intention was to provide pile foundations for the main structure and to provide flexible floors for the warehouses. It had, therefore, been decided to improve the ground conditions by preloading in order to minimise the post construction settlement of the floors which otherwise would have been excessive, exceeding very much over tolerable limits. Thus, the site investigation had to be planned to obtain geotechnical data required for the design of preloading and also for the design of piles.

Since, improving the ground conditions with preloading by placing an earthfill was going to be involved, the Geotechnical Engineer was expected to determine following ;

- Height of the fill
- Rate at which the fill should be raised
- The maximum slopes fo the fill
- The necessity or not of employment of special techniques to contain and / or drain the soft foundation soil
- The anticipated settlement of the fill.
- The predicted time settlement curves.
- The period of time the surcharge should be left in place.
- The stability of the fill and its slopes adjacent to surrounding canal.

The Author has had previous experience in the design and construction of earth embankments on soft ground in down south of Sri Lanka. Comparison of actual settlements measured in situ and theoretically predicted settlements, adopting one dimensional consolidation theory has shown that the actual settlement takes place much faster than the theoretical predictions. Bearing this in mind, settlement plates were installed in order to make time versus actual settlement observations. This would enable the Engineer to determine the degree of consolidation that has taken place at any given time during and even after construction.

2.1 WORK DONE

A total of 37 bore holes have been advanced, 21 detailed bore holes and 16 exploratory ones covering the three lots. The locations of the bore holes are shown in Figure I.

All the bore holes were terminated on encountering bed rock. Standard Penetration Tests were conducted inside the detailed bore holes at regular intervals. Undisturbed soil samples have been collected from highly compressible soft layers. Following laboratory tests have been performed on the undisturbed soil sample.

Natural Moisture Content
Specific Gravity
Wet Density
Dry Density
Natural Vioids Ratio
Atterberg Limits
Grain Size analysis
Un confined compression Test
Triaxial Compression Test
Consolidation Test

In addition to above, the organic content, sulphate content , chloride content and soil pH value have been measured.

Few open stand pipe type piezometers have been installed in order to check whether there will be further subsidence of ground due to the ancient earthfill. These were useful in monitoring development of pore pressures during the earth filling period and the dissipation of pore water pressures under the surcharge.

Details pertaining only to **Lot 1** are discussed in this paper. It had been planned to construct a 75 m x 120 m warehouse in this Lot.

2.2 OBSERVATIONS & DISCUSSION

The site had been part of the major flood plain of Kelani River long time ago. It has been reclaimed a few years ago under Peliyagoda Integrated Development Project, by filling with lateritic earth. The thickness of this fill varies between around 0.80 m and 2.80 m in Lot 1 area. Comparison of initial measurements of piezometric heads and the ground water levels revealed that the primary consolidation due to this fill had already taken place.

The ground water level in the area is generally quite high.

Complex interlayered soils were encountered. The fill is resting on a soft compressible clay / organic clay / peat layer. The thickness of all the layers encountered varies from place to place exhibiting the heterogeneity of subsoil condition. However, sub soil conditions are better than that of Lot 3 (A typical cross section is shown in Figure II) as basically only one thick highly compressible layer is present across this area except for some thin highly compressible layers encountered in some of the bore holes. Presence of sand lenses has also been observed. Since this thick highly compressible layer is often sandwiched between the fill and a sand layer, consolidation should take place much faster than in the case of Lot 3.

It should be mentioned, however, that the spatial variation of the thickness of this compressible layer would lead to uneven finished surfaces.

COMPRESSIBLE LAYERS - Lot 1

Bore Hole No.	Compressible Layers
BH 11	0.92-3.00 m
	3.00-6.00 m
	6.50-7.50 m
BH 12	2.45-4.00 m
	4.00-5.50 m
	5.95-7.00 m
	10.45-18.35m
BH 13	1.45-3.15 m
	3.55-5.50 m
BH 15	2.35-5.65 m
BH 16	0.80-7.50 m
BH 28	0.80-4.50 m
	15.25-18.10m
BH 29	1.15-3.10 m
BH 30	1.50-5.90 m
BH 33	2.82-6.70 m

Depths and thicknesses of different compressible layers encountered in all the bore holes are summarized above for easy reference.

Undrained shear strengths obtained from UU Triaxial tests varied between 18.0 kN/m^2 and 30.0 kN/m^2 while the **Compression Index** obtained from the Oedometer test varied between 0.45 and 3.66.

The consolidation settlement due to a design load of 30 kN/m^2 has been calculated for soil profiles encountered in each bore hole, after taking the height of fill required to raise the ground level to a desired floor level also into consideration.

Following Table shows the predicted total settlement under the design load of 30 kN/m^2, time for 90% consolidating under design load, and the preloading time required under different preloads for achieving 100% of the consolidation settlement due to design load.

SETTLEMENTS AND PRELOADING TIME - Lot 1

Bore Hole No.	Settlement (mm)	Time (t_{90}) yrs	Preloading Time yrs	
			150% surcharge	200 surcharge
BH 11	654	1.9	1.6	1.4
BH 12	492	14.7	11.7	8.0
BH 13	326	0.8	0.7	0.6
BH 15	390	0.3	0.2	0.1
BH 16	1689	1.4	1.2	0.9
BH 28	646	2.9	2.7	2.0
BH 29	370	0.8	0.8	0.8
BH 30	1162	0.5	0.5	0.4
BH 33	861	1.3	1.2	0.9

All these values were theoretical predictions. It has been observed in Sri Lanka that the time required for primary consolidation in organic clays and organic silts are much shorter than theoretical predictions. Therefore, it was strongly recommended to expedite construction of a trial embankment to observe the actual behavior of the fill. It appeared that in some areas; especially, BH 12 , BH 28; another technique will have to be adopted to accelerate the primary consolidation.

It should also be noted that in the case of preloading the height of the fill (surcharge) should be such that at the end of the preloading period a minimum of earth equivalent approximately to the design load could be removed without affecting the design ground level. In other words the settlement due to surcharge too will have to be taken into consideration in the design of preload.

It has also been proposed to provide pile foundations to support the main structure. If driven or bored piles are installed in compressible soil or any soil showing appreciable consolidation under its own weight or due to an imposed load, a load additional to the working load on the head of pile is transmitted in skin friction, i.e. **Negative Skin Friction** or **Drag-down** , to the pile surface. Negative skin friction must be allowed for when considering the safety factor on the ultimate carrying capacity of the pile.

The Table below gives negative skin friction estimated for a 20 m long, 350 mm x 350 mm square reinforced concrete pile, based on soil profiles at each bore hole.

ESTIMATION OF NEGATIVE SKIN FRICTION - Lot 1

Following estimations are based on a pile section of 350 mm x 350 mm. Computations were done for bore hole locations.

Bore Hole No.	Estimated Negative Skin Friction (kN)
BH 11	230
BH 12	800
BH 13	195
BH 15	200
BH 16	210
BH 28	1020
BH 29	125
BH 30	200
BH 33	250

Therefore the negative skin friction for a pile ranges from 125 kN to 1020 kN.

The piles should be founded at least on hard weathered rock. If driven, precast piles should be driven to refusal. At some places traversing the loose to medium dense layer of coarse sand mixed with pebbles would be difficult as pile driving may densify this layer.

3. SITE INVESTIGATION FOR SHELL LPG TERMINAL SITE AT MUTHURAJAWELA

Shell Terminal Lanka Limited was planning to install a bulk storage and distribution terminal for LPG at Muthurajawela, an Industrial Zone, around 7 km North of the Port of Colombo. This site forms part of a recently reclaimed area of 160 hectares. It is covered with around 2.0 m to 3.5 m thick layer of marine sands. This sand had been dredged from the sea bed off the coast and pumped ashore to reclaim Muthurajawela low lying marshes in the first half of 1995.

A preliminary soil investigation has been carried out in July 1995 in order to determine the general nature of the sub soil, the depth to bed rock and the quality of bed rock etc. It consisted of drilling only 5 Nos bore holes.

The preliminary soils investigation revealed the presence of a highly compressible soft organic clay / peat layer underlying the marine sand fill. Accordingly, the client had decided to provide pile foundations for the proposed structures. Therefore, the detailed soils investigation was planned to collect data for the design of pile foundations, prediction of future subsidence of the sandfill, etc.

Since pile foundations were involved the availability or not of aggressive chemicals in soils as well as in ground water too was checked.

In brief, the geotechnical engineer was expected to provide following information.

- Required foundation depths of piles
- Allowable vertical pile loads and related pile sections, for single piles and pile groups
- Expected negative skin friction
- Expected horizontal displacements with respect to horizontal loads.
- Allowable bearing capacity of shallow spread and raft foundations and corresponding elastic and plastic settlements.
- Expected settlement of the ground
- Depth of water table
- Permeability
- Availability or not of aggressive chemicals

3.1 WORK DONE

A total of 25 Nos. bore holes have been advanced at the proposed site. Bore hole locations are shown in Figure III. All the bore holes have been terminated after advancing around 3.00 metres into bed rock.

As in the case of Peliyagoda SPT were conducted at regular intervals, undisturbed soil samples have been collected in soft highly compressible soil layers. Laboratory tests have been performed on these undisturbed soil samples. Field permeability tests have also been conducted inside the bore holes.

The highly compressible layer must be still undergoing consolidation due to the weight of the sand fill. Therefore, three open standpipe type piezometers have been installed to measure the pore water pressure around the center of this compressible layer. This enabled us to estimate the excess pore water pressure within these compressible layers, hence, to predict further subsidence of the ground.

3.2 OBSERVATIONS AND DISCUSSIONS

The thickness of the sand layer varied from 0.92 m to 3.60 m .

Underlying this marine sand fill there is a thick layer of very soft to soft highly compressible organic clay peat layer covering almost the entire site. Its thickness too varies from 0.35m to 5.58m. The depth to bed rock also changes from bore hole to bore hole. These values are tabulated herein for reference.

In one piezometer an excess pore pressure equivalent to around 250 mm of water has been recorded. However, in other two piezometers there has not been appreciable difference in the pore water pressure in the compressible layer and the surrounding ground water level. It implied that only in some parts of the area further subsidence of the ground due to the sandfill will have to be anticipated. Moreover, the thickness of this compressible layer is non-uniform over the investigated area. Therefore , the subsidence of the ground too will not be uniform. However, for design purposes it may be assumed that at least around 90% of the primary consolidation due to the sand fill has already been taken place.

Bore Hole No.	Marine Sand Fill			Compressible Layer			Depth at which bed rock has been encountered (m/GL)
	From (m)	To (m)	Thickness (m)	From (m)	To (m)	Thickness (m)	
BH 101	0.00	2.51	2.51	2.51	2.83	0.32	18.01
BH 102	0.00	2.55	2.55	2.55	2.95	0.40	14.57
BH 103	0.00	2.90	2.90	2.90	5.95	3.05	20.28
BH 104	0.00	3.34	3.34	3.34	7.96	4.62	21.09
BH 105	0.00	1.92	1.92	1.92	3.71	1.79	23.50
BH 106	0.00	0.92	0.92	0.92	6.50	5.58	18.62
BH 107	0.00	2.10	2.10	2.10	6.50	4.40	19.32
BH 108	0.00	3.00	3.00	3.00	6.48	3.48	19.52
BH 109	0.00	2.66	2.66	2.66	4.97	2.31	19.30
BH 110	0.00	2.29	2.29	2.29	3.60	1.31	20.10
BH 111	0.00	2.51	2.51	2.51	5.90	3.39	20.65
BH 112	0.00	2.90	2.90	2.90	5.95	3.05	17.92
BH 113	0.00	2.30	2.30	2.30	3.35	1.05	13.92
BH 114	0.00	2.00	2.00	2.00	2.35	0.35	15.40
BH 115	0.00	2.55	2.55	2.55	3.33	0.78	19.54
BH 116	0.00	2.21	2.21	2.21	3.86	1.65	15.93
BH 117	0.00	2.20	2.20	2.20	2.55	0.30	15.78
BH 118	0.00	3.60	3.60	3.60	6.10	2.50	20.05
BH 119	0.00	2.55	2.55	2.55	3.81	1.26	16.20
BH 120	0.00	2.15	2.15	2.15	2.75	0.60	18.65
BH 121	0.00	2.41	2.41	2.41	3.73	1.32	18.45
BH 122	0.00	2.35	2.35	2.35	4.25	1.90	17.75
BH 123	0.00	2.47	2.47	2.47	3.85	1.38	19.51
BH 124	0.00	2.35	2.35	2.35	3.94	1.59	19.21
BH 125	0.00	2.00	2.00	-	-	-	16.19

Any additional load or a surcharge imposed on the sand fill will therefore, lead to increase the excess pore pressures in this peat / organic clay layer. Thus, long term consolidation settlements will have to be anticipated if shallow foundations are adopted for any structure.

The primary consolidation settlement of the ground due to newly applied loads have been estimated using consolidation parameters measured from laboratory consolidation tests.

Recognising above mentioned facts, the Client promptly decided to adopt pile foundations to support proposed structures.

According to the vertical soil profiles bed rock is available from around 13.92 m / GL around 23.50 m / GL. (A typical bore hole log is presented in Figure IV). The bed rock at these depths were observed to be highly weathered to moderately weathered. An allowable end bearing resistance of around 3000 kN/m² to 5000 kN/m² can be mobilised for piles in this type of highly to moderately weathered biotite gneiss. Therefore, an economical pile design would incorporate provision of end bearing piles going down to bed rock.

Considering the piling depths involved 400 mm * 400 mm square piles and 450 mm * 450 mm square piles appear to be the most appropriate sections for pre cast reinforced concrete piles for driving down to bed rock in this terrain.

If the piles are driven down to weathered bed rock as recommended here, the settlement of the pile toe will be negligible. Therefore, only elastic compression of the piles will have to be considered in settlement calculations.

It is also known that when driven or bored piles are installed in this type of terrain where there is a highly compressible layer, a load additional to the working load applied on the pile head is transmitted along the pile surface as skin friction, i.e. Negative Skin Friction or Drag Down. Therefore, this negative skin friction must be allowed for when considering the factor of safety on the ultimate carrying capacity of the pile or the group of piles. The estimated negative skin friction at each bore hole location is reported in the following Table.

Bore Hole No.	Estimated Negative Skin Friction (kN)
BH 101	60 - 90
BH 102	60 - 90
BH 103	150 - 180
BH 104	200 - 240
BH 105	100 - 150
BH 106	130 - 160
BH 107	120 - 150
BH 108	160 - 200
BH 109	130 - 160
BH 110	80 - 100
BH 111	150 - 175
BH 112	150 - 180
BH 113	80 - 100
BH 114	60 - 80
BH 115	80 - 100
BH 116	80 - 100
BH 117	60 - 90
BH 118	130 - 160
BH 119	80 - 100
BH 120	200 - 240
BH 121	160 - 200
BH 122	120 - 150
BH 123	80 - 100
BH 124	60 - 90
BH 125	0 - 60

After allowing for above mentioned negative skin friction, subject to comments and recommendations made in this Report, 400 mm x 400 mm square pre cast reinforced concrete piles should be able to carry a safe axial load of 750 kN to 900 kN in compression when driven down to bed rock in this type of terrain.

ACKNOWLEDGEMENT

The Author wishes to thank Clough Engineering Group of Australia and Shell Terminal Lanka Limited of Sri Lanka for granting permission to publish data relevant to the projects executed by them in Sri Lanka.

FIGURE I

WAREHOUSE COMPLEX - PELIYAGODA

BORE HOLE LOCATIONS

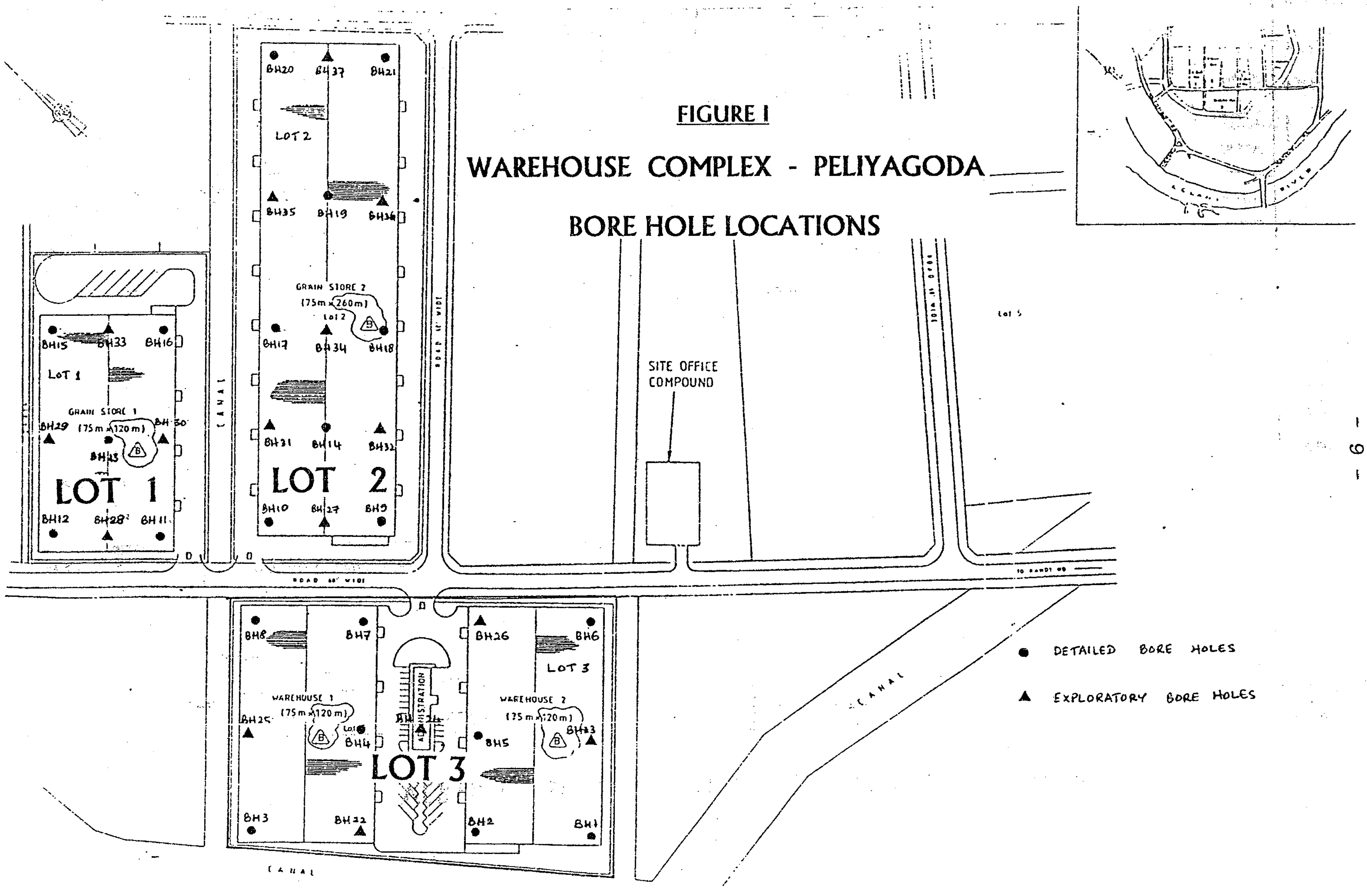


FIGURE II

VERTICAL SECTION THROUGH BH 3 - BH 25 - BH 8

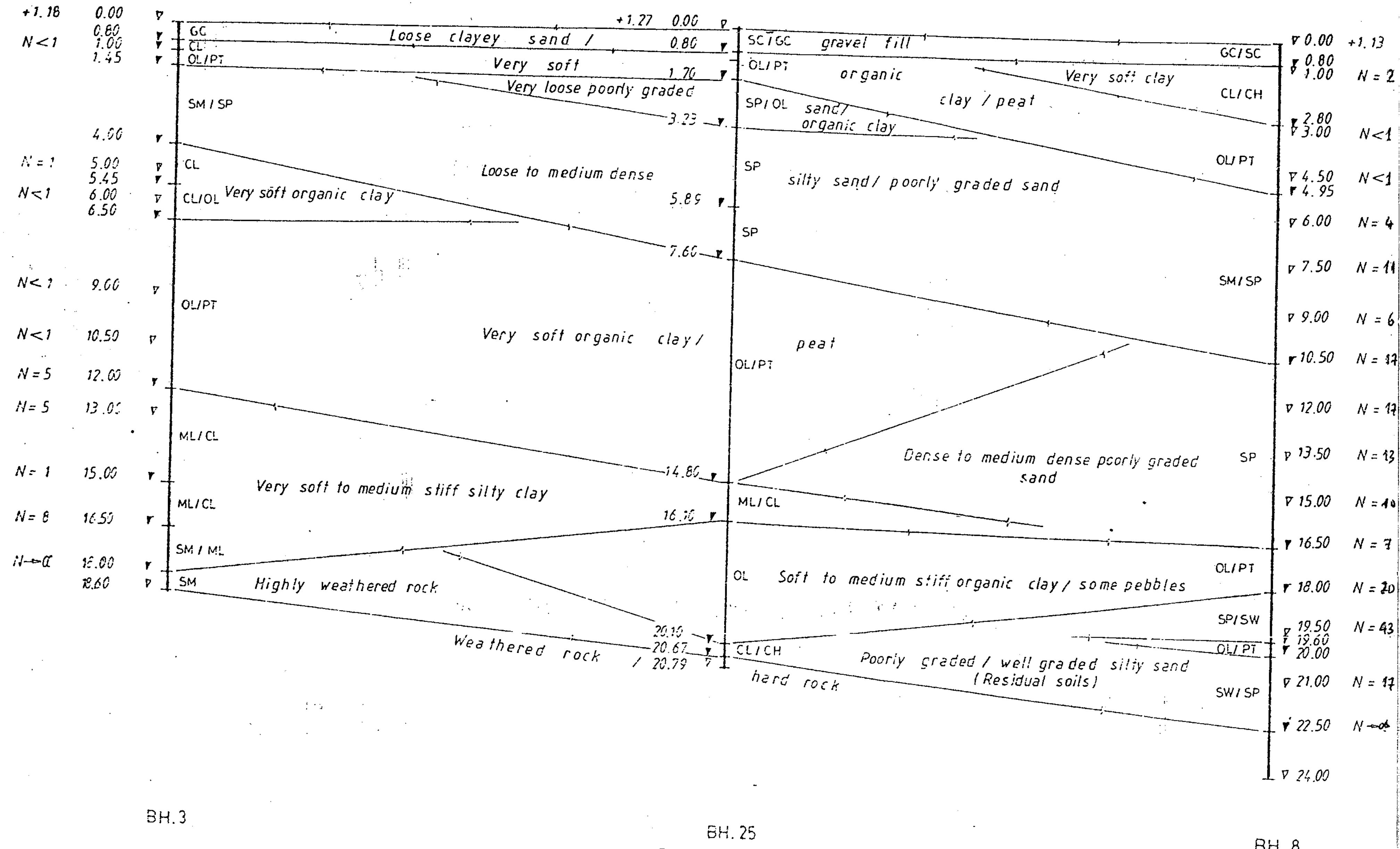


FIGURE III

SHELL LPG TERMINAL SITE

BORE HOLE LOCATIONS

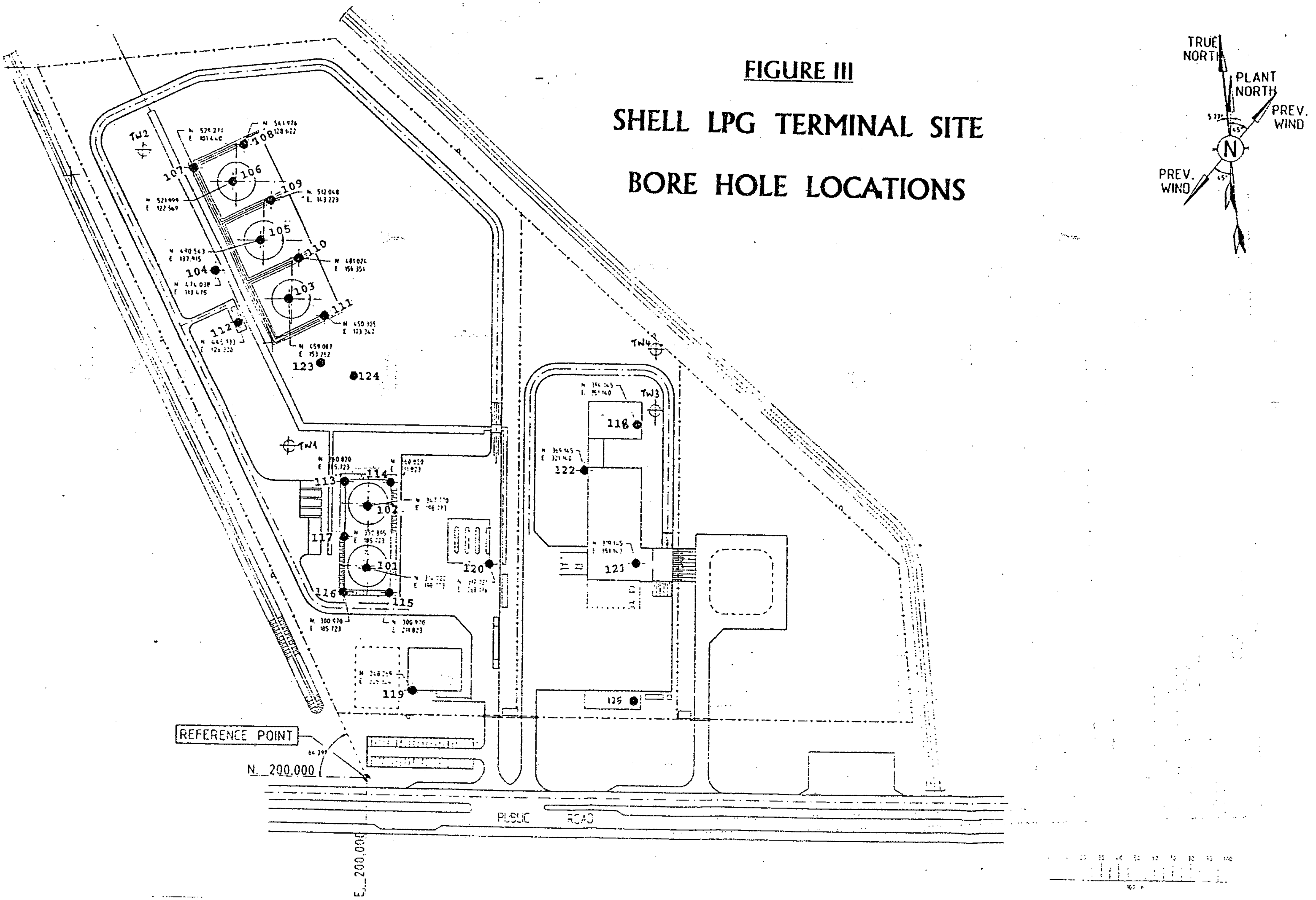


FIGURE IV

GEOLOGICAL RECORD OF BORING

HOLE No. BH - 104

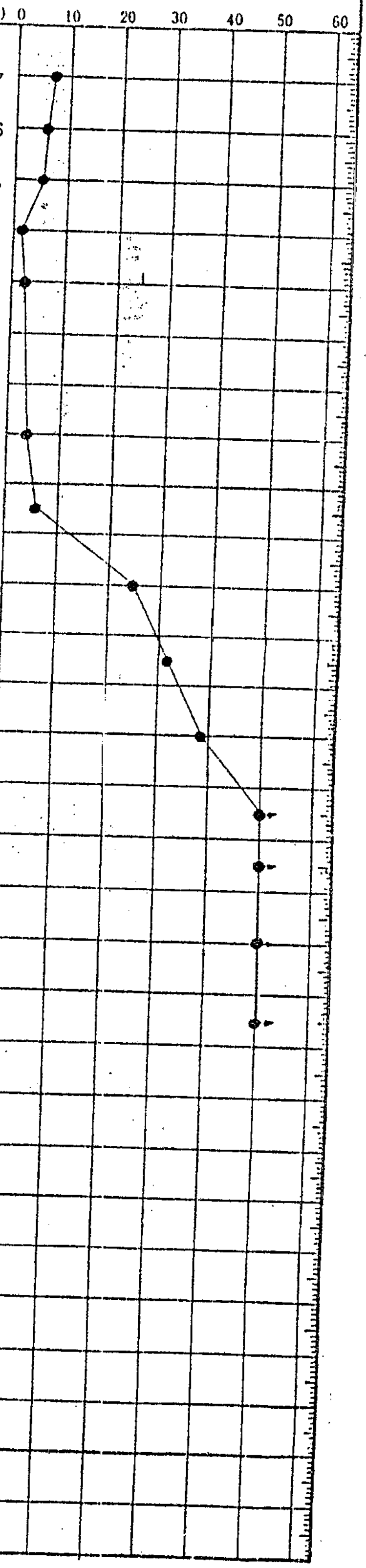
PROJECT	LPG IMPORT TERMINAL - STLL	LOCATION	Muturajawela - Sri Lanka
GROUND ELEVATION	+ 1.545 MSL	DEPTH OF HOLE	24.94m
ANGLE FROM VERTICAL	0	DIAMETER OF HOLE	100mm
MACHINE	NENZI	DATE OF DRILLING	28th April to 02nd May '96
CORE RECOVERY		DEPTH TO GROUND WATER LEVEL IN HOLE	1.46 m below ground level
DRILLED BY	S.K.P. Jayasundera	LOGGED BY	B.S. Yapa

ELEVATION (m)	DEPTH (m)	THICKNESS (m)	COLUMN SECTION	SOIL OR ROCK CLASSIFICATION	COLOUR	DESCRIPTION	CORE RECOVERY %	CORE RECOVERY cm
	3.34	3.34		SP	Yellowish Brown / Greyish Brown	LOOSE POORLY GRADED SANDS		
	7.96	4.62	6.50 U/D 7.50	Pt	Blackish Brown	VERY SOFT FULLY DECOMPOSED ORGANIC MATTERS		
	16.02	8.06		ML	Greyish Brown	SOFT TO HARD CLAYEY SILTS		
	19.50	3.48		ML/SM	Greyish Brown	HARD TO EXTREMELY DENSE CLAYEY SILTS/SILTY SANDS		
	21.09	1.59		SM	Greyish Brown	EXTREMELY DENSE SILTY SANDS		
	22.59	1.50		HIGHLY WEATHERED ROCK		CORE RECOVERY=45.3% R.Q.D. = 0%		
	24.09	1.50		HIGHLY TO MODERATELY WEATHERED		CORE RECOVERY=65.3% R.Q.D. = 0.0%		
	24.94	0.85		BIOTITE GNEISS		CORE RECOVERY=70.5% R.Q.D. = 0.0%		

BORE HOLE TERMINATED AT 24.94m BELOW GROUND LEVEL

STANDARD PENETRATION TEST

DEPTH (m)	NUMBER OF BLOWS (N)
1.0	07
2.0	06
3.0	05
4.0	01
5.0	02
8.0	03
9.5	06
11.0	24
12.5	31
14.0	39
15.5	50/29cm
16.5	50/10cm
18.0	50/8cm
19.5	50/6cm



A CRITICAL ASSESSMENT OF SITE INVESTIGATION PRACTICES FOR BUILDINGS IN LOW LYING AREAS

by

Prof. B. L. Tennekoon, Miss. H.A.Y. Chamindani and Miss. K.A. Madunishanthi

1.0 INTRODUCTION

A study of the site investigation practices for buildings in low lying areas in this country shows that

(a) the extent and costs of investigations have varied considerably;
(b) the investigations whilst being adequate in some cases, have in other cases failed to identify problems that were later encountered.

Making use of several case histories, this paper attempts to provide recommendations for site investigations in low lying areas that would be appropriate, adequate and economical.

2.0 SOIL FORMATIONS IN LOW LYING AREAS

Tennekoon et al. (1988) report that when geotechnical maps are prepared for making recommendations for the design and construction of foundations of buildings, much advantages could be had by describing the soils according to their genesis. In this respect, the low lying areas of Colombo were shown to contain both bog soils and alluvial deposits which are underlain by residual soils.

Bogs are regions of water logged soils where oxygen is scarce. Under such conditions, the anaerobic micro-organisms decompose organic matter (such as vegetation which get covered by water bodies due to minor oscillations of sea level in geologically recent times). One of the products of such decomposition is peat.

The alluvial deposits consist of a variety of soils found in river valleys and flood plains. These soils have been deposited during the flooding season. They generally vary between silty sands and sandy clays.

Of these two formations, the peaty soils are the most problematic when building in low lying areas on account of their high compressibility and low shear strength.

The residual soils which underly these soil types consist of the in-situ weathered product of the parent rock lying beneath.

3.0 SITE INVESTIGATION PRACTICES AT INITIAL PROJECT CONCEPTION STAGE

When large extents of low lying areas are to be developed for building purposes, a site investigation is undertaken to determine the methods and feasibility of developing the site. Therefore, the site investigation should provide information on

(i) the extent of the low lying areas (as often such areas are found as valleys between lateritic hills);

(ii) the nature and thickness of the weak sub-surface deposits.

The study of several case histories show that such information can be very economically obtained using a combination of boreholes and the less expensive Mackintosh or other Probing Tests.

3.1 <u>Sites consisting of valleys which are low lying and between hills</u>

When sites consist of valleys which are low lying and found between lateritic hills, the site investigation can be used to map the variations in soil conditions from the lateritic hill to the low lying area.

3.1.1 The site investigation for a factory building at Wattala in a land which had an extent of more than 1.25 acres consisted of only 2 boreholes which were supplemented with 17 Mackintosh tests. From this investigation, it was possible to locate the factory building within reasonably good ground conditions without having to undertake costly ground improvement work.

3.1.2 A similar site investigation was carried out at a 34 acre block of land at Battaramulla where it was envisaged to set up a housing estate. The initial investigations consisted of advancing 3 boreholes supplemented by 12 Mackintosh tests to evaluate the feasibility of developing the site.

3.2 <u>Sites consisting only of large flat low lying areas</u>

There are other instances in which the site may be entirely low lying; and the proposed development plan would then depend on the thickness and position of peat deposits which may be present.

3.2.1 A 13 acre block of land at Hendala, Wattala was to be developed for the construction of a modern container yard and freight station. The site investigation consisted of advancing 3 boreholes supplemented by 14 Mackintosh tests.

The investigation showed the existence of a peat layer at the surface within the marsh, and the thickness of this layer was greater than 10 m at over more than 80% of the site. Settlements of the order of 250 cm were anticipated as a result of filling the marsh to the statutory elevation required to prevent flooding. Further, it was found that unless special ground treatment methods were used to accelerate these settlements, the latter would take place over a period of about 6 years which was not acceptable to the client. Ground improvement methods in the form of installation of vertical drains or using lime columns (to accelerate the settlement process so that the project could be completed on time) were found to be too expensive for the viability of the project. As a result of this investigation during the Project Conception stage, the site was abandoned and the search for an alternate site commenced.

3.2.2 A housing development project at Madiwela consisted of the construction of 340 units of two-storeyed apartment blocks in a 13.8 acre block of land. In order to prevent flooding, the ground elevation of the site had to be raised by about 90 cm.

The project was to be implemented in 2 stages. The field investigations for the first stage consisted of advancing 2 boreholes supplemented with 10 Mackintosh tests. These investigations showed that the surface soils consisted of about (3.0 - 3.5)m of weak peaty soil followed by (1.0-1.5)m of soft organic clay.

Estimates for settlements showed that as a result of the filling there would be primary consolidation settlements in excess of 50 cm. Further, additional (10-20)% of secondary settlements were to be expected in the peaty soils.

From the results that were obtained from this site investigation, the Client indicated a desire to use a ground improvement method in which the settlements were over quickly. Without any further investigations, taking into consideration that

(i) the layers of peat and soft clay were limited in thickness and were present at the surface; and

(ii) the site was located away from built up areas so that vibrations would not be a problem;

the Contractor decided to adopt the Heavy Tamping Method for ground improvement.

4.0 SITE INVESTIGATIONS FOR FINALISED DESIGNS

When finalising the designs of foundations for buildings. often the choice between shallow foundations and pile foundations is decided on by

(i) the settlements that would be anticipated if shallow foundations were to be used. (This would include the settlements due to filling as well.)

(ii) the magnitude of the vertical stresses being imposed on the peat or alluvial deposits. (These should be less than their allowable bearing capacities)

If the settlements and the stresses can be controlled to lie within their limits, then shallow foundations are found to be feasible. If not, pile foundations have to be used.

4.1 Sites where shallow foundations are envisaged

Most low lying areas require to be filled in order to

(i) allow for mobility of construction equipment;
(ii) raise the ground level to an elevation higher than the maximum anticipated flood level.

Often this filling is carried out using a lateritic gravel.

4.1.1 A study of the recommendations made in Site Investigation Reports show that sites which contain a lateritic fill have often been found to be suitable for supporting 2 to 3 - storeyed structures provided that

(i) there is sufficient thickness of lateritic fill below proposed foundation level to reduce the foundation stresses sufficiently to those allowable on the alluvial deposits or the peat, as the case may be;

(ii) sufficient time has been allowed between the end of filling and commencement of building activities to ensure that most of the settlements due to the filling are complete;

(iii) the lateritic fill has been compacted under controlled conditions; and

(iv) the settlements due to hard core filling within the plinth area of the building and/or the ground floor loadings are not excessive.

Often the lateritic fill cannot be penetrated by the Mackintosh probe, and therefore, settlement estimates are possible only when boreholes have been used to penetrate the weak compressible layers.

The number of boreholes used have depended on

(a) the extent of the land;
(b) the availability of other information in the vicinity; and
(c) the estimated cost of the building

4.1.2 When a limited thickness of peat or other very weak alluvial deposit is found near the ground surface, its removal and replacement using good quality fill material is a well established method of ground treatment. The studies show that in the low lying areas, the water table being near the surface, the replacement material used below ground water level has usually consisted of sand or quarry dust. Further, this method of soil replacement is often restricted to the upper 3m from construction considerations.

When shallow foundations are envisaged in low lying areas, the Site Investigation Report should also indicate the manner in which a site should be developed to avoid subsequent problems. Two examples of such problems are given in this section.

In the first example, a 2-acre site at Borella was developed by the removal of as much of the peat as possible mechanically using an excavator, and then backfilling with quarry dust to just above ground water level. The quarry dust filling was followed with a filling of compacted lateritic gravel of thickness 2.2 m. The entire filling operation commenced from the side of the road which formed one boundary and proceeded towards the boundary at the rear beyond which was a canal. Several months after the filling, just after a period of very heavy rain, several deep vertical cracks were observed within the site. The investigations undertaken at this stage showed that it had not been possible to remove the entire thickness of peat. Therefore, after studying the crack pattern, it was postulated that if the filling of the site had been done too rapidly, then there was the possibility of having formed a failure surface running along the peat layer; and when there was an increased load on the ground surface caused by the saturation of the fill after the rains, then movements could take place along this failure surface with vertical cracks opening up at different locations in the site. Making use of this hypothesis, the site was subsequently stabilised by driving sheet piles along the canal boundary to intersect the assumed failure surface. The lesson to be learnt from this example was that since peat is a very weak material, the rate of filling should be carefully controlled so as to not cause any shear failure within it.

In the second example, another housing site which was several acres in extent was investigated after the soil replacement was carried out. At this site, the drainage canal ran through the centre of the site. In the site investigation carried out, the housing estate was divided into 3 areas, and one borehole only was advanced in each of these areas. Two-storeyed houses were then constructed, and it was found that whilst the houses in 2 out of the 3 areas did not show any foundation problems, in the third area some very serious foundation problems occurred as a result of which some houses had to be even demolished. Subsequent investigations within this area showed that damage to the houses had been caused by

(i) large compression of a very thick peat layer which had not been detected during the original site investigation;

(ii) excavation and widening of the canal running through the site which had led to unloading of the toe of the canal slope.

The conclusion arrived at from this example is that these problems could have been avoided if

(a) a greater number of boreholes had been used; and
(b) the borehole positions were better planned.

4.2 Sites where pile foundations are envisaged

When pile foundations are envisaged in low lying areas, the objectives of the site investigation programme should include the determination of

(i) the depth to rock on which the piles could be founded; and
(ii) the magnitude of the negative skin friction loads that could arise as a result of the

settlement of peat and/or other alluvial deposits relative to the pile.

4.2.1 The case history given in this section concerns a site where bored and cast in-situ RC piles were to be used to support the foundation loads of 4-storeyed blocks of flats in Colombo. The site was over one acre in extent but only 2 boreholes had been advanced during the site investigation. The depth to rock at these two locations were 10.95 m and 11.75 m. However, when actually constructing the piles, it was found that the depth to rock was quite variable and that some of the piles were founded as deep as 25.0 m. Plotting the bed rock contours showed that at the location of the deepest pile, the rock was dipping at an angle as much as 59.2^{o} to the horizontal.

Clearly, the number of boreholes advanced at this site is thoroughly inadequate. However, it should be pointed out that even with a better planned Site Investigation programme, there is a limit to the number of boreholes that could be considered economical. Therefore, as recommended in Geoguide 2 (1989) published by the Geotechnical Control Office, Hong Kong, the Site Investigation programme carried out during the Project Conception Stage and the Finalised Design Stage should always be supplemented by recording the actual ground conditions during construction stage. This would enable the engineer to re-evaluate the foundations and re-design them, if necessary.

4.2.2 The second case history given in this section concerns the failure of RC piles used for a warehouse complex at Wattala.

The initial Site Investigation of 2 boreholes advanced at the site showed that the sub-surface conditions at the time of investigation consisted of about 4m of lateritic fill underlain by about 12m of very weak and highly compressible organic clay and peat. After installing the piles, it was observed that the ground level had subsided considerably as a result of the compression of the organic clay and peat due to the filling. As a result, a further 1.2m of filling was then done.

Soon after the buildings had been constructed, it was noticed that there was distress to the structure. Further studies showed that the distress arose out of failure of some of the piles. Many reasons were attributed to the pile failure, one of them being that the negative skin friction loads had been under-estimated.

5.0 LABORATORY INVESTIGATIONS

A study of several site investigation reports showed that the laboratory investigations that have been undertaken have been limited to

(i) determination of properties such as particle size distribution, Atterberg Limits, moisture content, specific gravity, organic content, etc. on disturbed samples; and

(ii) determination of shear strength parameters and consolidation characteristics of deposits in which it had been possible to obtain undisturbed samples.

In the latter instance, the study showed that the limitations to the laboratory studies lay in the sampling methods that had been used. It was found that many soft organic clays and peats had not been tested for shear strength and consolidation properties because of the inability to recover undisturbed samples. As a method of improving the site investigation practices in this country, the method of obtaining undisturbed samples from boreholes is being recommended to be included in the document on Specifications for Site Investigations for Buildings, currently being drafted; ICTAD (1996). A recommendation that is being considered in this document is that 75 mm diameter (minimum) Piston samplers be used to recover undisturbed samples of peat and soft clays; and the use of Thin Walled samplers would be allowed only if the clays are not too soft and can be retained in the sampling tube.

6.0 RECOMMENDATIONS FOR SITE INVESTIGATIONS IN LOW LYING AREAS

Site Investigations in low lying areas are best carried out using a mix of boreholes and Mackintosh or other probe tests. Specific recommendations regarding the Borehole Investigation are given below.

6.1 Number of Boreholes

The number of boreholes should be decided based on the nature and stage of the investigation. The document published by ICTAD (1994) states that 'the cost of adequate exploration is low when compared with the total value of work, but varies with the type of structure and the nature of the ground; a usual figure being the order of 1 percent of the cost of building. However, it must be understood that the technical requirements of the investigation should be the overriding factor rather than the cost.'

6.2 Positions of Boreholes

When the investigations are for shallow foundations, the boreholes should as far as possible be distributed uniformly acorss the site. If a drainage canal is running through the site or close to a boundary, then at least one of the boreholes should be located close to the canal.

When the investigations are for pile foundations, the boreholes should be located as far as possible close to the pile locations.

6.3 Depth of Boreholes

When the investigations are for shallow foundations, one of the boreholes should be taken down to rock or to a depth of 20 m, whichever is less. The other boreholes can be terminated after penetrating the weak layers encountered within the upper 10 m.

When the investigations are for pile foundations, the boreholes must penetrate (3-5)m into the rock.

6.4 Diameter of Borehole

Since 75 mm diameter (minimum) undisturbed samples would have to be recovered, the diameter of borehole used should be large enough to recover these undisturbed samples.

6.5 Methods of boring in overburden

Rotary drilling methods are preferred as they can penetrate any logs which are sometimes encountered in the sub-surface in low lying areas. They have the additional advantage of causing less disturbance to the soft deposits than the shell and auger method of drilling.

Further, if pile foundations are envisaged, the same rotary machines can be used for coring in the rock.

6.6 Methods of drilling in rock

Rotary drilling is recommended so that

(i) rock could be penetrated by (3-5)m; and
(ii) rock cores could be recovered

6.7 Undisturbed sampling in overburden

The following recommendations are made:

(i) Use of 75 mm diameter Piston Samplers for peat and soft clays.

(ii) Use of 75 mm diameter Thin Walled Samplers can also be allowed in clays which are not too soft so that the samples can be retained in the sampling tubes.

6.8 Sampling of rock cores

The triple tube core barrel is preferred. Alternatively, double tube core barrels which would give a minimum diameter of rock core of 40 mm could be used.

6.9 Ground water levels

Ground water levels are likely to be high and should be observed during the investigation as the level at which the water level stabilises inside the borehole.

7.0 CLOSURE

This paper draws attention to several problems that are usually associated with foundations in low lying areas, and indicates how these could be adequately studied during a site investigation. Some of the existing deficiencies in Site Investigation practices are highlighted, and the paper concludes with some recommendations which might prove useful when undertaking Site Investigations in low lying areas.

8.0 ACKNOWLEDGEMENTS

Thanks ar due to Mr. V. Somaratne for typing the paper.

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SOME HARD REALITIES FACED IN PLANNING MEASURES FOR LANDSLIDE DISASTER MITIGATION THROUGH GEOTECHNICAL INVESTIGATION PROJECTS

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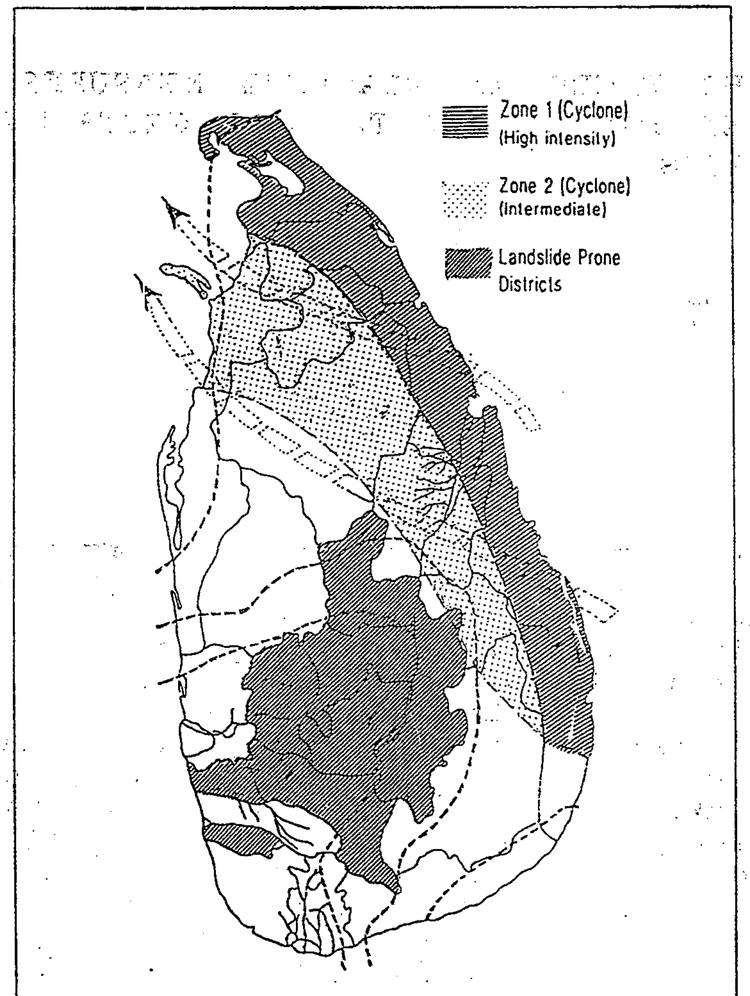
ABSTRACT:

The major events and disasters due to landslides are common in Sri Lanka. Landslide is an extremely complex phenomena, governed by multitude number of factors. This is the reason why our design for counter measures are either conservatives or unreliable. The paper spotlights some of the multifaceted aspects of hard realities confranted by a Geotechnical Engineer during geotechnical investigations, which made planning and implementation of mitigatory measures and landslide remediation project studies difficult in Sri Lanka. It is important that the observed difficulties in implementing mitigatory measures are significantly explained and guidelines for investigation models, their design and methodology for implementation, are discussed based on the experience gained in solving the actual geotechnical problems associated with landslide studies. It is also can be pointed out that engineering fallacies, practical aspects and systems introduced for project control in the atmosphere of changing dimensions allowing flexibility which are deserved to be studied into a greater deal specially in landslide disaster remediation . This paper identifies and looks ahead for the new directions govern by the information obtained through a scientifically oriented geotechnical investigation processes. In the overall investigations, planning, monitoring and management of disaster mitigation measures, more relevant examples can be found in the case of already undertaken studies on landslides and other mass movements.

1. Introduction:

In brief, the geotechnical engineering is not an inexact science, but the science of inexact. Answering the questions associated with the investigation and planning of geotechnical engineering projects in an appropriate manner considered to be the backbone of success of the project. Requirements which demands the knowledge behind the problems related to investigations, monitoring, planning, evaluation and execution are increasing. But the authoritive decisions made on above areas will be subjected to changes due to



Landslide & Cyclone Prone Areas of Sri Lanka

Fig. 1

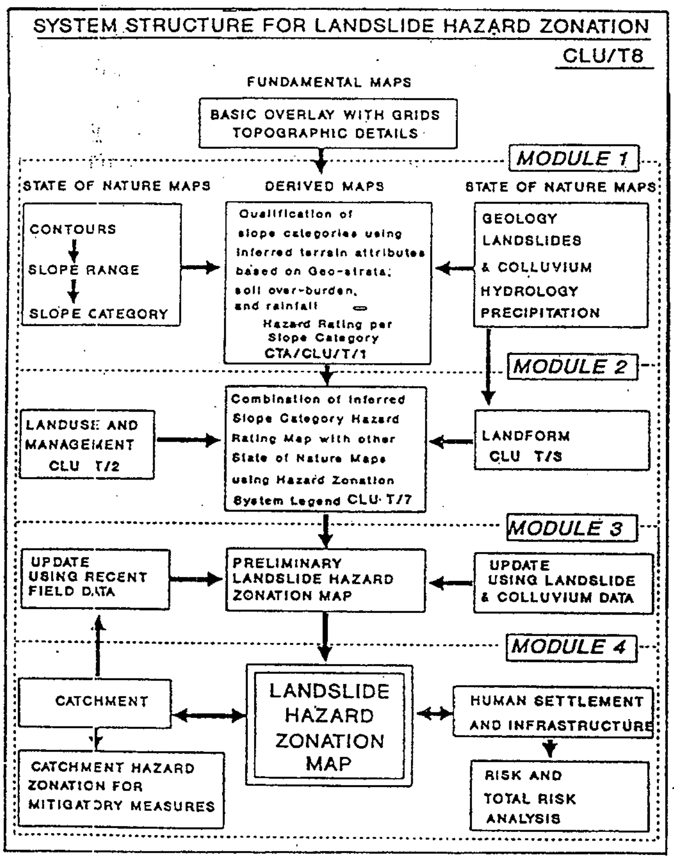


Fig. 2

economical reasons rather than the technological requirements. On the other hand the resource constrains during the planning process of a geotechnical investigation project is very high compared to the process involving in construction industry. Setting up of a realistic estimate on the expenditure on a project require much guess work in the initial stages. The amount of guess work increases if only a little is known about the project to be studied. The technological inputs could be bolstered through launching of a programma on method identification, assessment and selection, in tune with the felt needs and the requirements of the cost effectiveness. Unfortunately, it is in this area that most developing countries like Sri Lanka lack necessary expertise.

2. Landslide Disasters in Sri Lanka

Landslides are very common in the hilly regions and if appropriate measures are not taken in time it will develop in to a serious problem in future in terms of frequency of occurrence, disturbing forces and the extent of damages to the environment and economy. Seven administrative districts of Sri Lanka are prone to landslide and economic losses to infrastructure due to landslides(Fig-1) came under heavy strain in the recent past. According to the estimate Bhandari; 1994 the total economic losses approached Rs. 1.420 billion claiming more than 51 lives and rendering nearly 10,000 families homeless in the year 1989 along.

In order to examine the vulnerability and risk, requirements of geotechnical investigations have to be included in a landslide remediation package after a detail evaluation. Landslide Hazard Zonation Mapping is one of such constructive models based on the research studies, the ultimate aim of which is the sustainable development through natural disaster mitigation. The detail assessment of natural slope stability is needed for the design of mitigatory measures of landslide hazards. But the extent of the problem can be analysed only after scientific studies for examples slope erosion studies, studies of disasters connected with rockfalls and other mass movements, other natural ways of environmental degradations etc. On the other hand, even when the vulnerability is known, the systems introduced for early warning and its implementation strategy are new to the country and therefore some of the significant hidden realities still cause problems. Therefore it delays the achievement of better results.

3. Some Unique Features of Sri Lankan Landslides:

The extensive field investigation models (Fig-2) cum analytical performances of the Landslide Hazard Mapping Project (LHMP) through its outputs reflect unique features of landslide phenomena in Sri Lanka as given below,

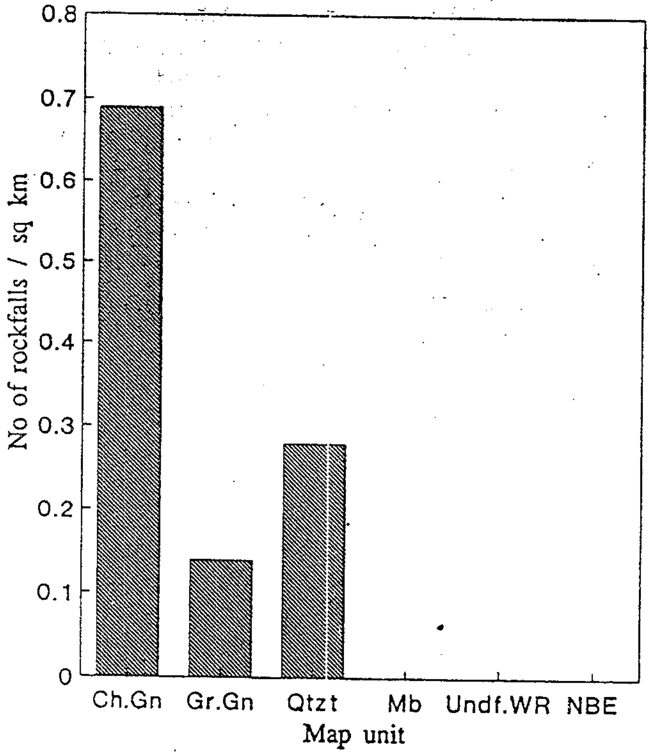


Fig. 3 - **Susceptibility of geological map units per unit area (Rockfalls/ Sq.Km.) to Rockfalls**

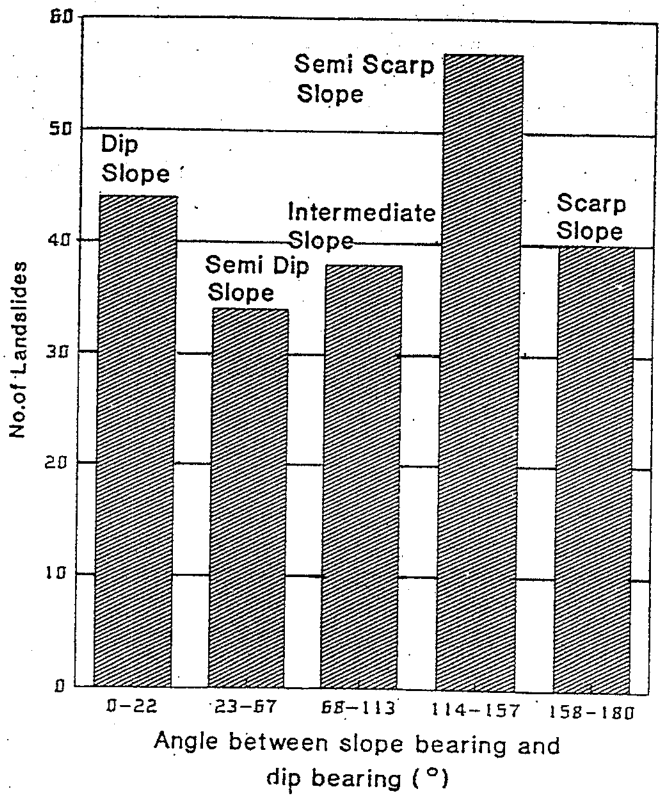


Fig. 4 - Landslides vs dip and scarp slopes

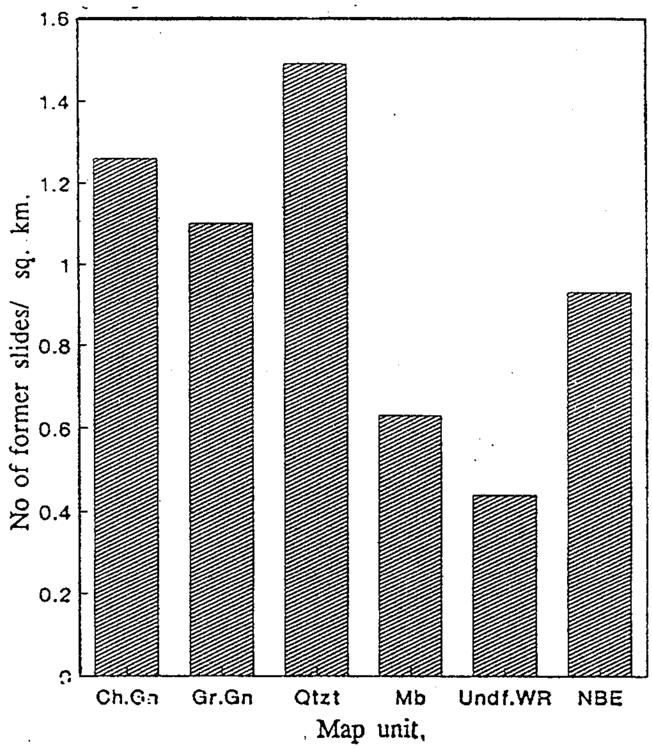


Fig. 5 - **Suceptibility of geological map units, per unit area (Slides/sqkm) to former landslides**

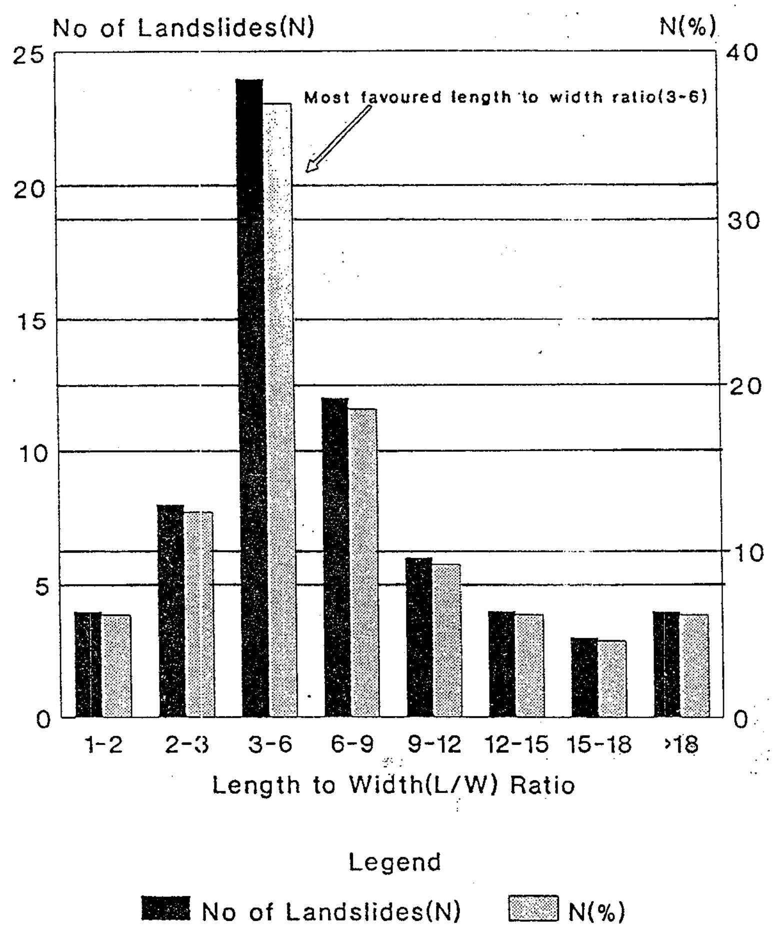


Fig. - 6.1

ANALYSES OF LENGTH TO WIDTH RATIOS OF SRI LANKAN LANDLSIDES MAPPED AT VERY LARGE SCALES (>1:10,000)

(Cases taken from Special Investigation Reports)

LANDSLIDE	TYPE	L-LENGTH (M)	W-WIDTH (M)	L/W
Subodagama (Udwaraslide	Rotational	120	50	2.4
Yelverton Estate	Subsidence	50	50	1
Deekillapotha		400	80	5
Marabedda	Rotational	1650	650	2.538
Kekunagasdawatte	Rotational	250	45	5.555
Soda Mola, Passara	Rotational	150	35	4.285
Kirimatiya	Earthflow/ subsidence	80	60	1.333
Kahagala	Earthslip	600	165	3.636
Beragala	Slope Movement	380	100	3.8
Hilltop Hotel, Kandy	Earthflow	77.77	42	1.85
Beerindewela	(Earthflow) Old slide	570	110	4.64
Lemastota	Debrisflow	1000	70	14.29
Helauda	Earthflow	410	32	12.81
Watagoda Estate	Earthslide	200	180	1.111
Madagala (Nawalapitiya)	Earthflow	50	22	2.272
Watawala	Earthflow	750	75	10
Bulathsinhala (Divulakanda)	Earthflow	60	20	3
Kirklees Group (Badulla)	Debrisflow	200	80	2.5
Eheliyagoda	Debrisflow	190	15	12.6

Fig. - 6.1

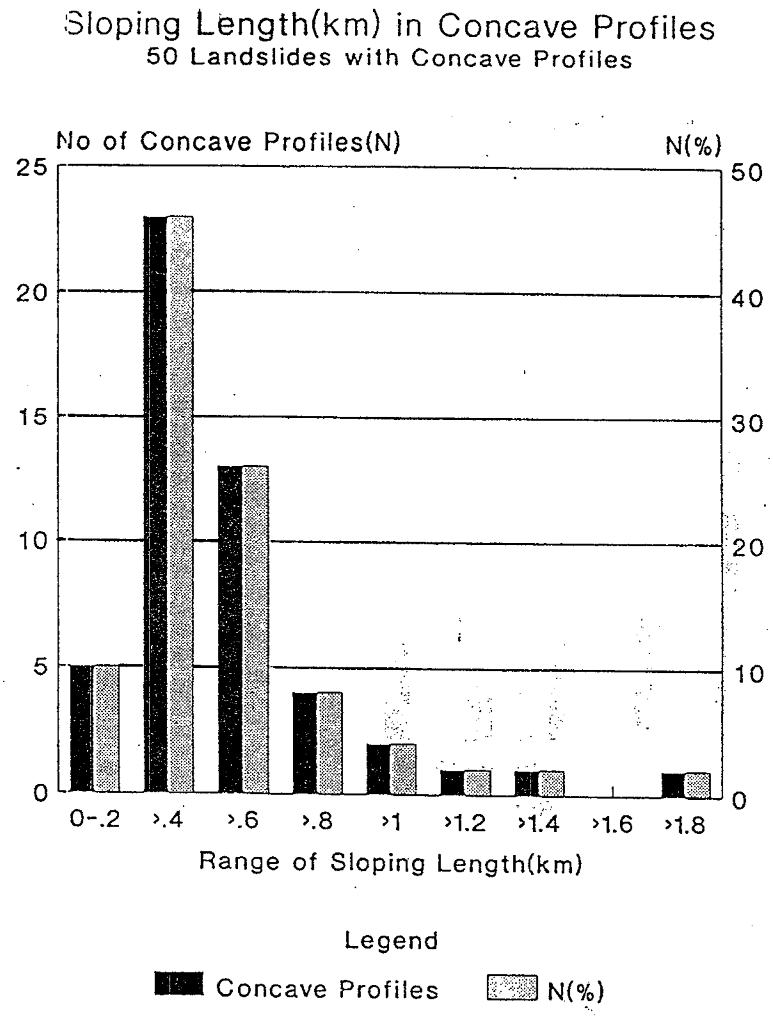

Sloping Length(km) in Concave Profiles
50 Landslides with Concave Profiles
No of Concave Profiles(N)
N(%)
25
20
15
10
5
0
50
40
30
20
10
0
0-.2
>.4
>.6
>.8
>1
>1.2
>1.4
>1.6
>1.8
Range of Sloping Length(km)
Legend
Concave Profiles
N(%)

Fig. - 6.2

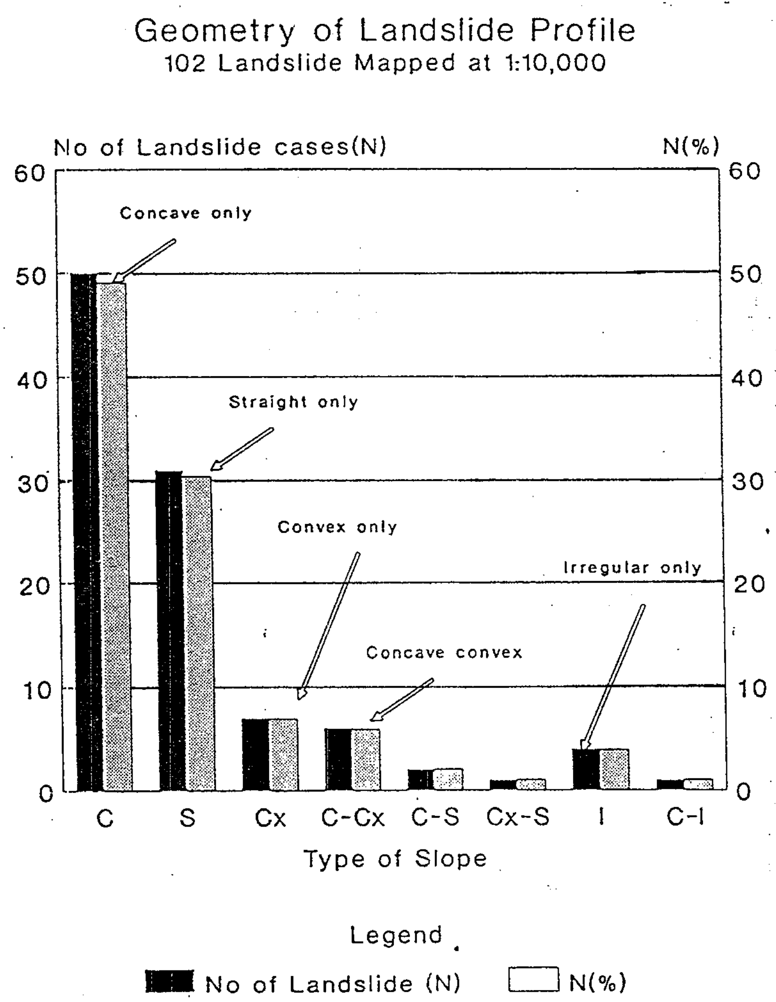

Geometry of Landslide Profile
102 Landslide Mapped at 1:10,000
No of Landslide cases(N)
N(%)
60
50
40
30
20
10
0
Concave only
Straight only
Convex only
Irregular only
Concave convex
C
S
Cx
C-Cx
C-S
Cx-S
I
C-I
Type of Slope
Legend
No of Landslide (N)
N(%)

Fig. - 6.3

i. The frequency of landslides on an average is about one per sq.km. where slope degradation processes are in their advanced stage , it is two per sq.km.

ii. Most landslides occur in soil cover on the slope or in the uppermost weathered crust of the bed rock.

iii. The most hazardous rockfall cliffs consist of Charnockitic lithologies with widely spaced joints (Fig-3) ; e.g. Viharagala, Aranayake, Nawalapitiya and Ratnapura.

iv. Failure of residual soils often involves the upper weathered portion of the bedrock as well. the bedrock usually fails along either joints or foliation surfaces, that dip in the downhill direction. Wedge failures, which also involve joints striking in the downhill direction are often present in such cases.

v. Scarp slopes appear to be more hazardous (Fig-4) than dip slopes because dip slopes in Sri Lanka tend to have only a thin soil cove, while scarp slope often have accumulation of colluvium.

vi. Deep seated landslides involving fresh bedrock are rare.

vii. Many slope failures occur in colluvium overburden.

viii. Landslides and debris flows, caused by monsoon rains,are very common in the country. Debris flows are channeled into gullies on the lower slopes. Most gullies therefore should be regarded as potentially dangerous locations particular when human settlements exist at their bases.

ix. There are differences in the degree of propensity of different types of bedrock types to landsliding in their overlaying soil layers. Quartzite was found to be associated with the highest incidence of landslides with charnokitic gneiss and granulite, as close second (Fig-5).

x. Slope failures and rotational landslides of lobate geometry(length to with ratio=2) seem to occur only on cutting. Otherwise, most mass movement tend to acquire elongated form, with length to with ratios falling in the range of 2 to 9, or more (Fig-6).

xi. Most landslides seem to occur on slope between 18° and 31. Landslide in slope less than 11° and more than 45° are rare (Fig-7).

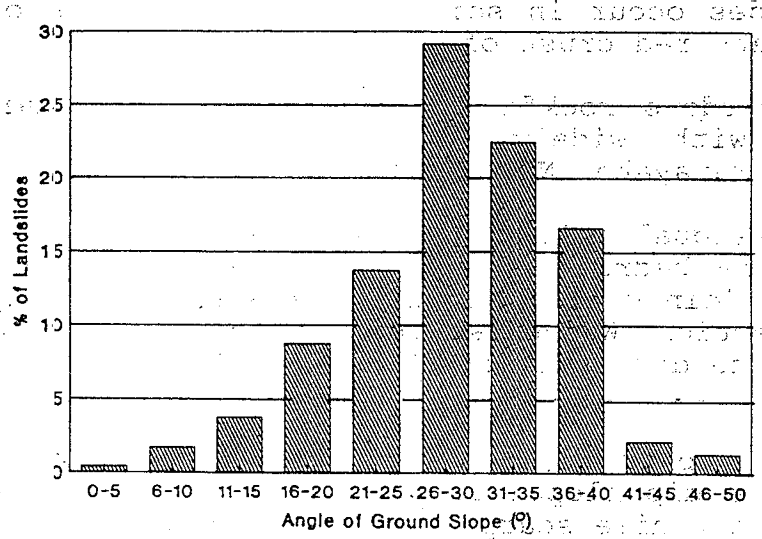


Fig. - 7.1 : Landslides by slope angle (Badulla District)

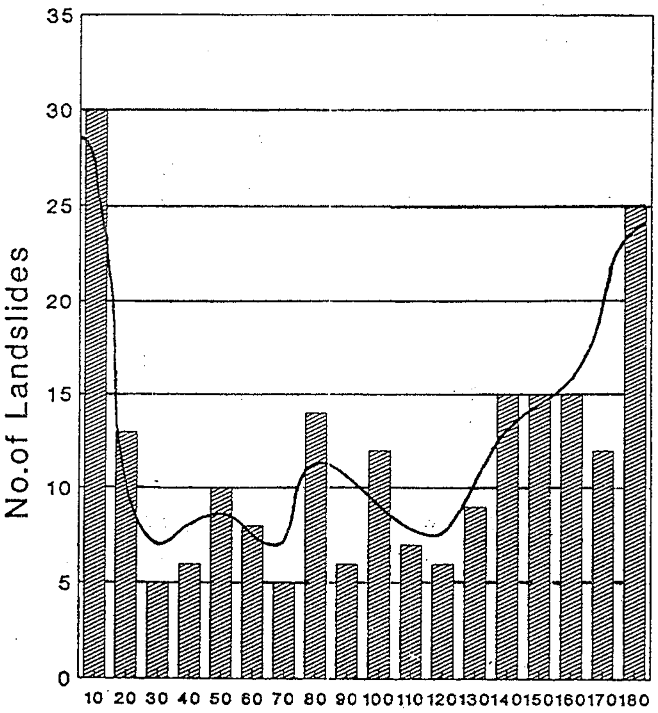


Fig. - 8 : Angle between slope bearing and dip bearing (^{0}) (Badulla District)

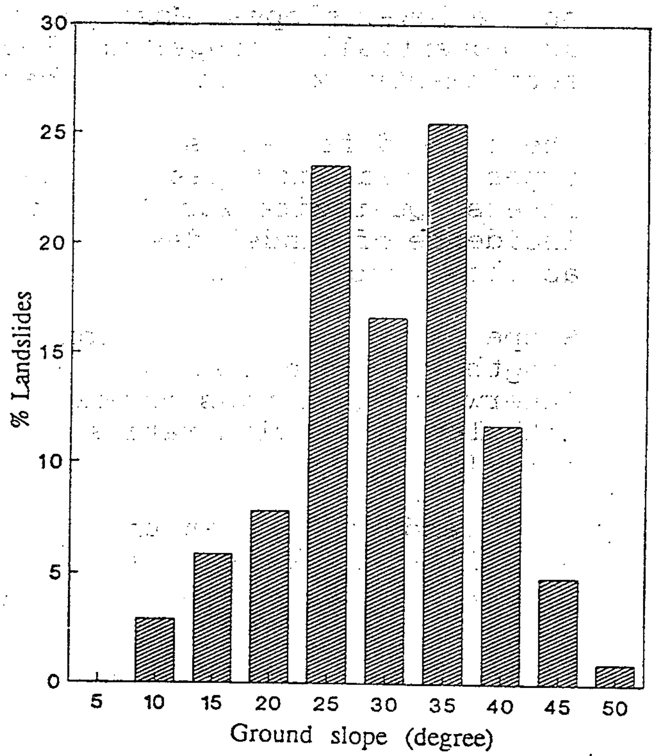


Fig. 7.2 : Variation of Landslide frequency with ground slope

xii. The highest incidence of landslides was found to correspond to derivation angle in the range of 0° to 10°. (Fig-8)

xiii. A grate majority of significant Sri Lankan Landslides were found to occur in basins with relative relief ranging between 200m to 300m (Fig-9).

xiv. Landslides vary in height from 100m to 200m, in nearly 55% of the cases studies. Lengths of stable slopes were found to lie between 800m and 1200m , but in catchments were landslides occurred, at least half the slopes were found to have lengths between 200m-400m (Fig-10).

xv. One limited study suggested that in the Sri Lanka, underdip slopes may even be more dangerous. In the rial-road excavations, slope failures involve bedrock. In such cases, the slope failures are generally along shear zones, faults, and joints or overdip situations can cause the failures.

4. Facts to be Considered during the Investigation Stage of Landslides and other Mass Movements in a Package Designed for a Landslide Remediation Model are

i. **Changes in the slope gradient:** These may be caused by natural or artificial influences (e.g. by the undermining of the foot of a slope by stream erosion, or by cavitation). The angle of the slope might get exceptinally is steepened as a result of tectonic processes, such as subsidence or uplift. An increase in slope gradient produces changes in the internal stresses of the rock mass and equilibrium conditions are disturbed by the increase of shear stress.

ii. **Changes in the slope height** as a result of vertical erosion or excavation work. The deepening of a valley relieves lateral stress and this in turn leads to the loosening of rocks in the slope and the formation of fissures parallel to the slope surface. The penetration of rain water is thus facilitated.

iii. **Overloading by embankments, fills and spoil heaps**. This produces an increase in the pore-water pressure in clayey soils, which results in decreased shear strength. More rapid the loading, the situation will be more dangerous.

iv. **Shocks and vibrations tremors produced by earthquakes**, large-scale explosions and machine vibrations affect the equilibrium of slopes on account of the temporary changes of stress that are caused by oscillations of different frequencies. The loess and loose sand shocks may disturb

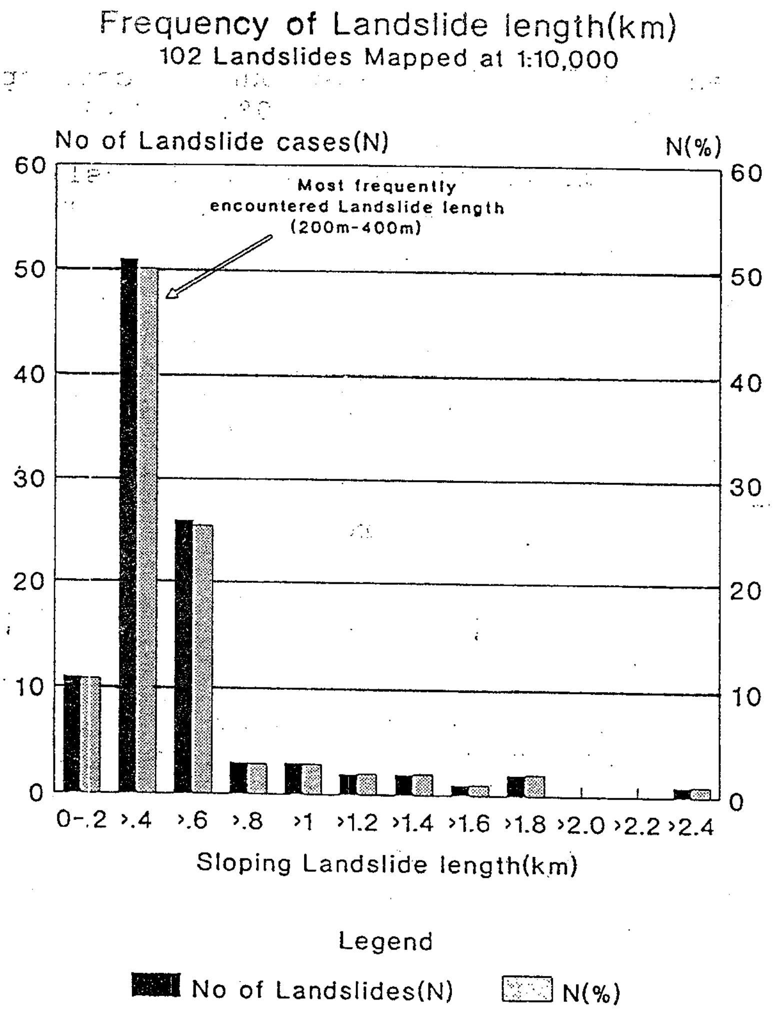

Frequency of Landslide length(km)
102 Landslides Mapped at 1:10,000
No of Landslide cases(N)
N(%)
Most frequently encountered Landslide length (200m-400m)
0-.2 >.4 >.6 >.8 >1 >1.2 >1.4 >1.6 >1.8 >2.0 >2.2 >2.4
Sloping Landslide length(km)
Legend
No of Landslides(N)
N(%)

Fig. - 9

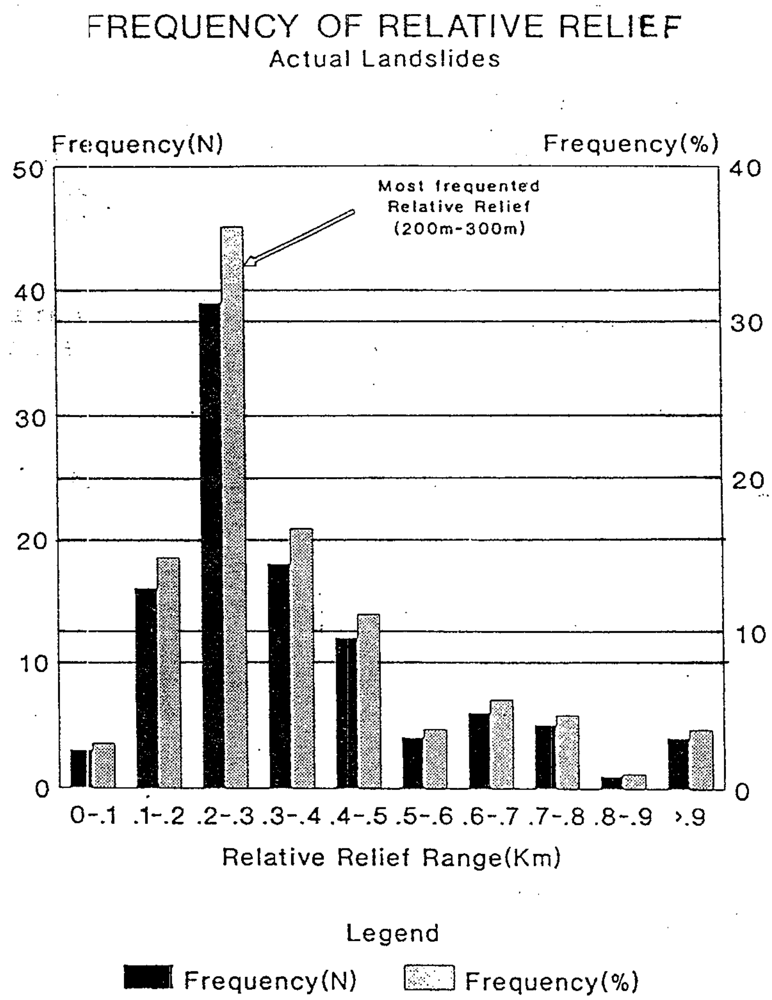

FREQUENCY OF RELATIVE RELIEF
Actual Landslides
Frequency(N)
Frequency(%)
Most frequented Relative Relief (200m-300m)
0-.1 .1-.2 .2-.3 .3-.4 .4-.5 .5-.6 .6-.7 .7-.8 .8-.9 >.9
Relative Relief Range(Km)
Legend
Frequency(N)
Frequency(%)

Fig. - 10

intergranular bonds and thus decrease cohesion. In water-saturated fine sand and sensitive clays, the displacement or rotation of grains can result in a sudden liquefaction of the soil.

v. **Changes in water content:**

(a) Rain and melt water penetrate into joints and produce hydrostatic pressure. In soils the pore-water pressure increases and consequently the shear resistance decreases. Measurements of rainfall have confirmed that recurrent slope movements occur in periods of exceptionally high rainfall.

(b) It has been found that between two beds of which the contact when acts as a sliding plane there is a difference in electric potential. The increase in the water content which leads to slope movement, is explained as a result of an electro-osmotic effect.

(c) In clayey rocks, the deleterious effect of atmospheric water is greater, when the rain occur after a long dry period; clayey soils are desiccated and shrunken so that the water readily percolates deep into the fissures.

vi **Effects of ground-water**

(a) Flowing ground-water exerts a pressure on soil particles which impairs the stability of slopes. Abrupt changes of water level as might occur in reservoirs, cause increase of pore-water pressure on slopes and this in turn may lead to liquefaction of sandy soils.

(b) Ground-water can wash out soluble cementing substances and thus weaken the intergranular bonds and reduce the mechanical strength of the ground.

(c) In fine sand and silt flowing ground-water flushes out fine particles and the strength of the slope is weakened by the cavities that are formed.

(d) Confined ground-water exerts an upward pressure on overlying beds.

vii. **Weathering, both mechanical and chemical, gradually disturbs the cohesion of rocks.** In many landslide events, chemical alterations such as hydration and ion exchange in clays thought to have contributed to the triggering of landslides. The tendency of slopes to slide has for example, been established in areas made up of glauconitic sands and clays.

viii. **Effects of vegetation**: The roots of trees maintain the stability of slopes by their mechanical effects and contribute to the drying of slopes by absorbing a part of the ground water. Deforestation of slopes adversely affects the water regime in the subsurface layers.

5. Landslide Remediation Works

The construction of Retaining Structures cum various ground improvements are commonly associated in stabilization of hill slopes . Variety of construction techniques involved in the above area can be sighed follows.

* Gabion walls
* Pile walls:
  With spacings or tangent piles, secant piles, "plates" of piles etc
* Diaphragm walls:
  Commonly installed as plates with a high resisting moment
* Pier walls:
  Consisting of large diameter caissons (d=2,5 to 25 m). Installed by sinking shafts which are reinforced and filled with concrete.
* Shotcrete arches between the caissons in the visible part of the structure
* Pinned or anchored retaining walls.
* Anchored element walls:
  Consisting of reinforced concrete panels; constructed in steps up-down the slope
* Walls cf reinforced concrete ribs and/or bars
* Rock nailing walls
* Revetment walls
* Covering of the rock face with (reinforced) shotcrete
* Arches of reinforced shotcrete and rock bolting
* Anchored ribs and conventional rock nailing walls with spacings
* Anchored ribs (cas-in-situ) combined with prefabricated crib wall elements. The cells between the stretchers may be filled with soil or reinforced concrete.

Retaining structures proposed for stabilizing of the slopes always should be considered in combination with accompanying measures(Fig-13), i.e.

* an efficient drainage system
* a proper profile of the slope surface
* a stabilizing vegetation
* erosion protection
* sealing of open cracks
* local rockfall protective measures etc.
* measures to increase the shear parameters

From a theoretical point of view berms are unfavourable, as they cause local stress concentrations (north stresses). But in engineering practice they have proved very successful for several reasons:

* They provide permanent access to the slope.
* They facilitate maintenance and remedial works.
* They facilitate long-term monitoring.
* They allow the construction of supplementary retaining structures at several level (e.g. anchored beams, ribs) and are therefore an essential prerequisite for a semi-empirical design.
* They permit proper surface drainage and subsurface drainage.
* They provide a braking effect on local slides or rockfalls and may act as traps with catch fences.

Other measures for ground improvements

* Horizontal / inclined drainage systems; Directional drilling
* Ground water pumping wells; open wells etc.
* Sealing of shear surfaces, tension cracks etc.; chemical treatments
* Ground improvement by chemical process or
* Hazard Protection galleies

6. Factors which should receive attention during the Investigation Phase in respect of Design Methods of Early Warning against Landslides

The question of design and implementation of early warning against landslides primarily arise in the context of the following two situations.

a. Early warning against possible reactivitation of landslides or development of new landslides (includes the rock/debrisflows of slope escarpments), in areas known to be landslide prone.

b. Early warning against occurrence of landslides, (for the first time) in the areas which are generally known to be safe and without any significant previous landslides history.

The NBRO study of landslides that occurred during the period of 1957 to 1989 reveals that the month of December to January and May to June are particularly dangerous from the landslides point of views.

Areas which require early warning are generally considered as unsafe, partially safe, or areas with some early symptoms of landslides identified within the subject area. Reliable forecasting of a landslide is always a big challenge and then happened to be landslides in areas either known to be safe or predicted to be safe.

6.1. Some of the Early Symptoms of Landslides are listed below only to serve as examples;

i. Proporation of Excessive rainfall spread over a long period of time, particularly unusual or unprecedented events.

ii. Sudden opening, and progressive widening of cracks on the slope surface or on the walls of buildings, houses and other structures.

iii. Progressive or sudden tilting (forward or backward) of trees, poles, towers etc. located on the slopes.

iv. 'Subsidences' or 'heaves' observed on the slopes.

v. Sudden oozing or appearance of water on the slope and continuous water logging due to poor slope drainage.

vi. Spurt of rockfall activity on the unstable upper slopes.

vii. Subsidence of roads and building of road side retaining walls.

viii. Usual behaviour of domesticated animals and birds.

7. Landslide Prone Area Identification and Mapping as the First Step in the Investigation Methodology

The landslide Hazard Mapping project(LHMP) of NBRO has investigated large number of landslide prone areas as well as individual cases. Determination of landslide prone areas basically involves four steps;

i. Propavation of state of nature maps on slope range and slope category;bedrock geology;landform; landuse & management category are prepared following the appropriate legends develop under the project

ii. Correlating the other factors with topography, geology, environmental issues(landuse & landform managements etc) and other factors which might contribute to landslide.

iii. Determining the relative importance of each factor in that particular location and relative ranking of land use and management category mapping units according to their instability potential

iv. Search and extrapolation of the risk to ares with similar conditions and improve statistical convergence in forecasting of slope failure potential. An index called Slope Failure Susceptibility Inde (SFPI) which is the final outcome of this analysis indicates the landslide potential at a site by integrating the relative hazards due to above factors.

8. Environmental Considerations:

Landslides have become a significant national disaster that causes sever damage to lives, properties and to the natural environment. Although, the action in the past has largely been an ad-hoc implementation strategy due to lack of information needed for planning the counter measures against natural disasters in the country, the Landslide Hazard Zonation will provide information on the vulnerable areas. However, some steps have been considered for better management, hazard mitigation and protection of the natural environment. The following common identifications are significant in each and every occurrence of landslide.

i. Danger to life, property and infrastructure

ii. Degree of soil erosion

iii. Increase of sedimentation

iv. Flooding

v. Impacts on agriculture

vi. Damage to forest cover and wildlife

In safeguarding the environmental impact,drafting of a natural disaster mitigation plan has been initiated by the Government in 1990 under the National Environment Action Plan. All activities associated with planning and implementation of mitigatory measures against landslides have to be in accordance with the rules and regulations of this act.

9. Strategy for Cost Effective Management & Planning:

A multi-disciplinary approach should produce positive results on the overall design. Management of activities associated with total planning of geotechnical projects is essential. It should pay more attention to the appropriate methodology, correct techniques and suitable equipment for the particular type of investigations. The experiences on above facts must create awareness on some hard realities associated with planning and implementation of various proposals at the investigation stage of a project.

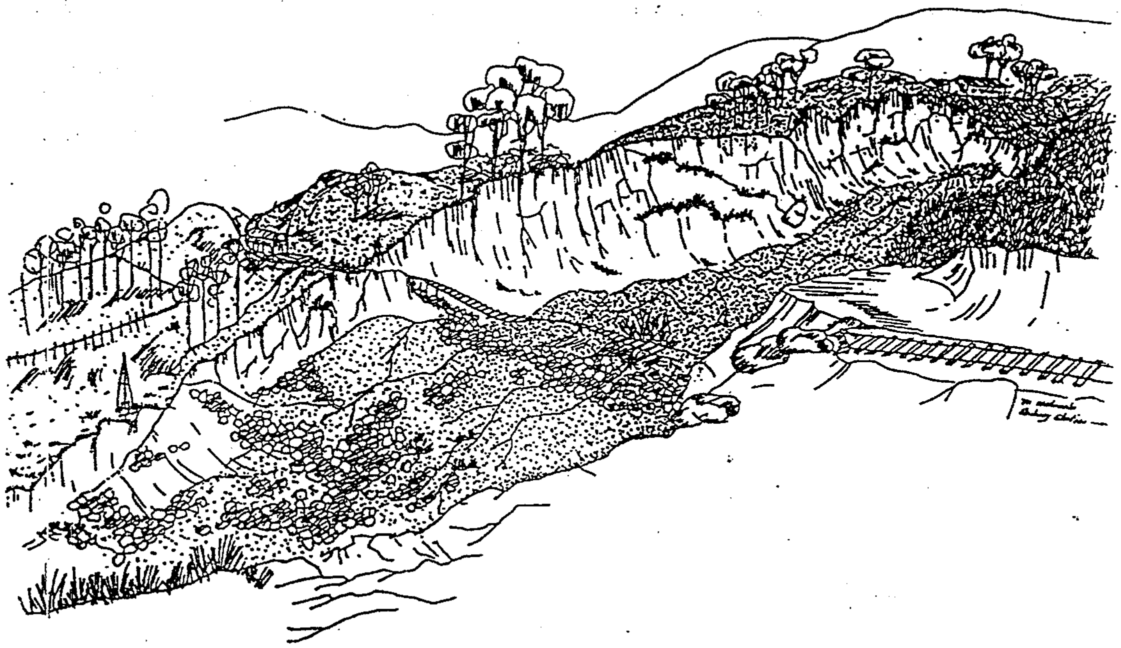
There is less time nowadays for application of trial and error methods in execution of a contract or a design. Time consuming planning steps cannot be proposed in the modern construction process and therefore it should be based on experience. Another problem is the limitation of expertise involved in the geotechnical investigations and for the remediaton works. The basic principles behind any effective project planning are, preliminary cost effective management, and time saving features. These concepts are even valid for the planning and implementation of research or site investigation projects on landslides. It is worthwhile to quote the following examples.

Case Study - I

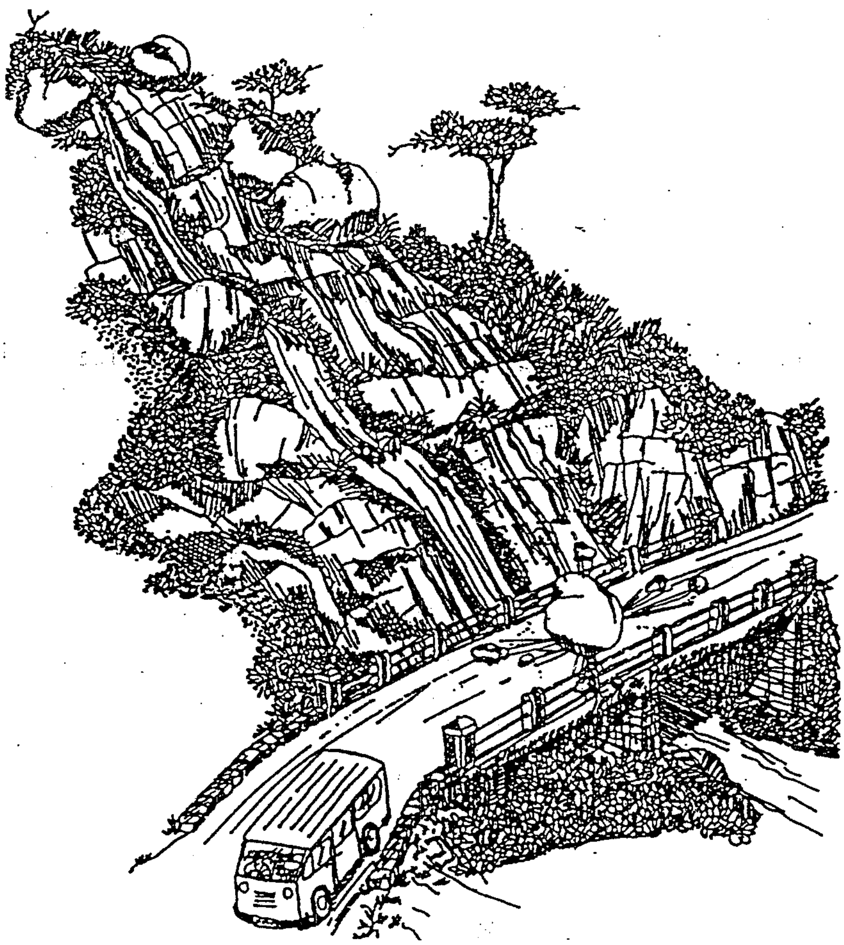
Take for an example the earthflow of 3rd June 1993 at the Watawala railway track in Sri Lanka(Fig-11). This has been known as a gradually growing and re-occurring problem for well over three and a half decades. The earthflow at Watawala is a serious problem due to its re-occurrence. It may entail losses of human lives, not to mention the short term and long term damages to the environment. Therefore immediate attention of authorities was essential for suitable actions in respect of design and implementation of interim control measures as well as permanent remediation packages.

Case Study - II

The debris flow cum rockfall occurred at a location 3km from Haputle town, on the Beragala - Haputale Road(Fig-12) on 16th November 1992. The heavy flow of debris and boulders destroyed 20m length of Beragala - Haputale road, 30m length of Viharagala Estate road and Beragala - Wellawaya road. The Beragala - Wellawaya Road has been blocked by the movement of huge amount of rock and debris. Also this event caused disruption to vehicle traffic for about 3



A Panoramic view of the Watawala earthslide (August 1992)



A Panoramic view of the Beragala earthslide (August 1992) weeks on the Beragala - Haputale Road and for about 1 week on Beragala - Wellawaya Road (Bandara and Kumarapeli; 1994). It was felt necessary to investigate the mechanism of the debris flow and possibility of reoccurrence for the benefit of road users to guarantee the safety.

Reviewing the lessons learnt from the experience of uncertainty prevailing in landslide remediation and management, it is observed that the focus has to be given to the following four aspects during the investigation phase:

i. Establishment of multi-disciplinary expertise to investigate, analyses the problem and formulate cost effective control measures.

ii. Development of appropriate technology for landslide control and training of personnel including training of small contractors in application area of new technologies.

iii. Prompt activation of agencies responsible for implementation of landslide control measures.

iv. Integration of project management with the planning process and recognition of costs involved in the prevention of natural disasters as a part of the cost involved in the sustainable development.

10. Suggested Additional Guidelines

The authors suggest that the following aspects to be considered in addition to other common priorities in planning and management

i. Engineering fallacies and practical aspects

ii. Flexibility of time and expenses involved

iii. Systems to be introduced for project control to suit the changes of dimensions

iv. Assessment of risk involved

10.1. Engineering Fallacies and Practical Aspects

In research and investigation planning, it may not always be possible to involve high calibre professionals in all functions in reality. However, the contractor should at least be aware of the need for his constant interferance to achieve the interrelationship between the design aspects and its implementation strateges. Most of the proposed ground investigations tend to change its activity during the course of the project. Hence it is necessary that the project manager do everything to prove that the project is acceptable in terms of benifits and the best possible experts are available to the project. Therefore, there will be additional localized cost added to the project.

The following significant fallacies are common in these studies.

i. Limitation of resource and financial constraints for an extension of investigation assignments.

ii. Area of investigations is not restrained to solve the client's problems. Flexibility for extension or detail study is not always possible.

iii. Investigation of large area through mapping may not be possible and the management strategy may tend to change its dimensions to cover only the areas necessary.

iv. Sub soil investigation methodology is still limited to borelogs, standard penetration tests and/or minor seismic interpretations.

v. Lack of knowledge on the legal implications on environmental considerations.

vi. Lack of necessary geotechnical instrumentation for monitoring

10.2. Allowing for Flexibility

Many geotechnical engineering projects subjected to change in different ways during their lifetime. However, the project administration must overcome or minimize the adverse effects caused due to these changes. The research component of such projects may require originally unforeseen extensions in time and budget mainly due to the expert recommendations. If the project schedule does not keep to the allowances for flexibility such extended research or investigation activities will not bring the desired overall success or otherwise may be jeopardized totally. The Samanalawewa Hydro-Electric power project will serve as one of the best examples from Sri Lankan experiences.

10.3. Introduction of Systems for Project Control to suit the changes of Dimensions

In order to avoid uncertainties prevailing in certain projects the authors suggests to draw general lessons from experience in hand, linking the research and investigation projects, by introducing systems to suit the overall project schedule as a measure of control. Any scientist who has questions on technical matter or on plans should be invited to attend a briefing on the entire programme. He should be encouraged to solve the same through a investigation plan in the relevant areas using appropriate technology. Control over the technical goals and the main thrust of its development efforts, however, must always be the technical responsibility of the project manager.

10.4. Assessment of Risk Involved

The effectiveness of system introduce for project control to suit the changing measures is dependent to a large extent on the depth of analysis about the probability and nature of changing strategy, resource involved and a degree of loss of elements at risk. Its, therefore, become necessary to undertake risk assessment with a reasonable degree of accuracy. The following suggestion are recommended to follow up work,

i assessment should adequately reflect the efficacy of low cost appropriate technologies and the action needed for their implementation by way of wider sharing, reinforcement of knowledge and experiences of other relevant parties.

ii The standards of risk assessment to be upgraded by fully applying the existing knowledge in the physical and sccial senses, cost and benefits involved in management planning. It may be taken up as a strategy in order to demonstrate the utility of these techniques;

11. Conclusion

Different experiences in different projects motivate people concerned to analyse the situations in order to undertake or propose suitable solutions. There is a risk of failure because, the financial constraints may not allow for unexpected extensions or modifications to either the investigation plan or construction schedule of a project. Such risk can be minimized by careful planning, introduction of systems for project control by taking into consideration the overall activities at the initial stages of the project. The above said facts have direct relations to the implementation of projects, connected with landslide investigations and its remediation.

12. Acknowledgements :

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GEOTECHNICAL INVESTIGATION IN MEDIUM SCALE IRRIGATION RSERVOIRS.

* Eng. E.A.C.Ekanayake ** Eng. K.W.Perera

INTRODUCTION

The general objectives of a site investigation for a proposed dam are;

i. To asses the suitability of the site for the proposed dam and relevant structures.

ii. To foresee problems that may arise during construction due to ground and other local conditions.

iii. To search for suitable construction materials.

iv. To check the basic design assumption made regarding ground condition as construction proceeds and tomodify the design if conditions differ from predicted ground conditions.(This type of investigation is done during construction)

A medium scale Irrigation reservoirs is one having a storage capacity sufficient to assure irrigation for an extent of 200 to 1500 Acs during Maha season. Also the maximum height of embankment should not exceed 50 ft. A dam is made up of a manmade part and the natural part which constitute the two abutments and the foundation of the manmade part. For economic reasons the investigations for medium scale reservoirs are limited to the investigations of foundation for manmade part, spill, sluice and on borrow materials for the embankment.

This paper deal with the methodology for sub surface investigation for a medium scale reservoir. Investigations are required mainly to decide on suitable foundation for the embankment, the spill, and the sluice and for barrow areas.

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GEOTECHNICAL INVESTIGATIONS

Investigations of foundation for a medium scale reservoir will reveal the type and condition of materials available under the dam foundation and spill structure and the geological parameters of the material affecting the ground characteristics.
The different characteristics of the ground normaly considered in the design of dams are as follows:

- Water tightness
- Stability
- Deformation

It is important to realize that each of the above characteristics is linked to the other two in various ways.

Geotechnical Investigation consist of two stages:

I. Preliminary reconnaissance survey.

II. Sub surface investigations under dam and investigation of borrow areas

I. Preliminary reconnaissance survey

Purpose of this survey is to select favourable alternate dam axes and sites for a spill structure.

This survey work involves:

i. Studying available maps (Topographical & Geological maps)

ii. Through inspection of area and studying rock outcrops, geological features, geological structures etc.

iii. Collection of information from man made structures.
eg. wells, quarries, road cuts etc.

General view of regional and site geology and ground water table could be obtained from the above survey.

II. The sub surface Investigation.

The aim of sub surface exploration is to determine and understand the nature of ground condition at dam site and to determine the availability of suitable material for bund filling in borrow areas.

The sub surface investigation consist of;

i. Description of materials in the dam foundation and borrow areas (Stratigraphy, Lithology)

ii. Recording of structural elements in the rock layer (joints, cavities, infill material etc.)

iii. Determination of Degree of weathering of rock layers.

iv. Determination of depth of water table

v. Determination of geotechnical parameters such as permeability, density & strength characteristics in soil & rock layers by in situ tests.

vi. Collection of soil samples from dam foundation and borrow areas for laboratory testing.

vii. Laboratory tests to determine the soil type under dam foundation and in borrow areas and also to determine the proctor density and shear parameters of borrow area material.

SUB SURFACE EXPLORATION METHODS

In general, dam foundation is classified into two layers viz.

i. Overburden	-	Consisting of soil layer
ii. Rock	-	Rock layers with different degree of weathering

The investigations in overburden of foundation of dam and borrow areas are carried out by digging trial pits and auguring. While digging and auguring, visual classification of soils are carried out and disturbed and undisturbed samples are collected for laboratory testing. The specimen of auger hole log is given in annex 1.

In rock layers the investigations are carried out by Rotary drilling. While drilling, rock samples are collected and arranged in core boxes for logging. The drilling is carried out along the dam axis and at spill site. The drill holes are located depending on site conditions (geological and topographical) and length of dam. At spill site drilling is carried out along the dam axis and also along centre line of spill to determine the ground condition in the spill foundation and spill tail canal. The drilling is carried down to a depth until fresh rock is encountered and 3m in fresh rock.

The following details are obtained by logging the core samples obtained by drilling:

i. Details of joints(spacing, roughness, infill material etc.)

ii. Lithology

iii. Change of strata with depth

iv. Degree of weathering of rock

v. Rock quality designation (RQD) & core recovery

The specimen of a bore hole log is given in annex 2.

IN SITU TESTS

In order to determine the water tightness of dam foundation, following tests are carried out in the field:

(a) Constant Head Permeability Test (1)

This test is carried out in overburden while boring, to assess the permeability of different strata. The test is carried out by measuring the amount of water accepted by the ground through the open bottom of the pipes or through uncased section of the hole. This test is done at 1m. interval.

$$k = Q/5.5*r*H$$

Where;

k - permeability (ft/year)

Q - Constant rate of flow into the hole (gls/min)

r - Internal radius of casing (ft)

H - Difference in head in water (ft)

(b) Water Pressure Test (3)

This test is carried out to determine the permeability of rock mass. This is done by sealing a length of a uncased bore hole with packer and injecting water under pressure and observing water intake for 10 minutes. This is repeated in five steps by ascending and descending order of pressure. From these test results the permeability of rock mass could be measured. This test is carried out at 3m. intervals. This test is used more frequently to determine the fractured condition of rock than its actual permeability.

Permeability $K = Cp * Q/H$

Where;

K - permeability in lugeons

Q - flow rate in lt./min.

H - effective applied pressure in metres

K - permeability in lugeons

Q - flow rate in lt./min.

H - effective applied pressure in metres

Cp - Constant depending on length and diameter of test section

In order to asses the bearing capacity of soil the following test is carried out:

(c) Standard Penetration Test (2)

This test is carried out at 2m intervals while driving the bore hole. The test consist of driving a split spoon sampler with an outside diameter 50mm into the soil. Driving is done by a trip hammer weighing 63 kg falling through a distance of 750 mm on the drive head. The number of blows required to drive 300mm is referred as 'N value' from which relative density of soil can be assessed. The N-value is also related to angle of internal friction and bearing capacity.

LABORATORY TESTS (2) & (4)

Using disturbed and undisturbed samples the following tests are carried out in the laboratory:

i. In-situ Moisture Content

This is to determine the available moisture content in the borrow area. Depending on the in-situ moisture content water is added to borrow material or borrow area is dried to achieve the required proctor optimum moisture content for the fill material.

ii. Atterberg limits (Plastic & liquid limits)

Plastic and liquid limits depend on the amount and type of clay in a soil. The plasticity index is the numerical difference between the liquid limit and plastic limit. This test is carried out to identify the clay material to enable placing of the material at the correct location in the dam section.

iii. Mechanical Analysis

This test determines the grading or percentage of the various particles namely clay, silt, sand and gravel contained in a soil sample and the soil is classified on this basis. The soil classification is done according ASTM standards. Unified soil classification chart is given in appendix 5.

iv. Proctor Compaction Test

Here, compaction is measured quantitatively in terms of the dry density of the soil. The sample is compacted using the automatic compaction machine. Proctor `A' standard uses a 5.5 lbs hammer falling freely through 12 inches and the soil is compacted in 03 layers with 25 blows per layer. This is done to ensure proper compaction of fill material (The compaction in fill material should be more than 98% of proctor compaction value). Field density calculation by core cutter method is given in appendix 6.

v. Triaxial shear test

This test is an indirect method of determining the angle of shearing resistance, o and the value of unit cohesion c as defined by Couluomb's law. For the purpose of obtaining the shear parameter of the fill material consolidated undrained test on saturated soils is performed.

GEOTECHNICAL INVESTIGATION REPORT

A final report is prepared by incorporating the following;

i. Description of work carried out

ii. Logs of auger hole (Appendix 1)

iii. Logs of all bore holes (appendix 2)

iv. In situ test results

v. Laboratory test results

vi. Location plan of all trial pits, auger holes, bore holes (appendix 4.)

vii. Interpretation of all test results

viii. Geotechnical profile along dam axis as given in (appendix 3.)

ix. Conclusion & recommendations.

REFERENCE

I. Design of Small Dams
U.S.Department of the interior, Bureau of Reclamation

2. Earth Manual - A Water Resources Technical Publication
U.S. Department of the interior, Bureau of Reclamation

3. An introduction to Engineering Geology
Prof. D.G.Price.

4. Operation Manual of the Engineering Materials
Laboratory - Irrigation Department

SOILS LABORATORY - IRRIGATION DEPARTMENT- SRI LANKA APPENDIX-I

LOG OF TEST PIT OR AUGER HOLE

PROJECT : MAU ARA SHEET : OF:

SITE : BORROW AREA (D/S - FOR CH MATERIALS)

PIT / HOLE NO: A.H. No. 02 LOCATION OF PIT / HOLE :

GROUND ELEVATION : DEPTH OF WATER TABLE : -

SUBMITTED BY : S.J. AND Y.U. DATE : 96·07·10

DEPTH BELOW SURFACE FT. INS	CLASS	SYMBOL	DESCRIPTION	NO.	SAMPLE TYPE	DEPTH IN FT.
0'- 0" – 0' - 3"			TOP SOIL			
0' - 3" – 1' - 3"	SM		LIGHT BROWN COLOUR ABOUT 70 % FINE TO MEDIUM SAND WITH NON PLASTIC FINES.			
1' - 3" – 10' - 3"	CH		BLACKISH BROWN COLOUR ABOUT 10 % FINE SAND WITH ABOUT 90 % HIGH - PLASTIC CLAY FINES.			
			BELOW 10'- 3" QUARTZ LAYER			

REMARKS :

<u>APPENDIX - 2</u>

DEPARTMENT OF IRRIGATION

BOREHOLE LOG FOR ENGINEERING PURPOSES

PROJECT WEMEDILLA TANK

FEATURE Alternative spill site 2 50 ft. away from center of outcrop NE

BH No. DH - 14 **SHEET** 01 OF 01

DRILLING DATA

DATE STARTED 16-09-94
DATE COMPLETED 22-09-94
INTERRUPTION (DAYS)
DRILL BIT TYPE & SIZE
FOREMAN
LOGGED BY. S. Kodikara
CORE STORED AT
CORE DISCARDED DATE

BOREHOLE DATA

X - CO-ORDINATE
Y - CO-ORDINATE
ELEVATION (COLLAR)
ELEVATION (BOTTOM)
FINAL DEPTH 20'-4"
INCLINATION Vertical
BEARING

KEY

JOINT ROUGHNESS
VR = Very Rough
R = Rough
SR = Slightly Rough
S = Smooth
SI = Slickensided

JOINT SEPARATION
VT = Very Tight < 0·1 mm.
T = Tight 0·1 - 1·0 mm.
MO = Med. Open 1·0 - 5·0 mm.
O = Open > 5·0 mm.

JOINT SPACINGS
VW = Very Widely > 2 m.
W = Widely 0·6 - 2 m.
MW = Med. Widely 0·2 - 0·6 m.
C = Closely 0·06 - 0·2 m.
VC = Very Closely < 0·06 m.

OTHER SYMBOLS
P.L.S.I = Point Load Strength Index.
U.C.S = Uniaxial Compressive Strength.
▼ = First Ground Water
R. = Rock Sample
W. = Water Sample

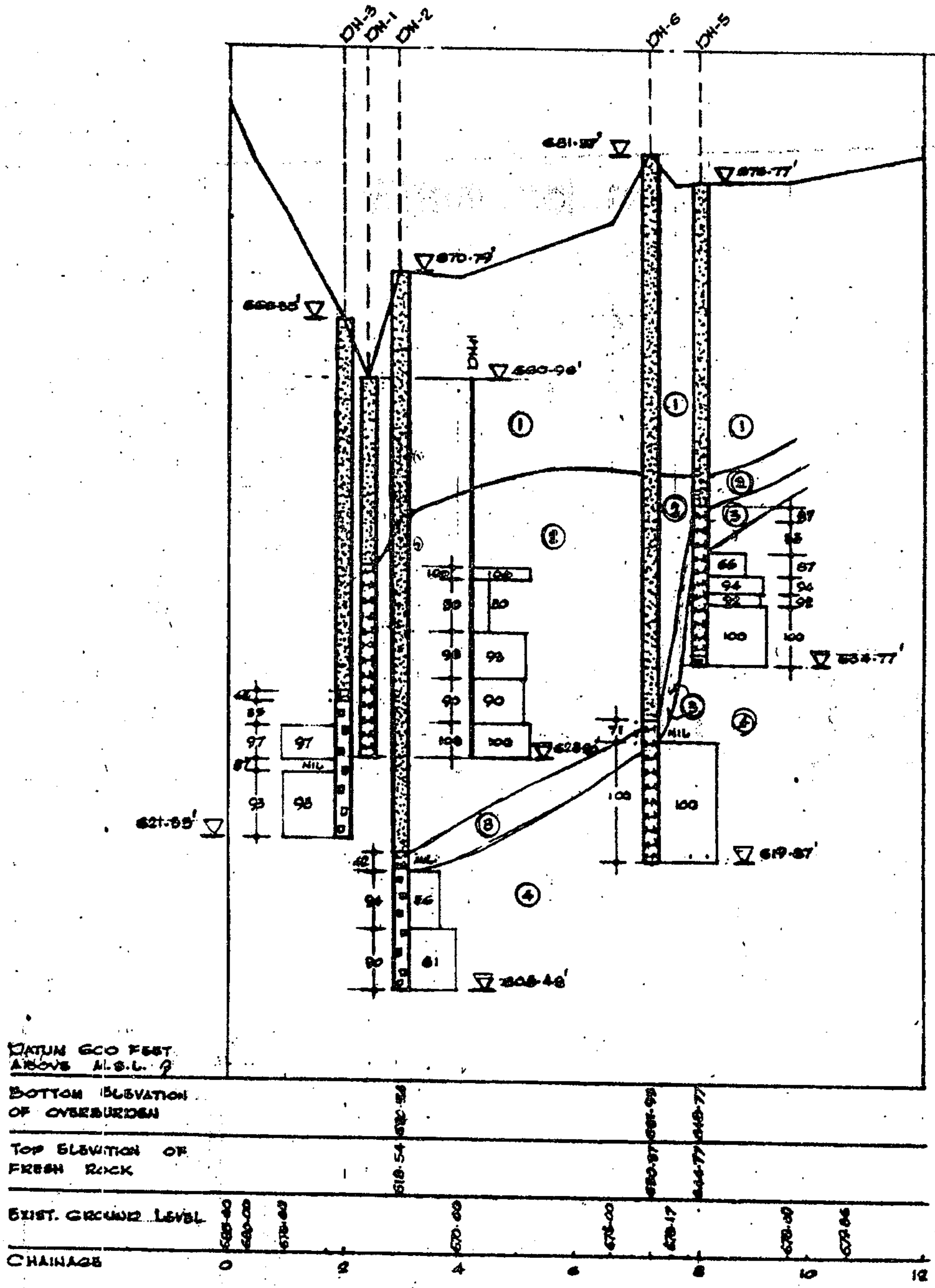
WEATHERING

Soil; Unconsolidated Materials
Completely
Highly
Moderately
Slightly
Fresh

DEPTH (m)	DRILLING: DRILL WATER (Colour, Loss)	PERMEABILITY: PRESSURE (P.S.I.) (Manometer Reading)	WATER LOSS Total (gal.)/Time (min.)	LUGEON OR K (m/s)	RECOVERY: CORE RECOVERY %	R.Q.D. %	GENERAL DESCRIPTION (Rock Type, Colour, Grain size, Texture and Structure (massive, Cleaved, Foliated, Lineated, Flow banded, Gneissose, Porphyritic, etc. Scale for Joint Spacing.) Weathering, Alteration Minor Lithological Characteristics Strength, Joints.)	ENGINEERING ASPECTS REMARKS (With Respect to Excavation, Grouting Permeability, Rock Quality, etc.)
1–2	Brown 100%						Brown soil with Quartz particals	
2–3	Grey 100%						Grayish silty sand	
3–4	Brown 100%						L/Brown sandy soil	
4–5					NIL	NIL	Brown medium grain Sand	
5–11	Grayish 100%						Yellowish fine to medium grain Sand	
11–12	Grey 100%						Fresh Garnetiferous granulitic gneiss fractured into 2" pcs.	
12–15	Grey 100%	8 12 14 12 8	1·32 1·41 1·50 1·38 1·30	20·8	65	52	M.W to Fresh garnetiferous granulitic gneiss 11'-6"-12" - H.F. into 2"-3" pcs.	
15–19	Grey 100%				95	95		
19–20	Grey 100%				100	100		

HOLE COMPLETED AT 20 feet 4 inches

APPENDIX-5



GEOTECHNICAL SECTION ALONG U/S. TOE LINE OF EXIST. DAM AXIS

LEGEND

LITHOLOGY

- TOP SOIL & COMPLETELY WEATHERED ROCK
- HORNBLENDE BIOTITE GNEISS.
- CHARNOCKITIC GNEISS

PERCENTAGE OF CORE RECOVERY IN DRILL LOGS

18

58

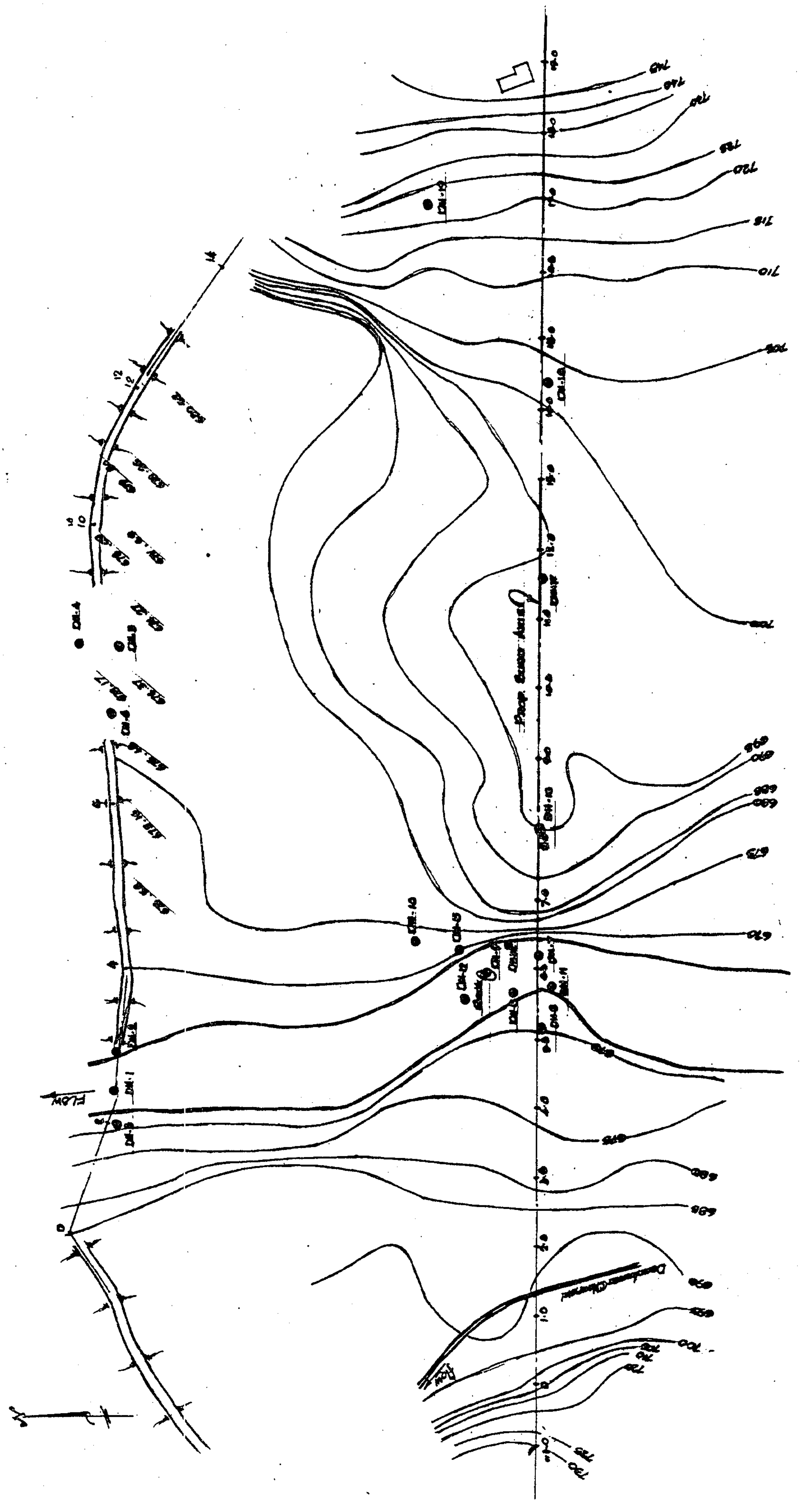
R.Q.D. VALUES INDICATED IN PERCENTAGE

Scale-½" = 100%

DEGREES OF WEATHERING

1. OVERBURDEN
2. COMPLETELY WEATHERED ROCK ZONE
3. SLIGHTLY TO MODERATELY WEATHERED ROCK ZONE
4. FRESH ROCK ZONE

APPENDIX-4



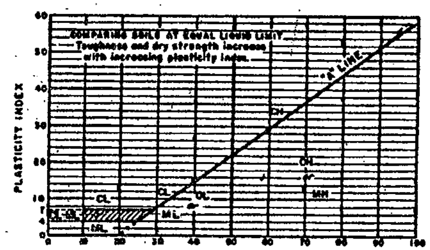
SITE PLAN
Scale: 2 Chains to an Inch

UNIFIED SOIL CLASSIFICATION
INCLUDING IDENTIFICATION AND DESCRIPTION

Major divisions			FIELD IDENTIFICATION PROCEDURES (Excluding particles larger than 3 inches and basing fractions on estimated weights)	GROUP SYMBOLS ¹	TYPICAL NAMES	INFORMATION REQUIRED FOR DESCRIBING SOILS	LABORATORY CLASSIFICATION CRITERIA
COARSE GRAINED SOILS: More than half of material is larger than No. 200 sieve size ² (The No. 200 sieve size is about the smallest particle visible to the naked eye)	GRAVELS: More than half of coarse fraction is larger than No. 4 sieve size. (For visual classifications, the ¼" size may be used as equivalent to the No. 4 sieve size.)	CLEAN GRAVELS (Little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	GW	Well graded gravels, gravel-sand mixtures, little or no fines.	Give typical name; indicate approximate percentages of sand and gravel, max. size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbol in parentheses.	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between one and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing	GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.		Not meeting all gradation requirements for GW
		GRAVELS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below).	GM	Silty gravels, poorly graded gravel-sand-silt mixtures.		Atterberg limits below "A" line, or PI less than 4 — Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols.
			Plastic fines (for identification procedures see CL below).	GC	Clayey gravels, poorly graded gravel-sand-clay mixtures.	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics.	Atterberg limits above "A" line with PI greater than 7
	SANDS: More than half of coarse fraction is smaller than No. 4 sieve size.	CLEAN SANDS (Little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.	SW	Well graded sands, gravelly sands, little or no fines.		$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between one and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing.	SP	Poorly graded sands, gravelly sands, little or no fines.	EXAMPLE:- Silty sand, gravelly; about 20% hard, angular gravel particles ½-in. maximum size; rounded and subangular sand grains coarse to fine; about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM)	Not meeting all gradation requirements for SW
		SANDS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below).	SM	Silty sands, poorly graded sand-silt mixtures.		Atterberg limits below "A" line or PI less than 4 — Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols.
			Plastic fines (for identification procedures see CL below).	SC	Clayey sands, poorly graded sand-clay mixtures.		Atterberg limits above "A" line with PI greater than 7

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

Major divisions		IDENTIFICATION PROCEDURES ON FRACTION SMALLER THAN No. 40 SIEVE SIZE: DRY STRENGTH (CRUSHING CHARACTERISTICS)	DILATANCY (REACTION TO SHAKING)	TOUGHNESS (CONSISTENCY NEAR PLASTIC LIMIT)	GROUP SYMBOLS	TYPICAL NAMES	INFORMATION REQUIRED FOR DESCRIBING SOILS
FINE GRAINED SOILS: More than half of material is smaller than No. 200 sieve size.	SILTS AND CLAYS: Liquid limit less than 50	None to slight	Quick to slow	None	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity.	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; color in wet condition, odor if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses.
		Medium to high	None to very slow	Medium	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	
		Slight to medium	Slow	Slight	OL	Organic silts and organic silt-clays of low plasticity.	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions.
	SILTS AND CLAYS: Liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	
		High to very high	None	High	CH	Inorganic clays of high plasticity, fat clays.	EXAMPLE:- Clayey silt, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (ML)
		Medium to high	None to very slow	Slight to medium	OH	Organic clays of medium to high plasticity.	
HIGHLY ORGANIC SOILS		Readily identified by color, odor, spongy feel and frequently by fibrous texture.			Pt	Peat and other highly organic soils.	



PLASTICITY CHART
FOR LABORATORY CLASSIFICATION OF FINE GRAINED SOILS

¹ Boundary classifications:- Soils possessing characteristics of two groups are designated by combinations of group symbols. For example GW-GC, well graded gravel-sand mixture with clay binder.
² All sieve sizes on this chart are U.S. standard

ADOPTED BY:- CORPS OF ENGINEERS AND BUREAU OF RECLAMATION - JANUARY 1952

Figure 7.—Unified soil classification chart. From drawing 103-D-347.

474-122 O - 74 (Face p. 12)

IRRIGATION DEPARTMENT, SRI LANKA
ENGINEERING MATERIALS LABORATORY

IN SITU UNIT WEIGHT

(Sand replacement method)

(Form EM 51)

Project: WEMEDILLA TANK Feature: INVESTIGATION

Location / Pit No.: 02

Elevation: 3'- 6" BELOW THE SURFACE Date:

(1) DETERMINATION OF VOLUME OF PIT

Weight of Can and Sand before pouring into Pit	(W_1)	
Weight of Can and Sand after pouring into Pit	(W_2)	
Weight of Sand poured into Pit	(W_1-W_2)	
Unit Weight of the Sand	(D)	
Volume of the Sand poured into Pit	$(V = \frac{W_1-W_2}{D})$	

(2) DETERMINATION OF WEIGHT OF SOIL REMOVED FROM PIT

Weight of ~~Can~~ Cutter and Soil removed from Pit	(W_3)	3.045 Kg.
Weight of Can	(W_d)	1.050 Kg.
Weight of Soil removed from Pit	$(W = W_3 - W_d)$	1.995 Kg.

(3) DETERMINATION OF MOISTURE CONTENT OF SOIL

Tin Number		A
Weight of Tin and Wet Soil	(M_1)	113.10
Weight of Tin and Dry Soil	(M_2)	100.40
Weight of Tin	(M_o)	32.20
Weight of Moisture	(M_1-M_2)	12.70
Weight of Dry Soil	(M_2-M_o)	68.20
Moisture Content	$(w = \frac{M_1-M_2}{M_2-M_o} \times 100\%)$	18.60

(4) DETERMINATION OF DRY UNIT WEIGHT OF SOIL

Bulk Unit Weight of Soil	$\gamma = (\frac{W}{V})$	125.40
Dry Unit Weight of Soil	$\gamma_d = (\frac{W}{V} \times \frac{100}{100+w})$	105.70

Remarks: CONTAIN FRIABLE LIMESTONE PARTICLES.

Tested by: KW

Computations Checked by: DLR

A REVIEW OF SITE INVESTIGATION PRACTICES IN LATERITIC SOILS FOR BUILDINGS

by

Prof. B.L. Tennekoon, Miss. H.A.Y. Chamindani and Miss. K.A. Madunishanthi

1.0 INTRODUCTION

It is now recognized that some form of site investigation is necessary before commencing any building operation. This could vary between excavation of a few trial pits for small structural loads to the advancement of boreholes upto rock for heavy structural loads. The investigation undertaken should be both appropriate and adequate. As a step in this direction, ICTAD has published 'Guidelines of Site Investigations for Foundations of Buildings'. This paper reviews some of the site investigation practices in this country in lateritic soils drawing attention to the dangers and pitfalls that may be encountered. Extensive use is made of several case histories for this purpose. The paper concludes with some recommendations regarding borehole investigations in lateritic soils.

2.0 GENERAL DESCRIPTION OF LATERITIC SOILS

Lateritic or 'cabook' soils fall within the broader category of residual soils which lie above the parent rock as a product of the in-situ weathering of the rock. The process of laterisation occurs in hot-wet climates often found in the tropics where the water percolating downwards during the rainy weather removes the soluble salts leaving behind the residual iron and aluminium oxides which give these soils their characteristic colour and mottled appearance.

In a study of Sri Lankan laterites, Tennekoon et al.(1986) report that in most of such soils tested there is very little silt content and the soils have a concretionary structure. They postulate that it seems possible that most of the silt has been removed by the percolating water during the rainy seasons, and the clayey particles together with the sand and remaining silt are bound into the larger concretionary aggregates by the cementing action of the iron oxides produced by leaching.

This physical model helps to explain some of the engineering properties usually associated with lateritic soils; e.g.

(i) The laterites have high in-situ permeability and are well drained in nature. Hence, these soils are usually subjected to loading under 'drained' conditions.

(ii) Hard laterites which are usually found above the zone of fluctuating water table have strength and deformation characteristics that are similar to those of 'cemented materials'. Thus, in the hard laterites,

(a) cohesion predominates, and the stress-strain relationship is linear, followed by a brittle mode of failure. Therefore, the usual bearing capacity formulae are not applicable in such soils.
(b) very nearly vertical cuts can be made in such soils.
(c) the soils are capable of withstanding some tensile stresses.

(The above behaviour is also partly due to the apparent cohesion arising out of soil suction or negative pore water pressures. This apparent cohesion would get destroyed with the rise in ground water level.)

(iii) soft laterites exist in the region of fluctuating ground water level.

(iv) Decomposed rock is found below the lowest ground water level. It is found that the amount of weathering could vary considerably both with location and depth. The weathered material could vary from kaolinised clay at one extreme to unweathered boulders or corestones in a matrix of soil at the other.

These properties give rise to the questions of

(a) applicability of classical Soil Mechanics theories;
(b) reliability of laboratory test results based on limited number of tests;
(c) reliability of laboratory test results based on small diameter samples.

3.0 SITE HISTORY

A study of some of the Case Histories has shown that there are some aspects which are critically important for safe foundations, and which can be easily missed in a routine site investigation programme. A knowledge of the previous history of the site would greatly assist in overcoming these problems.

3.1 <u>Use of site for preparation of cabook blocks</u>

During the construction of a 3-storeyed hostel at Kelaniya on a lateritic hill, there was after some heavy rains the 'wash away' of a portion of a retaining wall at the boundary and the subsequent collapse of a part of the foundation of the building under construction. Subsequent studies showed that the failure surface passed through some loose material found in pits. On speaking to the residents of the area, it was revealed that the site had been previously used for the preparation of cabook building blocks. Such information can be easily missed even with a detailed investigation involving a number of boreholes, and this type of information is best obtained during the reconnaissance survey.

3.2 Site as part of a levelled hill

3.2.1 The site investigation requested by the Engineer for a single storey factory building at Horana was the carrying out of a plate loading test at one of the excavated foundation pits in order to check the allowable bearing capacity that had been used in the design.

During the testing, it was observed that the site was part of a levelled hill and that in some other foundation pits which had been excavated and in which the concrete screed had been placed to receive the foundation reinforcement, the excavation had not proceeded upto the virgin ground. Consequently, the following recommendations had been made to ensure that there would not be any subsequent distress to the building:

(i) examine each and every single foundation pit; and

(ii) ensure that at every foundation pit, the excavation had been taken down to virgin ground.

3.2.2 The site investigation for a 4-storeyed Library building at Moratuwa consisted of advancing 3 boreholes. The site was part of a lateritic hill, and no filling was identified in any of the boreholes. The soil to a depth of 5 m was identified as a soft laterite in which the SPT values ranged between 2 and 6.

When excavating for the foundations, it was found that at a part of the site the foundations would be on fill at the designed foundation depth. Instructions were then given to take the foundations down to the virgin ground, and at one location this was so deep that it was possible to include a basement for the building.

Increasing the number of boreholes would not have been a satisfactory solution because although increasing the cost of the investigation, there was no way of distinguishing between the fill and the virgin ground since similar SPT values would have been recorded in both the fill and the soft laterite.

3.2.3 On the other hand, there was another site at Sapugaskande where the sub-surface soils consisted of a hard laterite in which the SPT values ranged between 11 and 19. Prior to the investigation, it was known that the site was a levelled hill and using the SPT values it was possible to distinguish between the fill and the natural ground. Hence, the results of the SPT test were made use of to estimate the thickness of fill across the site.

4.0 STAGES OF A SITE INVESTIGATION

Geoguide 2 (1989) published by the Geotechnical Control Office, Hong Kong recognises several stages of a site investigation starting from the initial Project Conception stage to finalised Design stage of the Project. This manual goes on to state that recording of actual ground conditions during construction and monitoring the behaviour of structures after construction are further investigation activities which can be used to update the knowledge of the site conditions.

The objectives of a site investigation during the design stage could vary considerably as indicated below.

4.1 Investigation during Project Conception Stage

Laterites generally exist in tropical hilly formations with high ground and valleys. When the valleys consist of low lying areas which are marshy, the type of construction and the cost of foundations are considerably influenced by the nature of the marshes. Therefore, investigations during the Project Conception stage are aimed at determining

(i) the extent of the marsh;

(ii) the nature and thickness of marsh deposits;

(iii) ground improvement techniques which may be necessary in the marsh.

Therefore, from economic considerations, the investigations in the marsh consist of a combination of boreholes and the less expensive Mackintosh Probing tests. Details of such investigations are given by Tennekoon et al.(1996).

4.2 Investigations for finalised Design of Project

Laterites are found to be generally suitable for supporting many multi-storey buildings, and hence many structures on lateritic formations are constructed on shallow foundations. Therefore, site investigations are usually planned assuming that shallow foundations are feasible.

But there are instances when piling has to be undertaken to support some very heavy loads. The magnitude of these loads and the location of these loaded areas should be known prior to commencing the site investigation if problems are to be avoided.

4.2.1 The first example given in this section concerns the construction of a factory at Galle. At the time of the site investigation, the scope of the investigation was to provide values for the allowable bearing capacity for shallow foundations. Five boreholes had been advanced upto rock within the large site, and the depth to rock at these locations lay between (20 - 25 m).

Subsequently, when doing the designs, the Design Engineer decided to use pile foundations end bearing on rock for supporting a very tall chimney. When the piling actually commenced, it was found that even after drilling 25 m, hard basement rock suitable for carrying load in end bearing was not reached. A second site investigation was then carried out at this location, and it was found that hard basement rock in which good core recoveries could be obtained was at a depth greater than 36 m. Consequently, the piles were re designed as friction piles so that the piles could be terminated at depth of about 25 m.

4.2.2 The second example given in this section was similar in that initially a site investigation programme was undertaken at Homagama on a lateritic hill, several acres in extent; and the scope of the investigation was to provide design recommendations for shallow foundations. Three boreholes had been advanced upto rock. After the investigations were complete, structural details of the factory building became available, and these showed some very high loads in part of the factory area and these required pile foundations taken to rock. The depth to rock at the closest borehole was found to be about 4 m, and therefore the decision had been taken to excavate upto rock and build the column base directly on the rock. After excavating 4 m at one such location, rock had not been reached and probing with a steel rod showed that the depth to rock at that location was at a depth of more than 7m.

Therefore, the decision was taken to change to pile foundations. A second investigation was then undertaken within this area with 6 additional boreholes advanced in and around the highly loaded area. In each of the boreholes, rock coring was carried out to a depth of 5 m below rock surface. The second investigation showed that

(i) within the highly loaded area, the depth to rock varied between 1.2 m and 14.5 m; with the rock slope changing considerably due to the weathering process.

(ii) corestones were found in some of the boreholes.

The conclusion to be drawn is that if piling is anticipated in lateritic formations,

(a) boreholes should be planned close to the highly loaded areas;

(b) the boreholes should penetrate (3-5) m into rock at these locations to ensure that the hard datum reached is basement rock and not a boulder;

(c) rock cores should be obtained to determine the strength of the rock;

(d) estimation of the depth to rock at locations away from the borehole (especially in lateritic hills) should be treated very cautiously as this could vary considerably even over short distance.

5.0 LABORATORY INVESTIGATIONS AND THEIR RELEVANCE FOR LATERITIC SOILS

A feature of lateritic soils is that they are extremely variable in composition. Further, in many cases they consist of gravel, cobbles (and even boulders or corestones) in a matrix of sand, silt and clay.

Therefore, when investigations are carried out in these soils by obtaining undisturbed samples, these are two major differences from the sampling and testing of clayey soil, which have been formed by sedimentation. These are

(i) because of its larger variability, many more samples of these soils should be tested to obtain the some reliability;

(ii) accuracy of results can be improved by using larger diameter samples.

For examples, if triaxial testing is to be done, the practice in Hong Kong is to use 100 mm diameter samples in preference to the standard size of 37.5 mm diameter.

Therefore, if undisturbed samples are to be taken, a minimum diameter of borehole of 100 mm would be required (as against a borehole diameter of 75 mm which would otherwise be sufficient.) The increased diameter of the borehole increases the cost of boring and the larger diameter should be used only for the category of problems where laboratory testing is essential

One such category is when there is a need to check the stability of a slope.

The strength parameters ($\bar{c}, \phi$) necessary for carrying out an effective stress analysis are best determined from laboratory tests.

6.0 THE STANDARD PENETRATION TEST AND ITS RELEVANCE FOR LATERITIC SOILS

6.1 General

In sands and other cohesionless materials, the SPT is found to be a very reliable empirical test done inside a borehole from which estimates of allowable bearing capacity, settlements, friction angle, etc. can be obtained. It has been found that site investigation practice in this country relies heavily on the SPT value in lateritic soils for use in engineering design. These empirical results require to be laid down in the form of a guideline which can then be improved. One such attempt has been made by Varghese (1983).

6.2 Classification of laterites

It is found that laterites can be broadly classified as Hard Laterites, Soft Laterites and Decomposed Rock. Some typical values for the SPT values are as given in Table 1.

Laterite Type	Average SPT values
Soft Laterites	(2 - 15)
Hard Laterites	(15 - 30)
Decomposed Rock	> 30

Table 1

6.3 Allowable bearing capacity

A study of several Site Investigation reports has shown that the allowable bearing capacity recommended for lateritic soils is as indicated in Table 2.

Laterite Type	Allowable bearing capacity (kN/m^2)
Soft Laterites	(65 - 140)
Hard Laterites	(140 - 200)
Decomposed Rock	> 200

Table 2

6.4 Shear strength parameters

As mentioned previously, the shear strength parameters are best determined from laboratory tests. However, as a first approximation, these could be estimated as indicated in Table 3.

Laterite Type	c'(kN/m^2)	ϕ' (deg.)
Soft Laterites	(0 - 60)	< 20
Hard Laterites	(20 - 125)	< 30
Decomposed Rock	(50 - 100)	< 35

Table 3

The lower values of cohesion can be taken when the ground water level is near the surface, and the higher values of cohesion would be applicable when the ground water level is well below the surface.

7.0 RECOMMENDATIONS FOR BOREHOLE INVESTIGATION IN LATERITIC SOILS.

7.1 Number of Boreholes

The number of boreholes should be decided based on the nature of the investigation. The ICTAD Publication on 'Guidelines for Site Investigations for Foundations of Buildings' state that 'the cost of adequate exploration is low when compared with the total value of the work, but varies with the type of structure and the nature of the ground, a usual figure being the order of 1 percent of the cost of building. However, it must be understood that the technical requirements of the investigation should be the overriding factor rather than the cost'.

7.2 Position of Boreholes

When the investigations are for shallow foundations, the boreholes can be distributed uniformly across the site. However, when the investigations are for pile foundations, the boreholes should be located as far as possible close to the pile locations.

7.3 Depth of Boreholes

When the investigations are for shallow foundations, one of the boreholes can be taken down to rock or a maximum of 20 m; the other boreholes can be terminated at a depth of 10 m.

When the investigations are for pile foundations, the borehole must penetrate (3-5)m into the rock.

7.4 Diameter of Borehole

If undisturbed samples are to be taken, then the minimum diameter of borehole should be a little more than 100 mm. Else 75 mm diameter boreholes would be sufficient.

7.5 Method of Boring in overburden

Rotary drilling methods are preferred as they can penetrates any corestones which may be encountered.

Shell and auger boring can also be used, the disadvantage being that if corestones are encountered then chiselling has to be resorted to in order to penetrate the hard layer. If penetrating the hard layer is difficult, then another borehole would have to be done after shifting the machine.

7.6 Method of Drilling in rock

Rotary drilling is recommended so that

(i) rock would be penetrated by (3-5) m;

(ii) rock cores could be recovered

7.7 Undisturbed sampling in overburden

The 100 mm diameter Open Tube Sampler or the Denison Tube Sampler is recommended.

7.8 Sampling of rock cores

The triple tube core barrel is preferred. Alternatively, double tube core barrels which would give a minimum diameter of rock core of 40 mm should be used.

7.9 Ground water levels

Ground water levels can be observed during the investigation. But consideration should be given to the fact that the water levels could change with the outside weather conditions.

Investigation of Kabaragala Rockfall cum Debrisflow

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Abstract

This paper highlights investigations carried out by the National Building Research Organisation (NBRO) after the occurrence of a rockfall cum debrisflow at Kabaragala (Kegalle District) on 12.04.96. The mechanism of the development and the possibilities of recurrence as revealed during the investigations are discussed. The investigation had been carried out by NBRO on the request made by Provincial authorities. As in most other cases the debris flow was found to be confined to a gully but still the flanks are covered with unstable mass. So further growth of the debris flow in future can be expected. The investigations have been focused on demarcating the vulnerable zones for implementation of control measures by the provincial authorities in order to reduce the damages and also to take appropriate mitigatory actions in time.

Introduction

The village of Kabaragala is located about 12 km. away from the Aranayake town within the Aranayake A.G.A. Division of Kegalle Administrative District. The distance to the location of debris flow from Aranayake town is 8 km. on Aranayake Ambalakanda road. (Fig. 1 - Location Map)

The rockfall occurred on 12.04.96 around 15.30 daytime initiated from the Kabaragala escarpment by destroying 4 houses completely and 4 houses partially. No deaths reported except minor injuries caused to a child who was trapped among the debris before a rescue action had been taken by the neighboures.

History of the Slope

The failed slope is the southern escarpment slope of Kabaragala mountain of which the peak elevation is extended up to 970 MSL (Plate 1). The regional geological structure, (according to the geological information published by the Geological Survey and Mines Bureau - GSMB) and mountain forming rocks are associated with the formations of the highland series. The encountered rock within the area is biotite gneiss.

Minor rockfalls in this area have been the common hazard during the last two decades. But no incidents of serious nature have been reported. The gulley along which the debris flow has been developed was a perennial water way which carried only little surface water. The rainfall of 55mm recorded on 12.04.96 has triggered the rockfall. The recorded average rainfall of the month is 655.12mm. (This data have been provided by the Meteorological Department according to the measurements of rainfall at the gauge station at Kelly Estate.) This is believed to be the longest debris flow in the recent history. The length of the flow path is nearly 2km. and the approximate total area involved is 0.075 Km^2. The volume of rock debris carried by the rockfall approximately is about 66,000m^3 and deposited over an area of 0.042 Sq.Km. on the lower parts of the mountain slope.

Method of Investigation

The investigations have been commenced on 30th April 1996 on the request made by the A.G.A. Aranayake. A brief survey has been carried out to identify the areas vulnerable to further developments and potential of occurrence of similar debrisflows in the immediate vicinity. A rough estimate of the damaged property and infrastructure facilities as well as an environmental impact assessment were carried out to facilitate provision of relief measures for establishing the normalcy after the incident.

At the 2nd phase, study of aerial photo sets of scale 1:20,000 was carried out after which a mosaic has been prepared to a scale of 1:500. The morphological features have been transferred along with regional tectonic features to a ABMP map sheet 1:10,000. Geological data and regional structures have been transferred from the published data by the Geological Survey and Mines Bureau (1:63,360). The above data have been verified during the field mapping exercises and field measurements were incorporated into the map.

Detail mapping has been done along the flow path and joint measurement have been plotted on the Equatorial equal-area stereo net to obtain the prominent joint sets. The joint fill material samples and samples from the matrix of the colluvium layer have been collected for obtaining the geomechanical characteristics. Random rock samples were obtained for obtaining the average uniaxial compressive strength (UCS) of the exposed rock by point load testing.

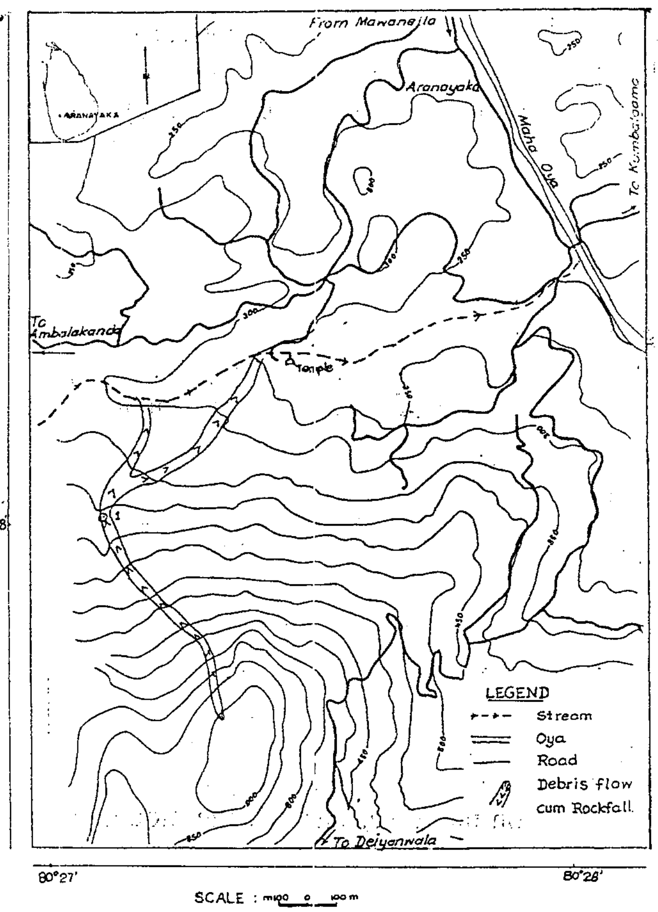

Fig. I Location Map

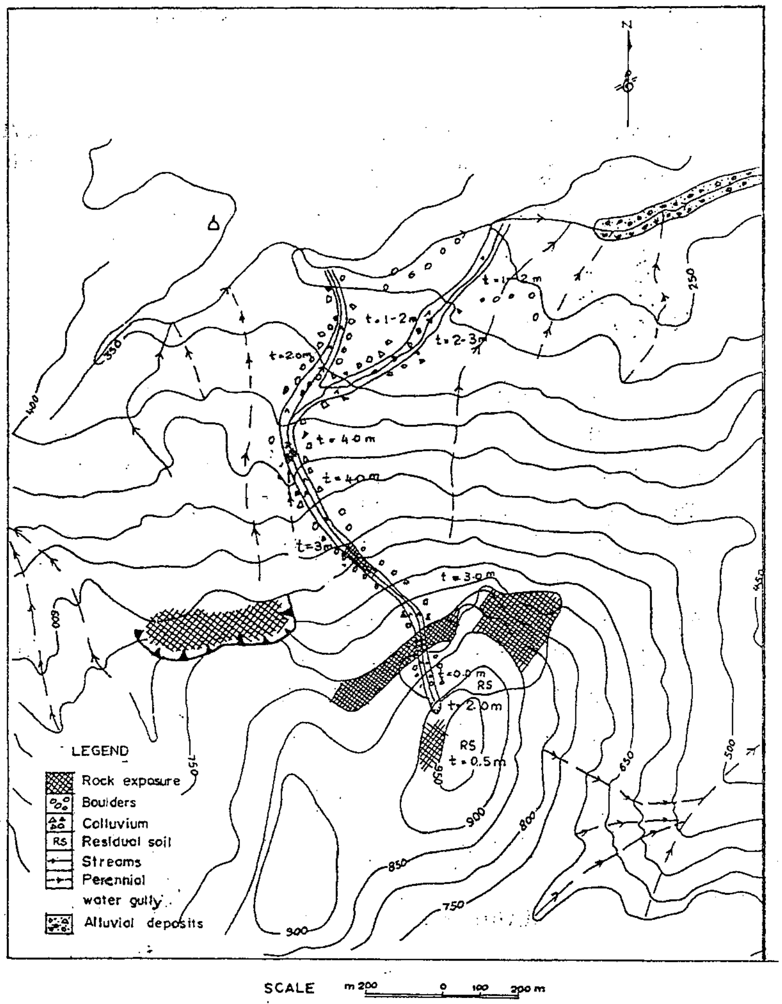

Fig. 2 Overburden deposits and Hydrology

Surface deposits .

The crown region of the debrisflow consists of 1-2m thick residual soil overburden which is underlain by a massive biotite gneiss rock. The same rock is exposed on the Kabaragala escarpment. This escarpment extends several kilometers horizontally and exposed at several locations on the hill slope. Due to the occurrence of debrisflow, perennial gully has transformed into a incised deep gully with flanks sometime high as about 5-10m vertically. The debris which carried along the path has been deposited on several breaking slopes and spread over a considerable area of about 100-200 m^2 broadly around the flow path(Fig. 2.). These deposits consist of boulders of different shapes. Among the debris Subrounded, angular and rectangular shaped boulders with volume variations of $1m^3$ - $50m^3$ can be observed. Material encountered in the immediately adjacent area on both sides of the debrisflow path consists of thick talus deposits including surficially distributed huge boulders from head to toe region.

Bedrock - geology

The rockfall area is located on the western limb of a Double plunging synform which is known as Aranayake Arena. Most of the rocks which are associated with this area belong to the highland series (Fig. 3). At the crown of the debrisflow, rock formations show the presence of minor folds of highly deformed nature. The adjacent rock formation at the same elevation which can be seen within a length of about 1Km. spread area horizontally, show well developed open macro joint systems (Plate No. 2). Mainly the bedrock formations of biotite gneiss is associated with the debrisflow. Foliation strike of the bedrock formation is 164° and dip angle is 73°E according to the equal area stereonet analysis.

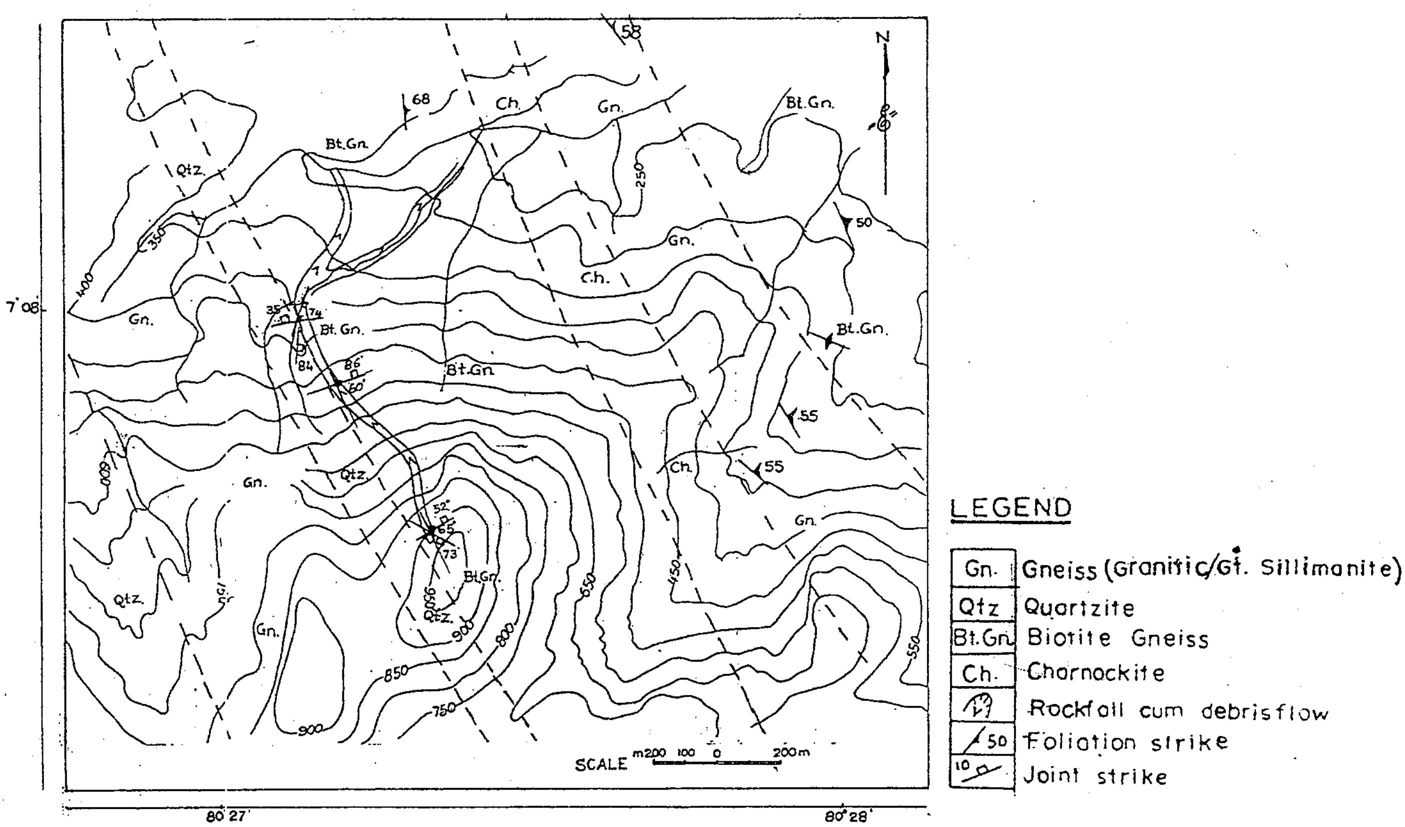

Fig. 3 Geology Map of the vicinity of the rockfall cum debrisflow at Aranayake

[Resource Map: Geology of Kandy (1" : 1 mile, manuscript) Geological Survey Department, 1971 - 1972]

Studies carried out using the equal area stereonet plots show 3 prominent joint systems and they are identified as (Fig. 4 & 5).

Strike	Dip-direction	Dip-angle	Referred Joint System
071°	161°	34°	1
087°	177°	88°	2
008°	278°	86°	3

The debrisflow originated at the head region and moved down through a narrow path. Material on the flanks of flow path consist of highly disintegrated insitu boulders which are mostly of unstable nature and liable to move downwards. The open fissures within the disintegrated boulder terrain contains gouge materials very often terrain which has been intervened by the root systems of large trees. This may have influenced by the rapid mechanical/chemical weathering which contributed to the reduction of stability of the rock mass.

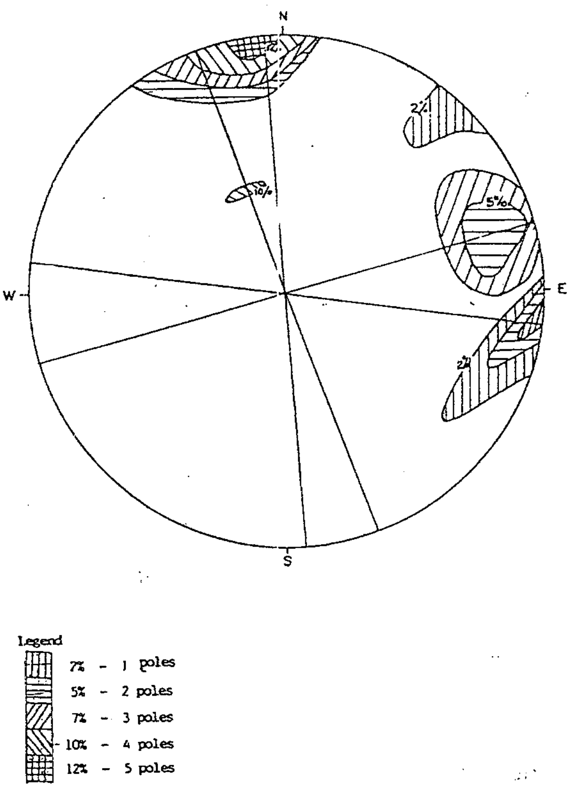

Fig. 4 Equal area streo plot contour diagram developed from the pole plots.

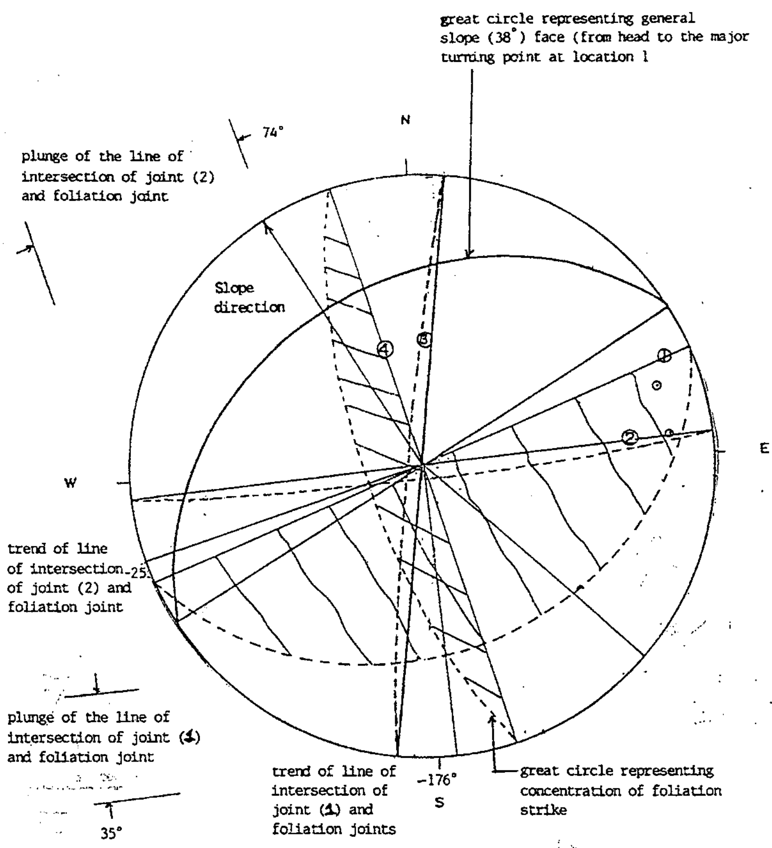

Fig. 5 Equal area stereo plot of structural conditions developed using contour diagram in the Fig. 4

The exposed bed rock along the debrisflow path show steep dips and well developed sharp joint faces. The pegmatitic intrusions of the bed rock exposures display differential weathering patterns and of highly deformed nature. (Plate No.5 & 6). In the lower part of the debrisflow, mainly after the turning point at location 1 (see location map) no exposure of bedrock could be observed. This area is covered with a thick colluvium cover which consist of rock debris (>1m).

Hydrology

The combination of dendritic & Rectangular drainage patterns is prominent in the area and the main river called Maha-Oya follows in conformity with the foliation strike of the bed rock. Few poorly developed gullies can be seen on the mountain slopes. The sudden appearances of huge water courses are common especially in the toe region & people use to collect this water through springs and shallow wells for domestic purposes. Many of these water resources occur in the thick colluvium overburden but also rarely appear through large cavities or open fracture systems. As a result of the debrisflow, the gully has been converted to a streamlet which carries a considerable quantity of water over the newly exposed fresh rock surface. It is being discharged into a tributary of the Maha Oya at the valley bottom.

Laboratory Testings

The gouge samples from location 1 was subjected to laboratory testing & results are as follows. (See Fig.6)

Table 1

Sample No:	Clay Content %	Specific gravity	Average Moisture content	Classification of soil (BSTM)
FG. 1	8.0	2.81	16.3%	Silty sand with gravels.
FG. 2	13.0	2.74	22.7%	Silty sand.
FG. 3	7.0	2.80	13.4%	Silty sand with gravels.
FG. 4	11.0	2.87	19.8%	Silty sand. with gravels.

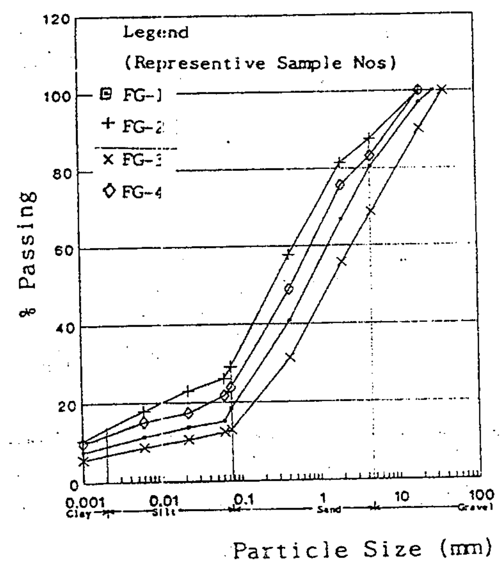

Fig. 6 : Grain Size Distribution of associated materials of the debris flow.

The point load tests were carried out for the irregular rock samples collected from the debrisflow. The results are summarized below.

Table 2

Type of work	State of weathering	Average Uniaxial Shear Strength (MPa)
Hb-bt-gneiss	Highly to Moderately	85
Bt-gneiss	Highly to Moderately	35

Discussion on the mechanics of the development of the rockfall cum debrisflow.

The problem of slope safety on a particular slope is greatly affected by the non-homogeneity anisotropy of the earthen mass as well as by the conditions in the environment. The validity of the results of analysis depends on the efforts made in presenting the true picture of the geological and geotechnical parameters associated with the site. Therefore important factors subjected to discussion are: initial stresses present in the ground, hydrogelogical conditions, the intensity of the influences of ground water and climatic forces which attacks the slope, other geological features in existance and changes of conditions of stress pattern prior to the failure.

The exposed fresh outcropping rock at the stream bed in the head region (break away zone) and the zone of transportation is represented by the main slip surface along which the debris mass has moved downwards. Therefore, it is important to have a sort of geotechnical zoning of the foundation rock mass to understand the factors contributing to the distablization of the upper slope.

The Geomechanics classification system (Bieniawshinata 1979) has been developed also for the assessment of foundation rock mass although it is mainly used for tunnelling applications. The scheme for classification with the use of rating values needs representative values for strength of intact rock material.

The UCS values obtained through point load testing shows that the values range from 35Mpa to 85Mpa in fresh rock. The estimated values for rocks represented in this case can be graded as highly weathered rock. The estimated values for fresh rock can be 2 to 4 times higher than above given values. Therefore rock mass rating varies between 15 to 7.

R.Q.D.

There are zones with closely to very closely spaced fracture zones due to intensive fracturing. These fractures form fracture systems in perpendicular as well as subparallel direction to the flow path. They have been developed in conformity with major regional fracture systems. Therefore the R.Q.D. values drops from 100-75 (in zones of wide and moderately wide fractured rock) to <25 (highly or closely fracture rock). Thus the rating drops form the value of 20 down to 3.

Spacing of joints

There is a direct relationship between the spacing and R.Q.D. As discussed earlier spacing of joints varies drastically depending on the representation values eg: for the zones with high R.Q.D.t. it is 2-3 m and where there are closely to very closely spaced fracture zones spacing of joints found to be less than 50 mm.

Condition of joints

Most of the joints found to be with gouge filling with the thickness of few mm(1-10) (sometimes more). Secondary mineralization along the fracturing, intersection of weak zones and pegmatitic intrusion developments and concentrations of micaceous minerals along the foliation planes and availability of soft wall rock on some of the joint planes reduces the rock mass rating in certain areas. The presence of clay gouge material (average clay content of samples 9.75%) influences significantly the shear strength of total rock mass. The shear strength of clay gouge material will gradually reduce due to progressive rearrangement of clay particles during shear displacement along the slip surface. A rock slope which may be stable at present also have undergone displacements over several decades as evidenced by the development of slicken slides within the clay gouge with clay. When the clay content is hegher as 25% or more probability for a failure along the same is higher which also depends on the thickness of the clay gouge.

Ground water conditions

The water pressure in a discontinuity plane directly counteracts the strengthening effects of normal stress patterns within the rock mass. Water pressure in a rock mass decreases the factor of safety. It causes failure due to relative movement along the discontinuity planes. Therefore, variations of ground water pressure developments play a major role in the stability of the rock mass as it reduces the friction resistance along rock separation planes. The flow path has been developed along a perennial water way. Apparently it had more ground water than the surface water prior to the occurrence of debris flow otherwise it cannot bring more water now than it was before.

Joint orientation

The orientation is reflected in the equal area stereonet plots (Fig. 4 & 5). This shows following results;

(1) One joint set (No.2) is in conformity with the flow direction **(165/75 E)**

(2) Another set (Set No.3) is subperpendicular to the flow direction **(278/86)**

These two joint sets created a block forming situation when intersected each other.

(3) The third joint set No.1 161/34 provided a possible slip plane (dip angle (25° - 30°) movement downwards along the plane of separation under gravitational forces.

From the discussion above it is evident that domains of different rock mass ratings where quality differ from extremely poor rock to fair rock exist. Therefore climatic changes can trigger a debrisflow due to the development of external forces associated with excessive pore water pressure so as to have the favourable conditions in the rock mass which can cause reduction of the factor of safety.

Conclusions

(1) The rockfall/debriflow thought to have been triggered by monsoon rain.

But rainfall measurements done by the Meteorological Department have confirmed that the occurrence of debrisflow cum rockfall was not due to exceptionally high rainfall. Therefore similar incidents can take place in future with low rainfalls within the area below the Kabaragala escarpment along the perennial water ways developed along gullies.

(2) Geotechnical conditions (decrease of the residual shear strength along the joint set parallel to the slope surface with pore pressure developments, relatively rapid weathering of overlain rock masses, fissure fillings and thick loose colluvium deposits on the slope) at the slope area below the Kabaragala escarpment are conducive to slope movement

(3) The efforts taken by NADSA to encourage villagers to grow suitable cash crops in home gardens, and preserve the thick vegetation cover seems to be prevented slope destabilization up to some extent. Reforestation should be encouraged and living on the slope should be discouraged.

(4) Unexpected movement downwards associated with loosened rock blocks or blocks of rock detached from steep rock escarpment is possible. The fall of large rock masses detached from a position high up on a mountain wall can attain a high velocity during free fall. The falling masses therefore possess a high kinetic energy and rock debris can scatter over a very large area at the foot even to the flat ground. This can result serious losses in future from the point of economic social and environmental.

(5) A systematic survey needs to be conducted using a network of measuring points laid out within the area. The monitoring is advisable to be carried out at regular intervals during the rainy periods.

(6) Natural water causes should not be allowed to be disturbed and restrictions should be imposed over the removal of well grown trees for commercial purposes.

Acknowledgements

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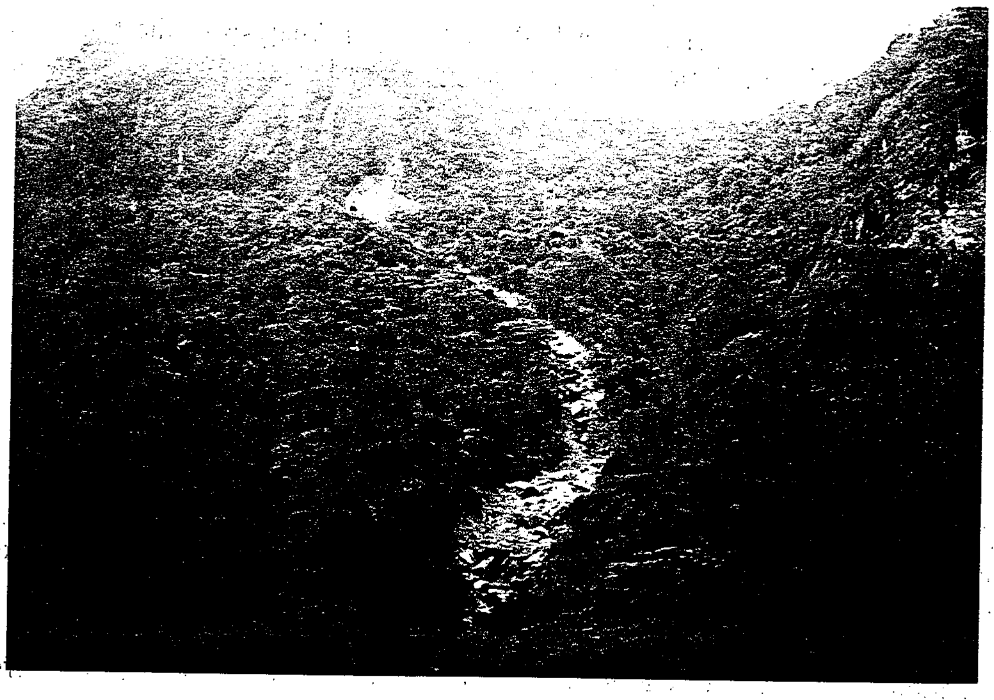

Plate 1 : An overall view of the rock fall cum Debrisflow at Kabaragala, Aranayake

Plate 2 : Existing macro joint systerms in the escarpment rock mass

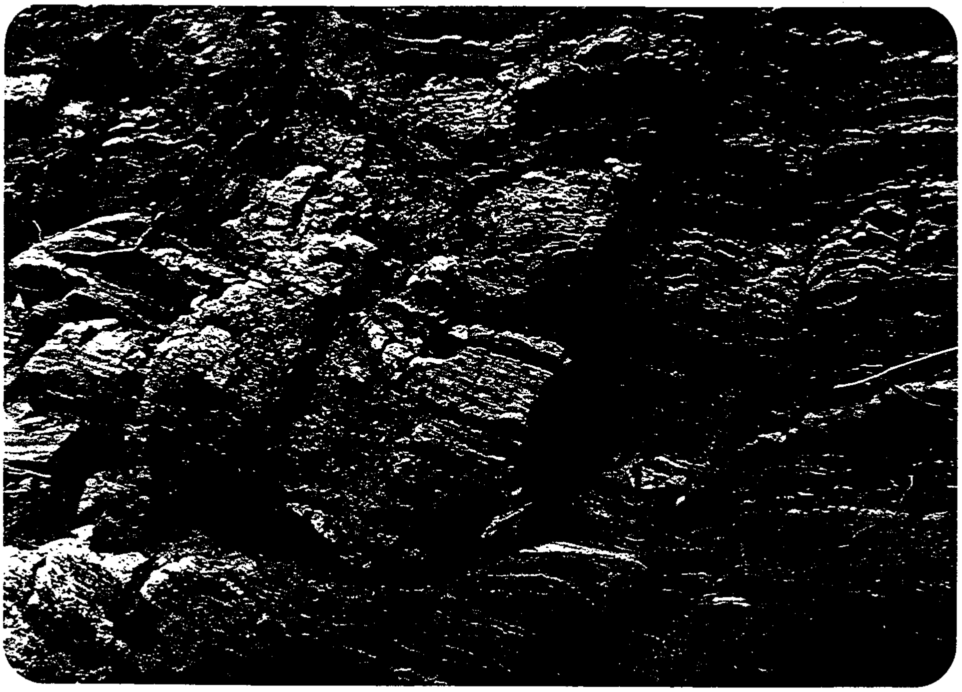

Plate 3 : Rock layering & discontinuties associated with the bed rock mass which is exposed on the debrisflow path

Plate 4 : Prominant joints (3 systerms) seen on the flanks of the newly created deep gully

Plate 5 : Joint fill materials consist of secondary formations and pegmatitc intrusions found along the discontinuties

Plate 6 : Thin layering & boudinar structures developed in the rock mass which might have contributed to formation of zones with differential weathering in bed rock mass

Determination of Piezometric Pressure Fluctuations in Construction

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Abstract:

In geotechnical engineering designs, constructions and monitoring activities, the Effective Stress Principle (Terzaghi, 1923) of saturated soil is the governing factor. The pore water pressure is one of the key parameter associated with above. In order to determine the pore water pressure at intergranular soil particle high degree of confidence and accuracy is necessary. Because of natural complexities and numerical difficulties of simulation of detail behavior of sub soil under differential loading and ground water flow, natural slope stability (instability predictions in deterministic approach) evaluation depends unavoidably on the high accuracy of interpretations of the fluctuations of piezometric pressure. Thus, large number of hydraulic, electrical;nowadays even in electronics and pneumatic devices are available to measure of their variations under deferent environmental conditions cum loadings. The monitoring of sub soil settlements in highway and embankment construction is equally important with piezometric fluctuation measurements. The paper highlights some of the piezometers and their performances in different ground conditions. A case on the effect of seasonal variation of rainfall and piezometric fluctuation in slope stability calculations has been taken as a model study.

1. Introduction:

The simplest method for determination of piezometric fluctuation is, through observations using an open stand pipe piezometer or an auger hole. However, observations may be misleading due to rainfall, surface runoff and aquifer recharge effects in open bore hole monitoring. Therefore, scientific assessment of piezometric pressure variations depend on instrumentation which is important when consider case by case. The reliability of water level observations in bore hole can be improved by using various and piezometer devices. The methods described in various devices, basically require ground water flow in or out, in the measuring system before the recorded pressure reach equilibrium status. The rate at water flow depends on the soil permeability and the response time of the measuring device. Therefore, selection of a measuring device will largely depend on the response time.

2. Requirements of Water Pressure Measuring Devices

The following requirements are important in selecting piezometric pressure measuring device.

i. Piezometer should response quickly to changing ground water conditions. i.e it should have a small response time and shall be able to use for measuring rapid changes in pore water pressure due to rainfall infiltration, changes in stress imposed by super imposed loads or excavations and pressure changes due to tidal variation etc.

ii. The pressure may be positive or negative at the piezometer tip and the errors in the readings should be within tolerable limits. (The air entry value of the tip need to be defined to ensure that the soil suction will not create full water out of the piezometer cavity. Hence the suction will not remove water completely from the pores of the tip their by prevent air entry from the soil through the tip).

iii. Irrespective of the principle of sensing pressure in the ground, falsification of reading should not be resulted.

iv. It should be rugged and reliable in the field condition and suitable for long time monitoring.

3. Piezometric Measurements can be used in: Site Investigation, Foundation Design, Construction and Performance Studies.

The areas of its usage are as follows,

(a) for locating 'perched' and 'Permanent ground water tables (if encountered within the depth of exploration) and estimating the seasonal variation of their levels prior to construction.

(b) to understand pore water pressure dissipation patterns and calculate in-situ effective stresses thereof.

(c) for revealing the presence of artisan pressures, their magnitudes and distribution.

(d) for sketching realistic flow nets in slopes, cuttings and fills etc.

(e) for estimating influence of

i. ground water lowering in the neighboring wells, etc.
ii. tidal fluctuations in the nearby creek etc.
iii. pounding of the adjacent low-lying area, or
iv. flooding from adjacent highlands, as the case may be.

(f) for forecasting potentiality or otherwise of sliding along old slop surfaces of landslides, shear zones, or any other discontinuities relevant to the site explored.

(g) for measuring ground water permeability.

(h) for studies of shear strength and volume changes as related to change of effective stresses.

(i) for determination of pore water rise or fall-consequent volume change-soil fabric inter relationships.

(j) in liquefaction studies & liquefaction-proof designs

(k) in effective stress analyses of all kinds of foundations, embankments, dams, slopes, cuttings etc.

(l) in estimating the quantity of seepage as related to design of filters, relief wells, cross-drainage works, controlling of de-watering operation

(m) for working out corrective-designs for foundations in distress

(n) in controlling magnitude, manner, sequence and rate of construction

(o) in examining the short-term design forecasts on the basis of the pore water pressures developed during construction

(p) in finding co-relations between pore water pressure and observed ground deformations, and may even be an aid to refinement in design as construction progress.

(q) in comparing the short-term, intermediate-stage and long term predictions of soil-foundation-structure-soil behavior with that calculated using the actual values of pore water pressures.

(r) for back-analysing the observational data with a focus on checking the validity of design assumptions, incorporate necessary revisions and to improve the state-of-the-art on the subject.

(s) for monitoring the efficacy of drainage systems etc.

4. Classification of Sub Soil

The classification of sub soil strata or the depth of soil horizon is some what important to predicting the selection of location and the depth at which soil horizon the piezometer tip is to be installed. Simplified arrangement of the soil horizons for geotechnical engineering is,

Horizon	Comments
A	Top zone consisting of topsoil and organic matter, and in humid areas, highly leached materials, in arid areas it may be rich in various water-soluble salts remaining as water vapor from the lower depths evaporates; it generally is highly weathered, dark-colored material including various shades of blacks and browns of a few centimeters to 1 or 2 m thick and grading into the B horizon.
B	Zone underlying the A horizon and containing considerable leached materials (water-soluble salts such as carbonates, sulfates, and chlorides) and clay minerals; this zone may be on the order of 0.5 to several meters thick and grades into the C horizon
C	Transitional zone of freshly weathered parent material (rock); it may consist of considerable rock fragments or may be absent or of very shallow depth and grades into the D horizon
D	Parent (or bed)rock

5. Classification of Engineering Problems

Most of the geotechnical engineering problems fall within one of the following two classes with particular reference to piezometer pressure developments.

A. **The problems in which the pore water pressure is independent of the magnitude of the total stress acting in the soil and is controlled by the ground water table or by the steady state of seepage flow corresponding to the stage prior to construction or that obtain in the long term.**

B. **The problems in which the pore water pressure depends on the magnitude of stresses acting on the soil, properties of the soil and the time lapsed since load application. This covers pore pressures setup up during construction and the time based rise or decreases towards the state of ultimate equilibrium as in Class A above.**

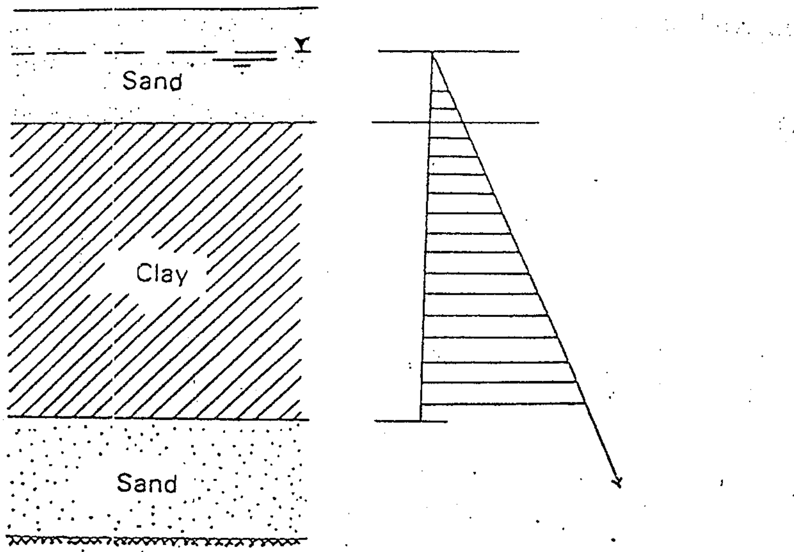


Fig. 1

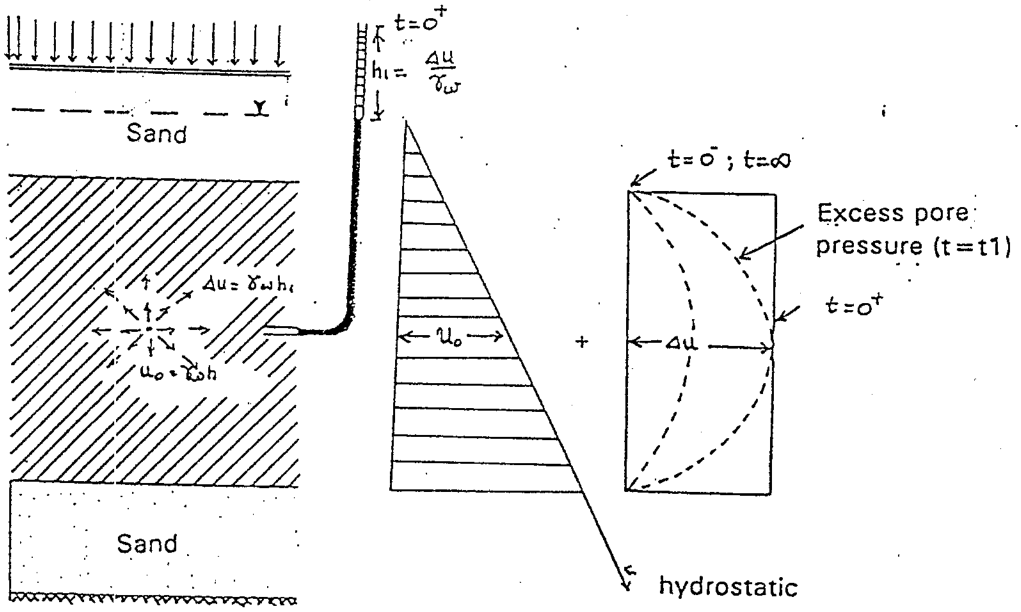


Fig. 2

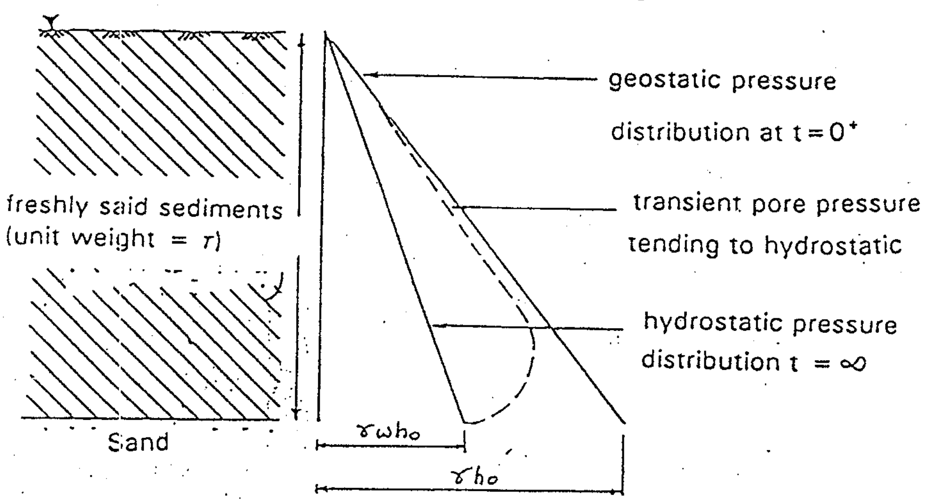


Fig. 3

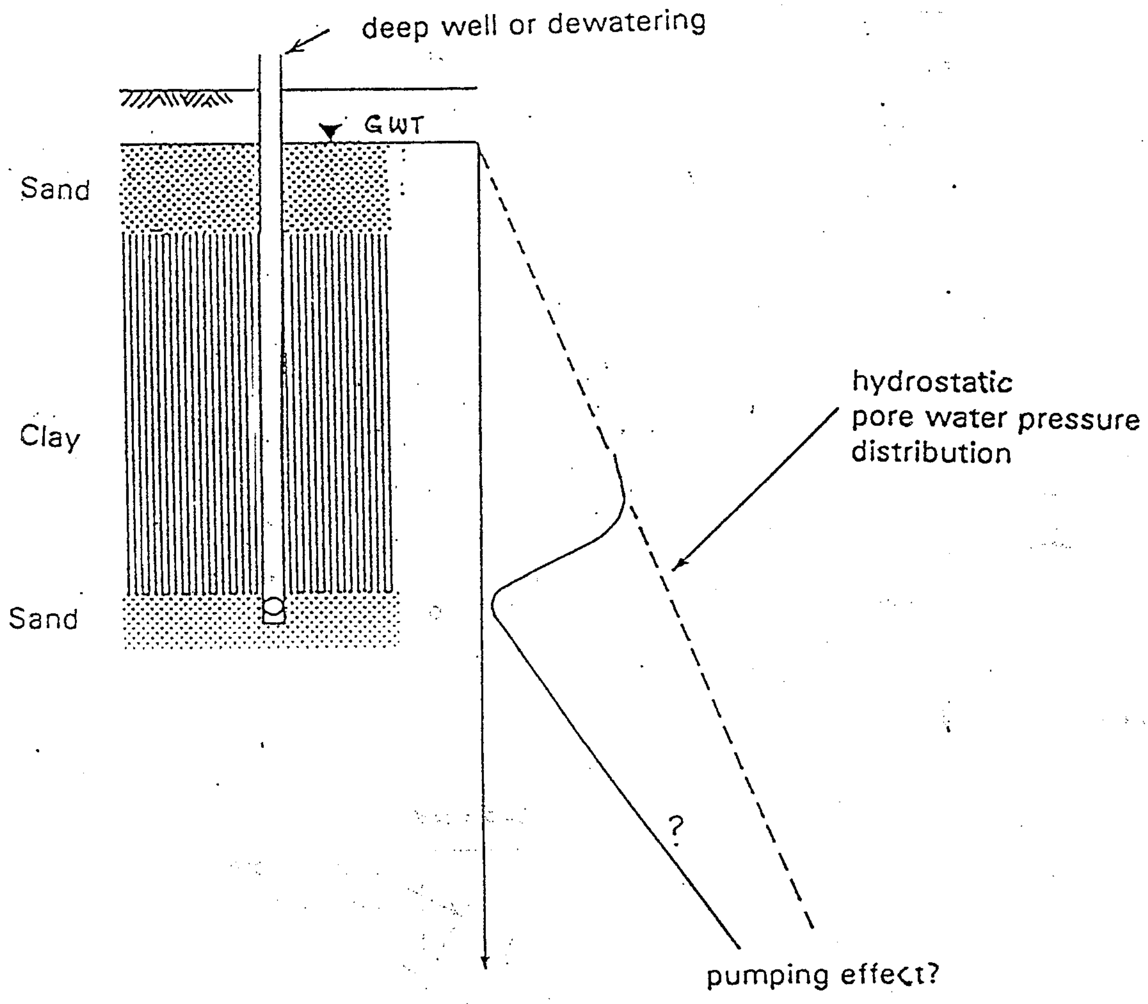


Fig. 4

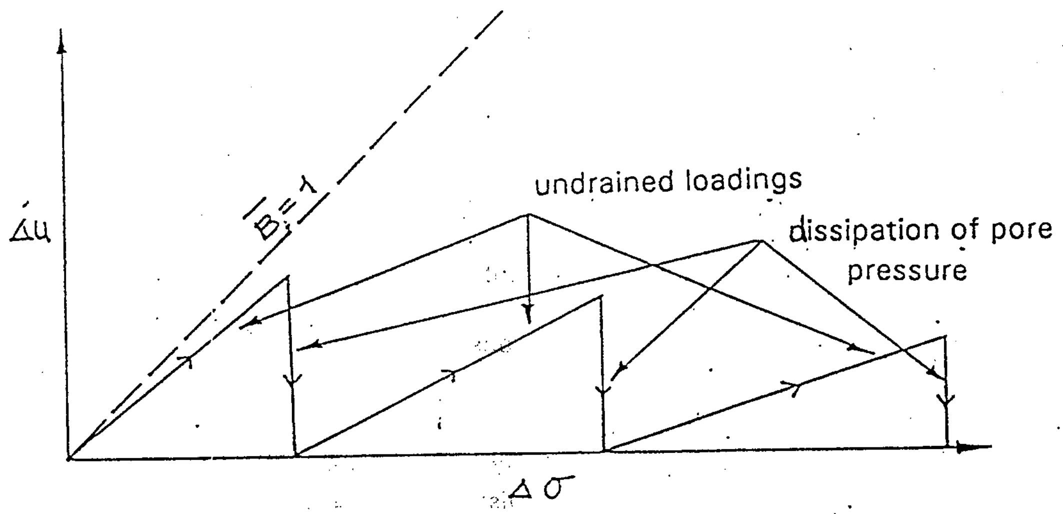


Fig.5a - Pore Pressure dissipation with time in steps of construction

5.1 Problems of Class A

For a soil profile which assume to be at the ground water equilibrium, one would normally expect the hydrostatic distribution of pore water pressure to be below the ground water table ,Fig. 1. Additionally, excess hydrostatic pressures would develop, if the soil strata is loaded. These excess pore water pressures would dissipate with time before acquiring equilibrium state once again due to hydrostatic pressure distribution, Fig. 2. The level of effective stress in the soil mass would thus increase. The same explanation would be valid for a fresh laid sediment deposit of a undergoing embankment fill gravitational consolidation with time. Initially, pore water pressures may be as high as the total stresses due to self weight, which would eventually dissipate down to the hydrostatic pressure level Fig. 3. When that happens the deposit could be assumed as 'normally consolidated'.

Unloading by way of erosion of an over-lying deposit, e.g., the sand layer in Fig. 2. would generate negative pore pressures in the deposit which will also dissipate down to the hydro-static pore pressure distribution level as the deposit swells.

Departure form the hydrostatic pressure distribution could also be observed in a situation resulting negative process for example continuous pumping, Fig. 4.

5.2 Problems of Class B

Every foundations or constructions on the ground generate shear and normal stresses within itself to a certain extent. Shear stresses are carried by the skeleton of solid particles where as normal components of the applied stresses are shared both by the soil skeleton and the pore fluids. The pressure in the free fluid tends to dissipate causing volume change. For an equal all round change in stress, this is expressed by the relationship;

$$v/V = C_c(\sigma\text{-}u)$$

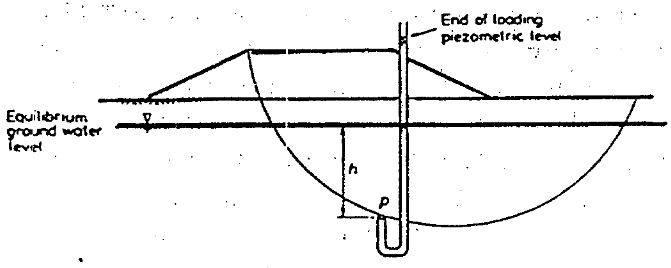
in which v/V = the volume change per unit volume of soil

σ = change in total normal stress

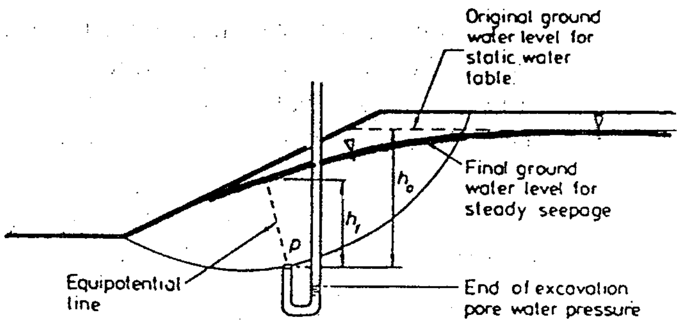
u = change in pre water pressure

C_c = compressibility of soil skeleton

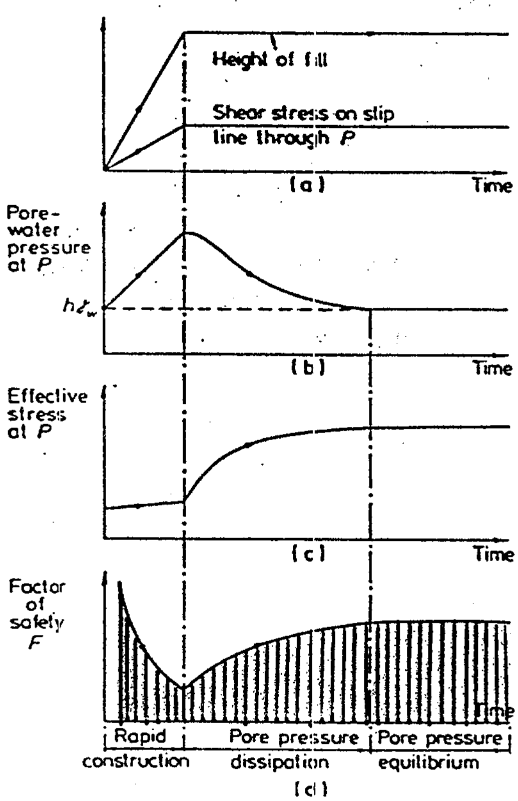
for a particular stress-range considered.



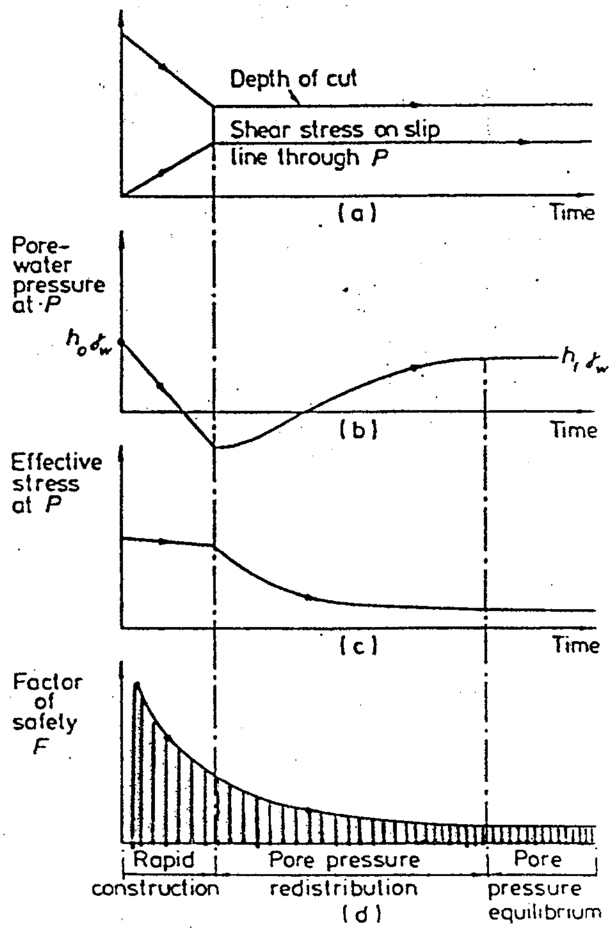
. Pore pressure generated on a potential slip surface by embankment loading.



. Short term and long term pore pressures in a cutting.



. Variation with time of the shear stress, local pore pressure, local effective stress, and factor of safety for a saturated clay foundation beneath an embankment fill (after Bishop & Bjerrum).



Variation with time of the shear stress, local pore pressure, local effective stress, and factor of safety for a saturated clay excavation (after Bishop & Bjerrum).

Fig. 5b – The effect of Pore water pressure during construction of the (a) embankment fill and (b) excavation

Changes in stress relationship is of practical importance i.e. a volume change will occur, without a change in applied total stress, if the pore water pressure undergoes a change. This is the reason why buildings founded on low permeability soils settle slowly and for a long time. This is also the reason that end-of-construction factor of safety, Fig-4, is lower than the long term factor of safety. The explanation for added settlements on account of ground water lowering is also provided by the above relationship.

High embankments and embankment dams are usually built in stages. The pore pressure (u) build up as the total stress ( ) is added and for undrained loading of a saturated clay the ratio would approach unity, Fig-5. It is an interesting finding that, if dissipation of pore water pressure is allowed after every stage of loading, two advantages would accrue. The first is at the rate of pore water pressure developments with loading would decrease and the second is the magnitude of over-all pore water pressure which would be much lower than that generated in a single stage loading.

6. Types of Piezometer:

Different types of piezometer are usually classified as
(a) Hydraulic (b) Electrical or (c) Pneumatic.

6.1 Hydraulic Piezometer:

6.11.Standpipe Piezometer(Open-Hydraulic piezometer)

The standpipe piezometer, Fig-6, perhaps better termed as open -hydraulic piezometer, consists of a tube with a porous filter element on the end that can be sealed into the ground at the appropriate level. It has a cylindrical porous element protected by a perforated rigid base of 35mm diameter and 300mm long. This element is connected to a 19mm or 25mm internal diameter pipe. The response time generally does not become a significant factor until the soil permeability is less than about 10^7 m/sec.

6.12. Hydraulic Piezometer(Closed-hydraulic piezometer)

The ground water level is detected in a small piezometer tip with porous wall, and conduct through small diameter plastic tubes to a remote point, where the pressure is measured, usually with a mercury manometer, Bourdon gauge or pressure transducer, Fig-7.

The limitations are de-air the system(flush out any air bubbles) before take readings, the measuring point and the connecting tubes should not be more than 7m, in order to avoid cavitation and regular maintenance. The main advantage are; simplicity during installations, small response time and that can be used for measuring rapid changes in pore water pressure due to rainfall, changes in stress imposed by super imposed loads etc.

The above described piezometers are due to US Bureau of Reclamation (Armstrong, 1945), Casagrande (1949); BRS, U.K. (Penman, 1956): NGI (Bjerrum et.al., 1960) and IC type (Bishop, 1960) are all hydraulic types.

6.2 Electrical Piezometer:

The electrical piezometer,Fig-8, basically consist of differential resistance measuring transducers or vibrating frequency detective transducers and a gauge. Therefore, simplicity during installations, the smallest response time compared to other devices, accurate negative pressure measurements and changes in stress imposed by super imposed loads etc.

Electrical piezometer have been described in the literature by Speedie (1948), Boiten and Plantema (1948) Plantema (1953), Burland (1967) and Bhandari (1970).

6.3 Pneumatic Piezometer:

The pneumatic piezometer, Fig-9, consist of a respond diaphragm which divide the in situ pressure development unit chamber and the pressure required for equalization chamber which directly connected to a pressure measuring device; mercury manometer or pressure gauge. This system is having definite advantage due to connecting tube may extends to longer length as required in a particular case.

7. Response time of a Piezometer:

The scientific meaning of the response time, Fig-10, of a piezometer is equalization of the out-of-balance pressure at the piezometer tip which requires certain quantity of water to flow in or out of the piezometer until its acquire the state of equilibrium. In fine-grained-soils, because of their low coefficients of permeability, the time requirement for complete response of pore pressure changes is high particularly where the piezometer is an open hydraulic type. The sensitivity of a piezometer is therefore measured in terms of response time which depends on,

(a) Compressibility or expansibility of the subsoil in which it is installed.

(b) Shape or intake factor of the piezometer tip.

(c) Quantity of water required to operate the device.

(d) degree of stiffness of the pore pressure sensor in the piezometer tip.

(e) Permeability at the installation depth of the ground.

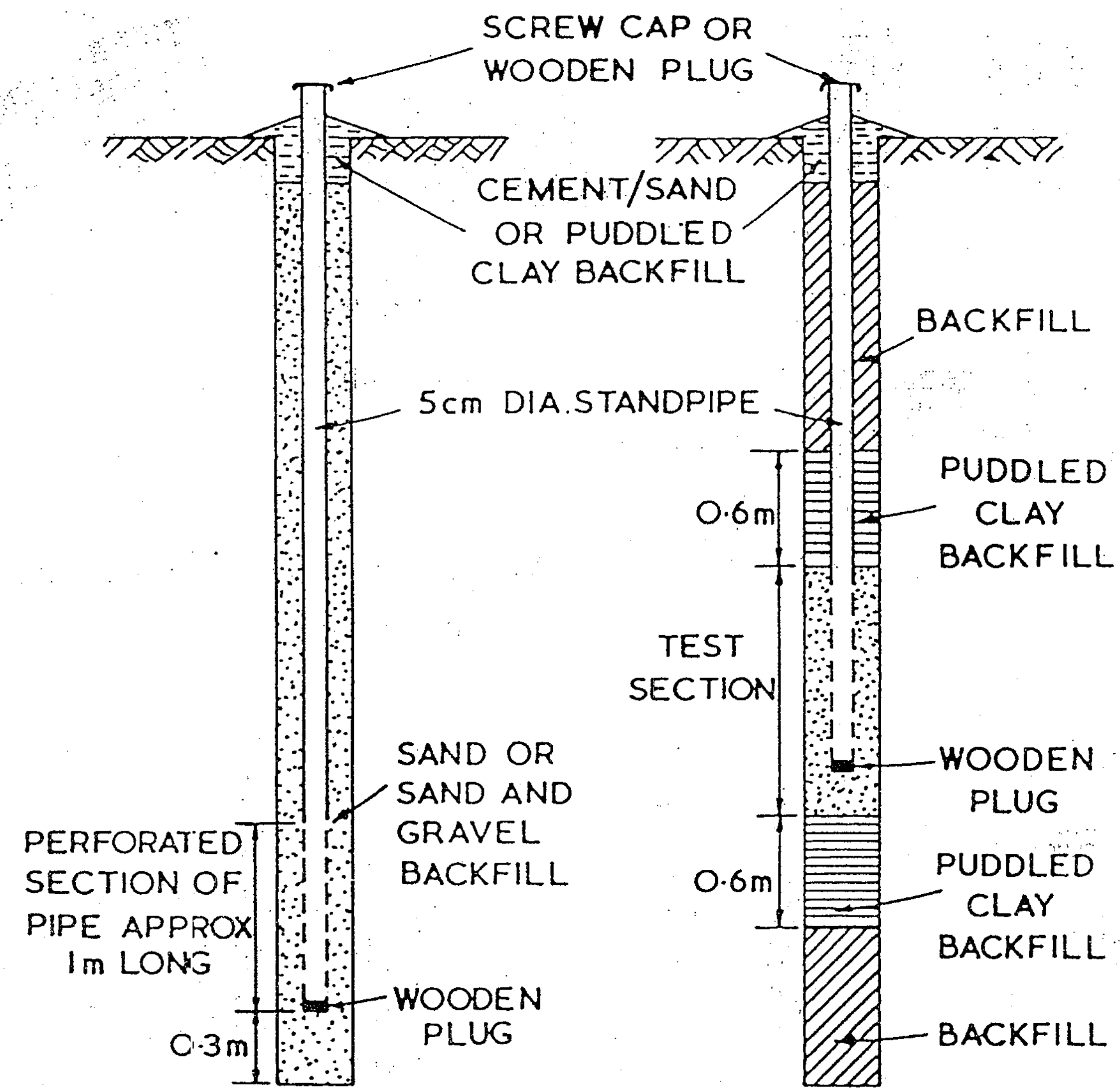


Fig. 6 - Detail of an open standpipe piezometer

Borehole installation of hydraulic piezometer tip

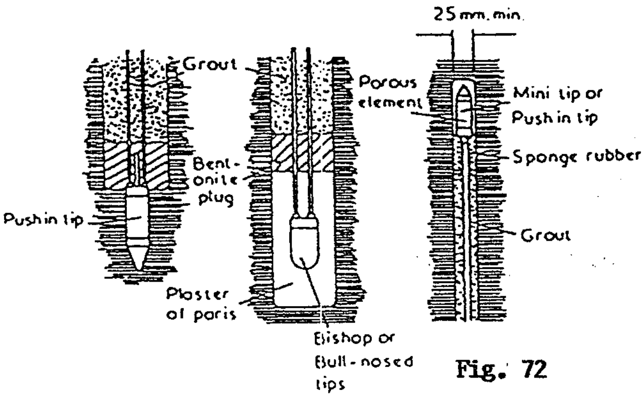


Fig. 72

Use of disc hydraulic piezometer

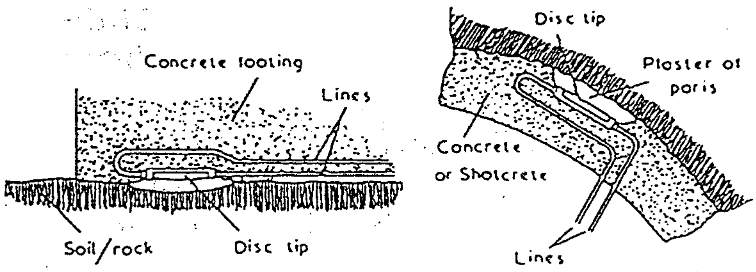


Fig. 73

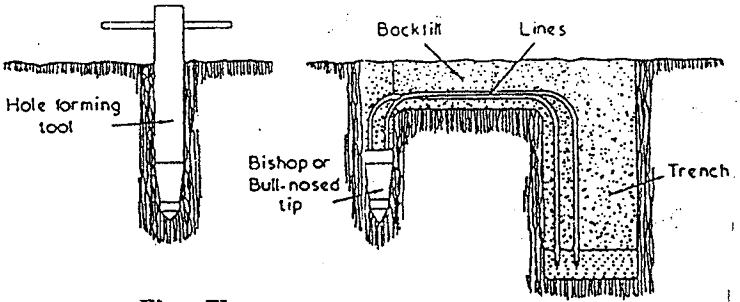


Fig. 71

Hydraulic piezometer

Fig. 7

Various arrangements of hydraulic piezometers in boreholes

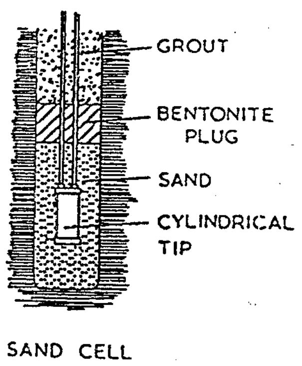


Fig. 7.4

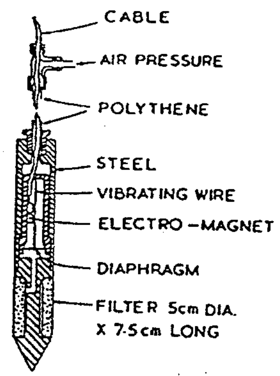


Fig. 8.2 r vibrating-wire

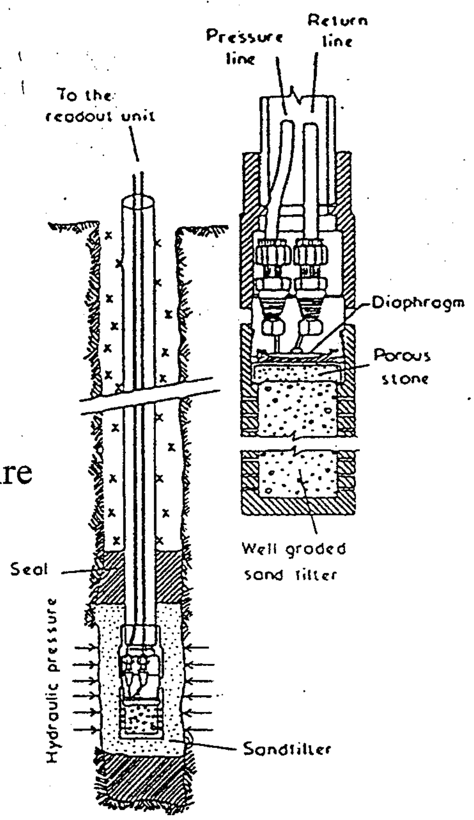


Fig. 9

pneumatic piezometer

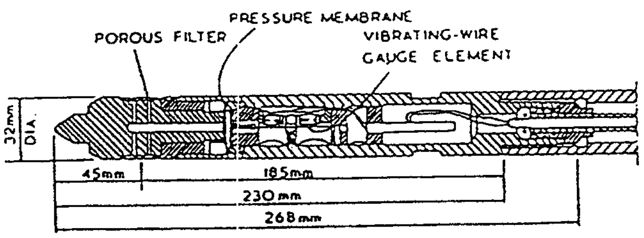


Fig. 8.1 BRS-Type vibrating-wire piezometer

7.1 Calculation of Response Time (days)

Hvorslev (1951) provided an approach to calculate response time assuming that the soil surrounding the piezometer tip is isotropic, incompressible and fully saturated. According to the approach;

$$\frac{P_o - P}{P_o} = 1 - e^{\frac{Fkt}{vxw}}$$

in which
F = Intake factor
K = Coefficient of permeability
P_o = Excess hydrostatic pressure causing flow at time t = o
P = Excess hydrostatic pressure causing flow at time T = t
w = density of water
v = Volume factor (volume of water flowing into the piezometer per unit change of pressure)

The value of **Fkt/vw** for various percentages of pressure equalization (Po-P)/Po % is given in Table 1.

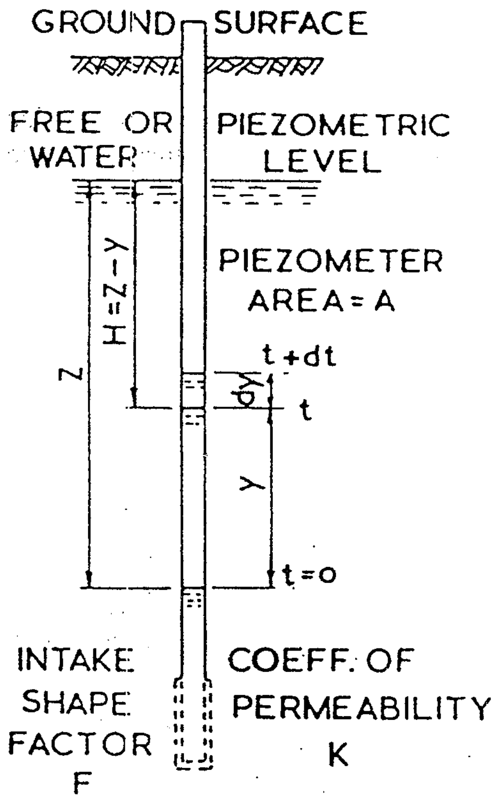
Table 1

Fkt/vw	0.11	0.36	0.69	1.20	2.30	3.00	4.60	6.61	9.21
P_0	10	30	50	70	90	95	99	99.9	99.99

The intake factors values for different tip geometries are given in the Appendix-A. The intake factors corresponding to different heights of the sand surrounding for borehole diameter of 0.16m are presented in Table.2.

Table 2

$L_{(m)}$	0.8	0.9	1.0	1.1	1.2	1.3	1.4	1.5
F_m	2.19	2.35	2.50	2.65	2.80	2.94	3.10	3.22



RATE OF INFLOW AT t

$$q = F.k.H. = F.k.(Z-y)$$

VOLUME OF FLOW IN TIME dt

$$q.dt = A.dy$$

AND

$$\frac{dy}{Z-y} = \frac{F.k}{A} dt$$

TOTAL FLOW FOR EQUALIZATION OF PRESSURE DIFF. H

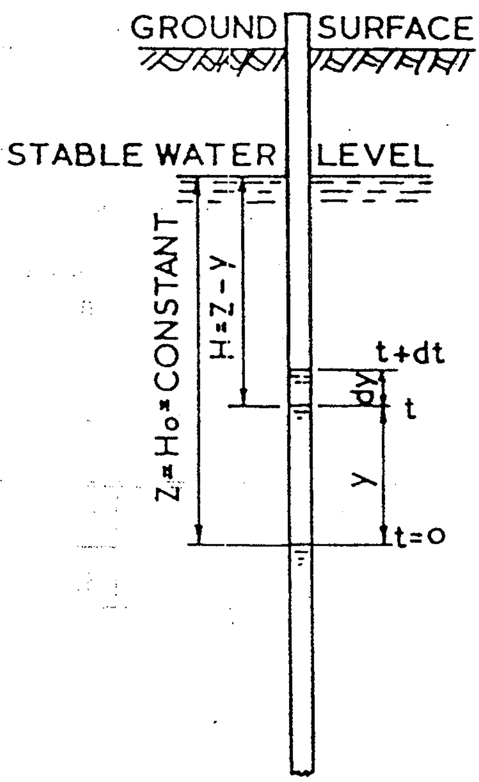
$$V = A.H$$

BASIC TIME LAG -T- DEFINED

$$T = \frac{V}{q} = \frac{A.H}{F.k.H} = \frac{A}{F.k}$$

DIFFERENTIAL EQUATION

$$\frac{dy}{Z-y} = \frac{dt}{T}$$



FOR CONSTANT OUTSIDE PRESS.

$$Z = H_o = \text{CONST.}$$

DIFF. EQN.

$$\frac{dy}{H-y} = \frac{dt}{T}$$

T=BASIC TIME LAG

TIME LAG RATIO

$$\frac{t}{T} = \ln \frac{H_o}{H_o - y} = \ln \frac{H_o}{H}$$

HEAD RATIO

$$\frac{H}{H_o} = e^{-\frac{t}{T}}$$

EQUALIZATION RATIO

$$E = \frac{y}{H_o} = 1 - \frac{H}{H_o} = 1 - e^{-\frac{t}{T}}$$

Fig. 9

Using Table 1 the response times corresponding to the various percentages of pressure equalization at different intake factors, Table 2 are calculated and results are given in Table 3.

Table 3: **Area of Standpipe = 2.01cm²; K = 1x10⁸ cm/sec;** Response time 't' in days corresponding to different 'F' values

$\frac{P_0-P}{P_0}$ %	F Fkt/a	**2.19**	**2.35**	**2.50**	**2.65**	**2.80**	**2.94**	**3.10**	**3.22**
50	0.69	7.32	6.76	6.4	6.0	5,70	5.45	5.15	5.00
90	2.30	24.5	22.7	21.4	20.2	19.2	18.2	17.2	16.6
99	4.6	49.0	46.0	23.2	41.0	38.4	36.4	34.6	33.2
99.999	9.21	98.0	91.5	86.0	81.0	76.5	73.0	69.0	66.5

According to the findings of Hvorslev(1951) equalisation of the order of 90% considered desirable in most practical problems. Lets take an example, if it was aimed at 90% of pore water pressure equilisation is reached within a period of say 20 days, a study of the tabulated values of intake factor and response time in Table 2 and Table 3 respectively, revels that surrounding height of 1.0m to 1.2m would yield the desired response.

8. Installation and Construction Procedures of an Open Standpipe Piezometer

A typical open standpipe is shown in, Fig.11. It consists of a piezometer tip and connecting standpipes. And observations are recorded by means of a Water Level Indicator. Since piezometer are read manually, one water level indicator can serve all the piezometer in the area.

Piezometer tips could be of several kinds but each one of them would essentially have a filter which could be of ceramic, carborundum, stainless steel, centered bronze or other synthetic materials. The filter is suitably housed in a metallic or plastic body with washers or a 'O' - ring seals to make it leak-proof.

Typical installation details of a piezometer adopted from a Casagrande (1958) is shown in Fig. 12. The dimensions of surrounding are the only indicative and must be worked out from the consideration of Intake Factor. Sealing of a piezometer against leakage is another important aspect. Installation of number of piezometer at different elevations in a single borehole, require a proper sealing of intermediate elevations as shown in the Fig.13.

In many situations, the outer casing could be dispensed, fore example, if a borehole could stand without caving, and if the a piezometer tip could be sealed effectively, a casing is unnecessary.

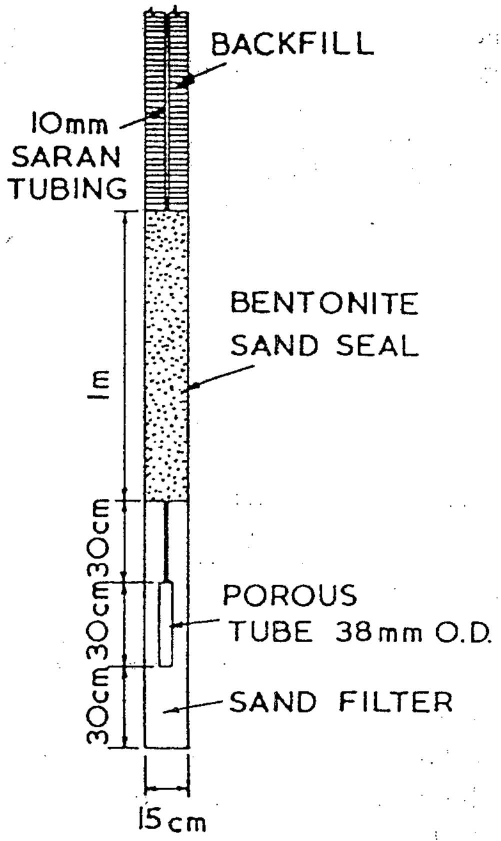

BACKFILL
10mm
SARAN
TUBING
BENTONITE
SAND SEAL
1m
30cm
30cm
30cm
POROUS
TUBE 38mm O.D.
SAND FILTER
15cm

Fig. 12

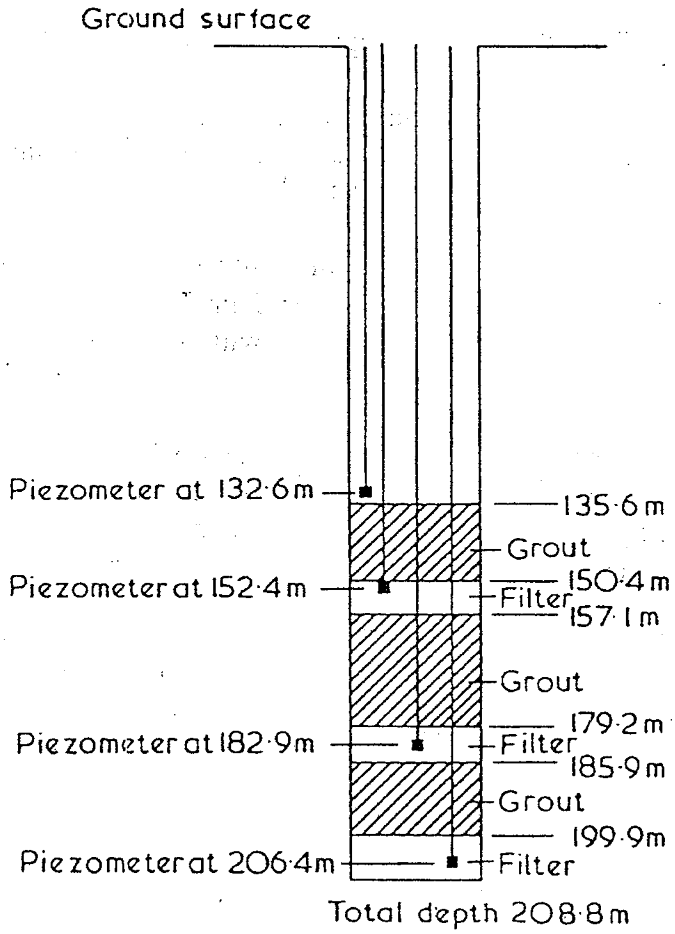

Ground surface
Piezometer at 132·6m
135·6m
Grout
Piezometer at 152·4m
150·4m
Filter
157·1m
Grout
179·2m
Piezometer at 182·9m
Filter
185·9m
Grout
199·9m
Piezometer at 206·4m
Filter
Total depth 208·8m

Fig. 13

The standpipe of the piezometer must always be kept full of de-aired water while lowering the tip into the borehole. When more standpipes are added, the joints between them should be leak proof and strong. Lack of strength at the joints may cause loss of piezometer tip into the borehole. Sand use for shrouding should be free from silt and fines. Its quantity and permeability should be checked.

9. Site Selection,Instrumentation and Monitoring of Piezometric Observation in Slope Instability Potential studies:

Case Study - 1

The piezometric measurements are very much important for evaluation and prediction of slope instability in the case of moving slope. The instrumentation program of the Watawala Earthslide Investigation Project and monitoring during the interim control stage was taken as typical case study.

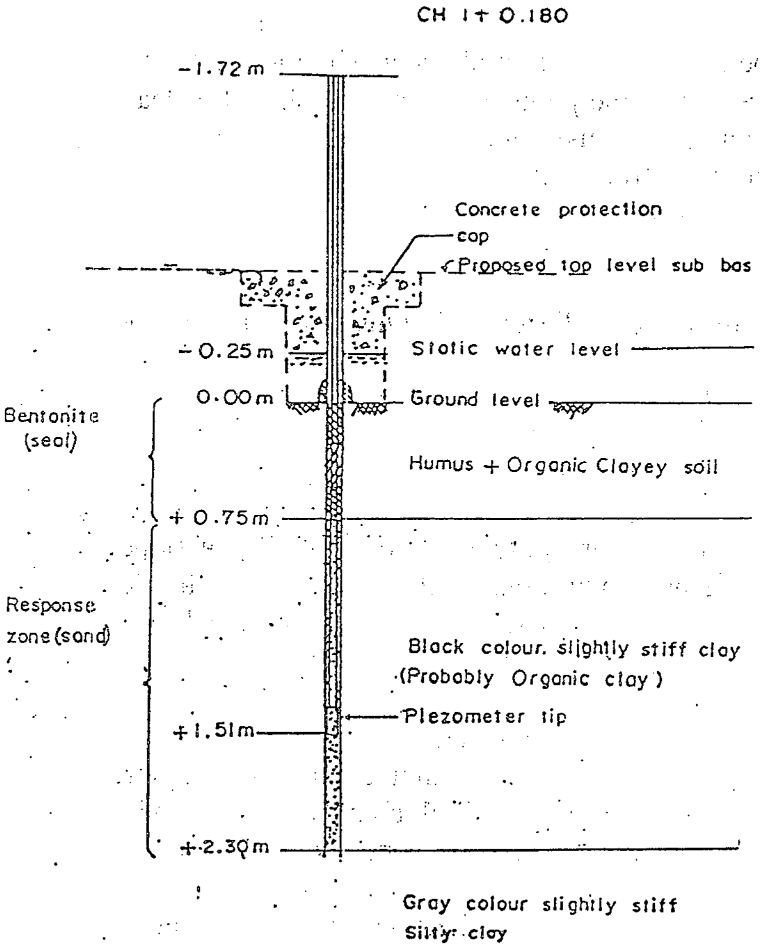
The slide boundaries of the Watawala Earthslide were established through a systematic programme of field mapping, site investigation, slope instrumentation, monitoring and mapping of discreate side boundary shears. The depth of failure or the basal boundary shears was established through the inclinometer observations which were installed at the axis of the slide.

There were nine Casagrande Open Stand Pipe piezometer and four Vibrating Wire piezometers installed during the instrumentation programme as shown in Fig.14. A typical patterns of piezometric measurements with the time of rainfall and slide surface movements is shown in Fig.15.

It is however pointed out that hydraulic piezometer rank among the most preferred because of their simplicity, low cost and rugged nature. Where the high response time hydraulic piezometer , is unacceptable. Therefore, quick responding Vibrating Wire piezometer were recommended. Watawala Earthslide, the Hydraulic piezometer worked very satisfactorily for a long length of time and yielded good results responded to rainfall and rise in piezometric pressure as shown in Fig.15. The Vibrating Wire piezometer was able to captured raid changes in pore water pressures during high intensity rainfall. The significant effect of piezometric pressure fluctuation was recorded during pumping test wells which were installed by the Water Resources Board.

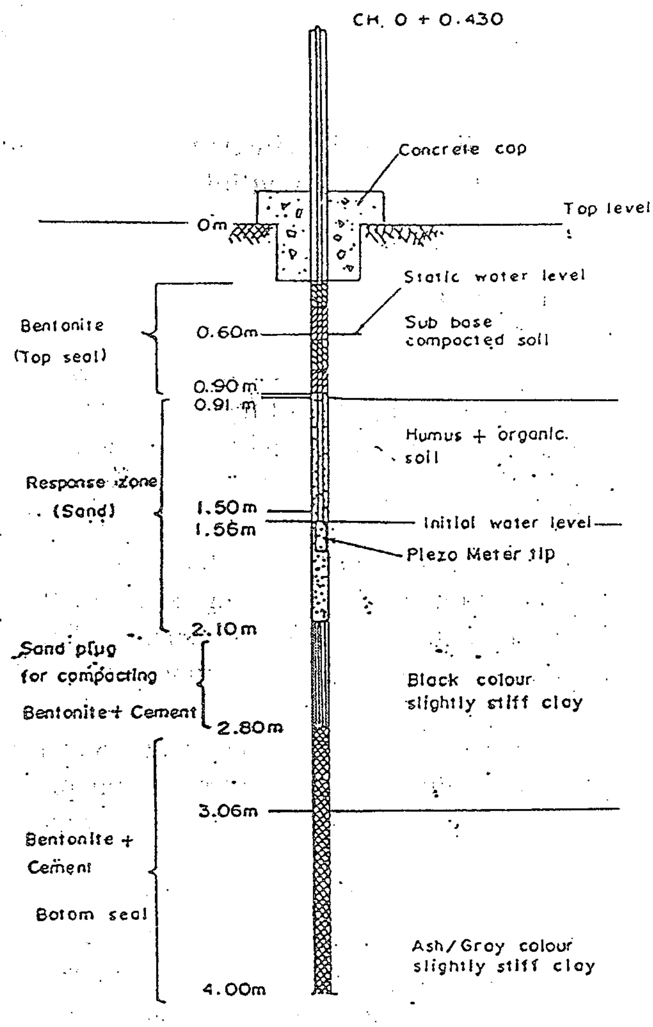
9.1 The effect of Piezometric Fluctuation in Stability of the Watawala Earthslide

Slope stability analysis of an earthslide is ideally suited in deterministic approach coupled with reliable contents of geometry of boundary shears, shear strength and pore water pressure developments on the sliding surfaces. In order to evaluate the overall stability of the Watawala

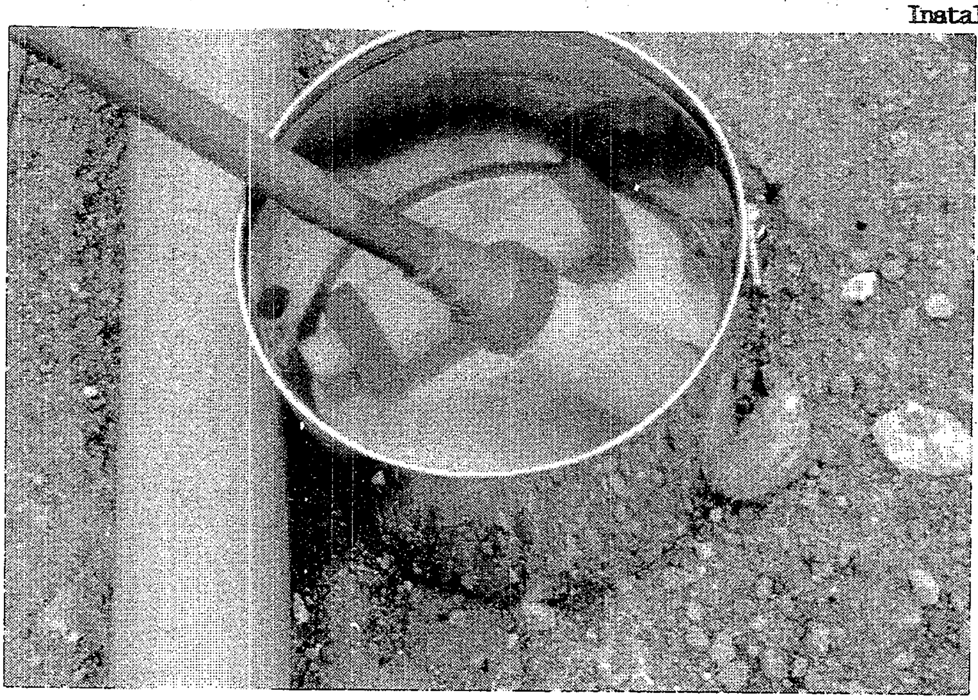


Instalation at marshy land

Installation at marshy area

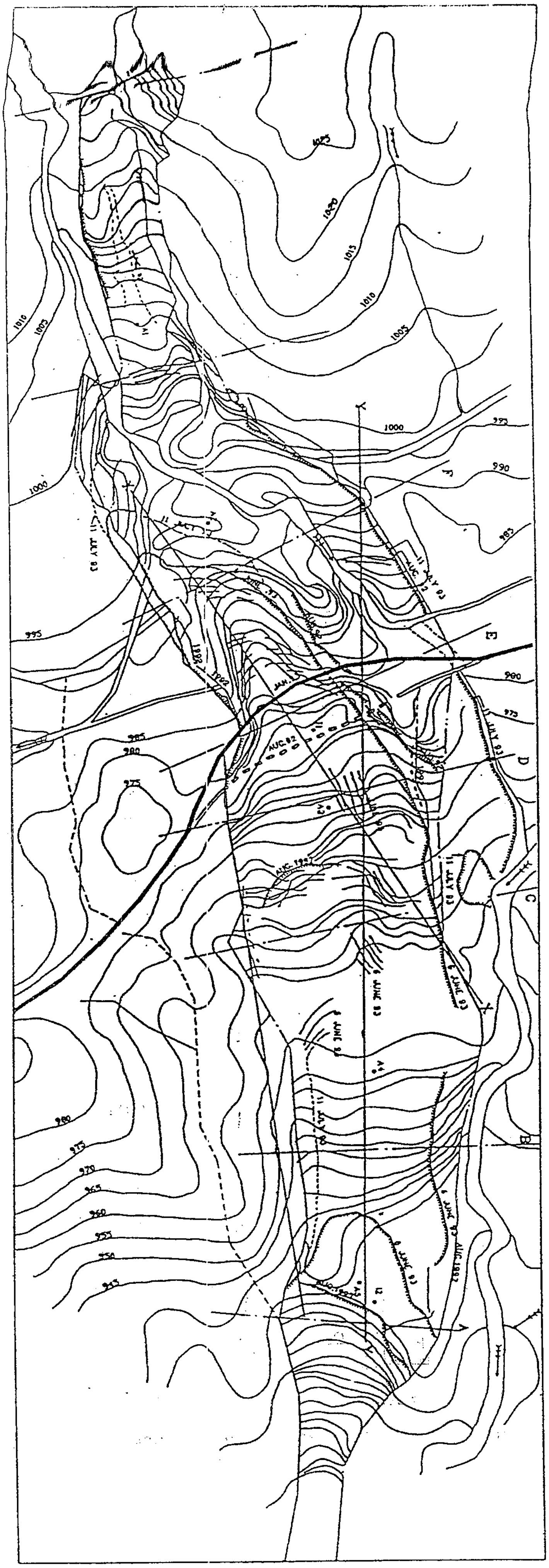


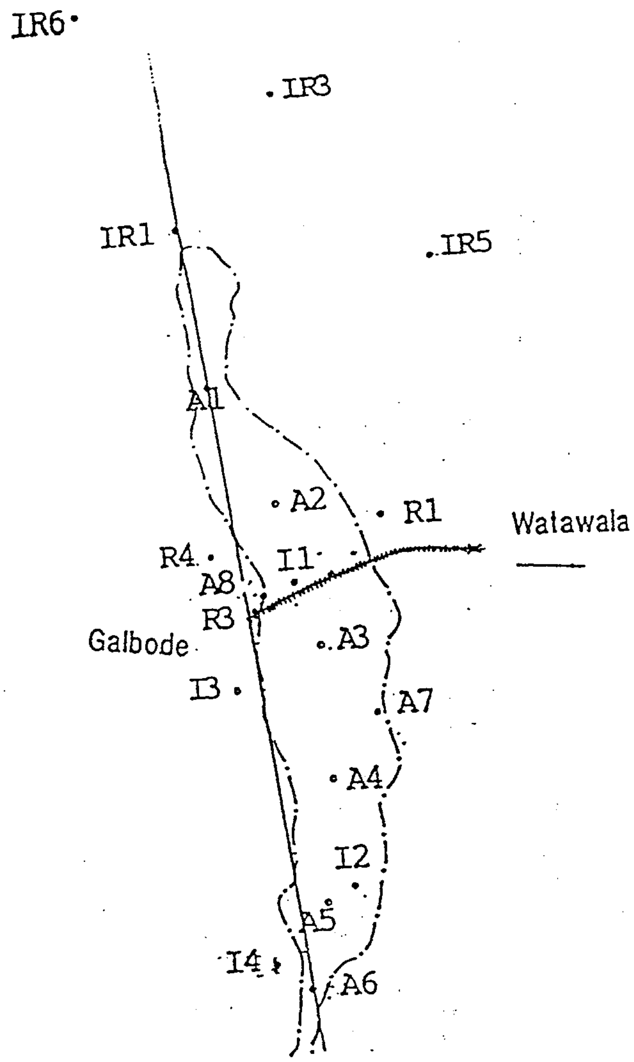
Installation at soft clay



De-aring of piezometer tip

Fig. 12





Deflection tubes	Inclinometer
R4	R3
I3	A3
	A2
	A6
Casagrande Piezometer	**Vibrating wire Piezometer**
R1	A4
IR1	A5
IR3	A7
IR5	A8
IR6	
A1	
I4	

Fig. 14 Field instrumentation at the Watawala Earthslide area

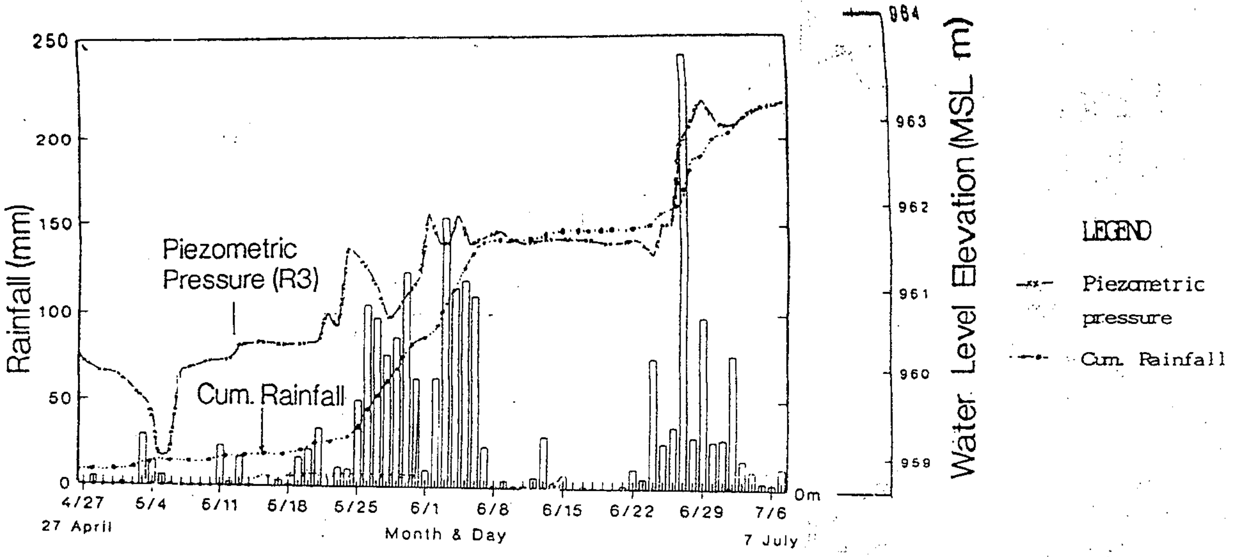


Fig. Correlation between Rainfall Intensity, Cumulative Rainfall & Piezometric Variation

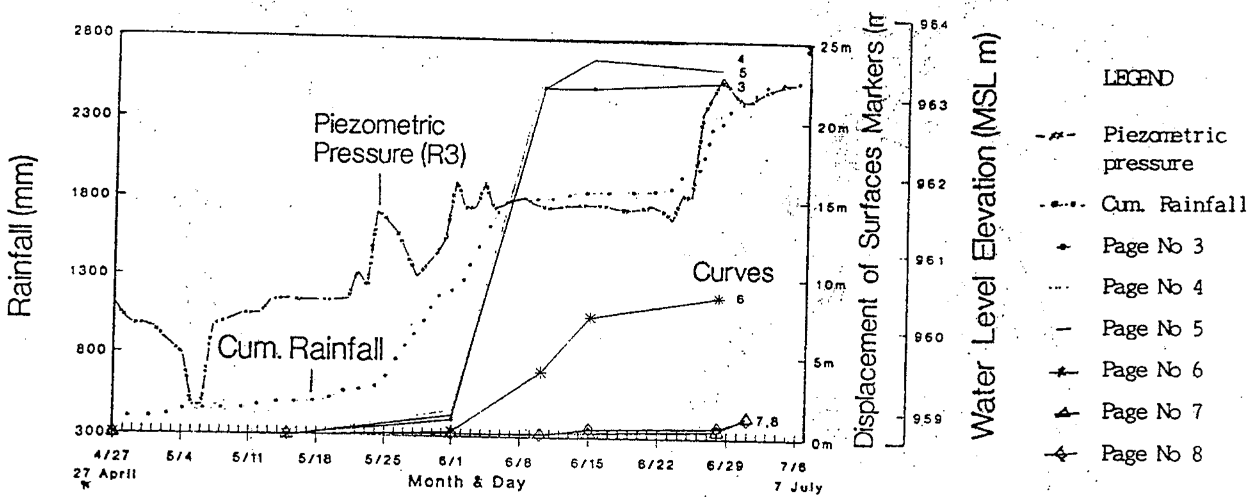


Fig. Correlation between Rainfall, Piezometric Pressure and Displacement of Slope Surface Movement Markers

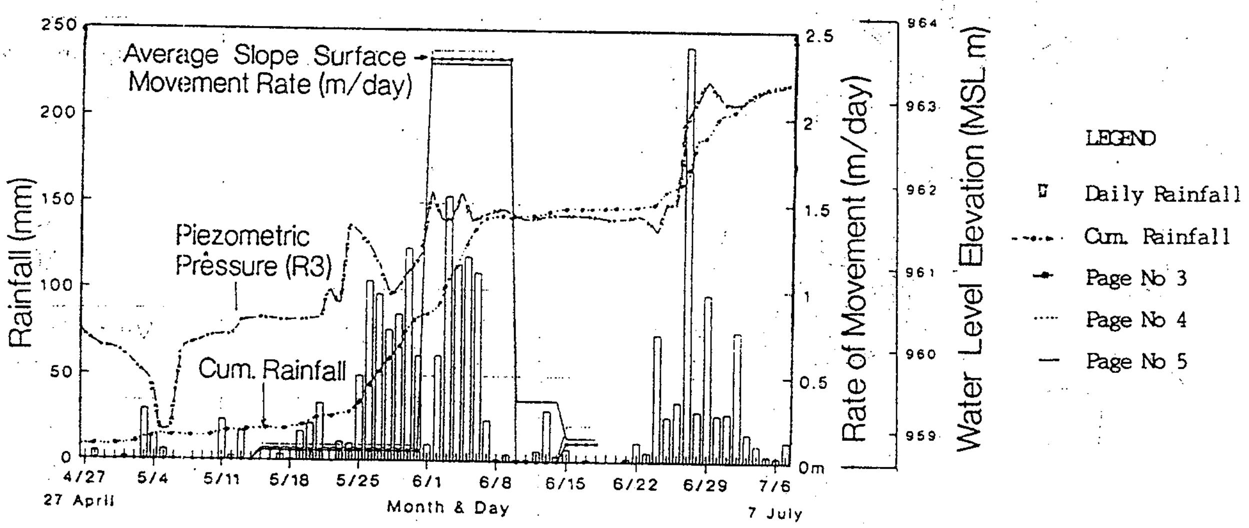


Fig. : Correlation between Rainfall, Piezometric Pressure and Slope Surface Movement Rate

Fig. 15

Earthslide at the limit equilibrium condition, 12 cases of actual non circular geometry of failures were studied along the two critical axis; Fig-14. It is significant to point out that, the calculated slip surfaces are at different depth of 11 to 24m and the back calculated angle of shearing resistance lies between 16° to 19° at the maximum piezometric profile. The influence of ground water table in stability at the Watawala Eartslide and improvement of the factor of safety by drawdown effect of ground water level is shown in Table 5.

Table 5: Stability Improvement due to Drawdown of Water Table
Friction angle = 15°, C = 0 kPa, Bulk Density = 21kPa

Depth of Water Table / Critical Slip Surface	3m	5m	8m	15m	18m	25
1	1	1.1	1.2	1.4	1.5	1.7
2	0.9	1.0	1.1	1.3	1.4	1.5
3	0.8	0.9	1.0	1.2	1.3	1.3

10. Sources of Error

As in other measuring instruments a number of sources errors are significant in the instrumentation of piezometers. In the standpipe and Casagrande type piezometers the principal sources of errors are summarised by Hvorslev (1951). see Appendix B. Brief notes of the list of errors as given below,

i. General instrumental errors; inaccurate measurements,faulty of construction or calibration, leakage through joints,poor electrical connections,and sensitive to temperature changes

ii. Seepage along conduits

iii. Stress adjustment time lag

iv. Hydrostatic time lag

v. interface of other liquids

vi. Gas bubbles in open systems, closed systems or in soil formations

vii. Sedimentation and clogging

viii. Erosion and development at the location of the piezometer tip.

11. Conclusion:

The proper instrumentation and accurate methods of monitoring of piezometric pressure is an important roll of the civil engineering construction activities. The success of these measurements are directly depends on the effectiveness of the actual response and the performance of pore water pressure development in the ground. Therefore sound experiences coupled with the knowledge behind the methodology of instrumentation, monitoring and maintenance of piezometric devices are definite advantage in the process of construction activities.

12. Acknowledgements :

The authors would like to thank the project team of the Landslide Hazard Mapping Project of National Building Research Organisation (NBRO) for their association and help extended at various stages. The analysis reported in this paper was pursued as a part of the Landslide Hazard Mapping Project, (SRL/89/001),Phase 1, implemented by the Government of Sri Lanka, executed by the UNCHS (Habitat), Nairobi, and funded by the UNDP, Colombo. It is being published with their permission. The views express in the paper are however those of the authors only.

Our grateful thank are due to Mr. C.H.de.Tissera, Director General, National Building Research Organisation for the permission, guidance and encouragements.

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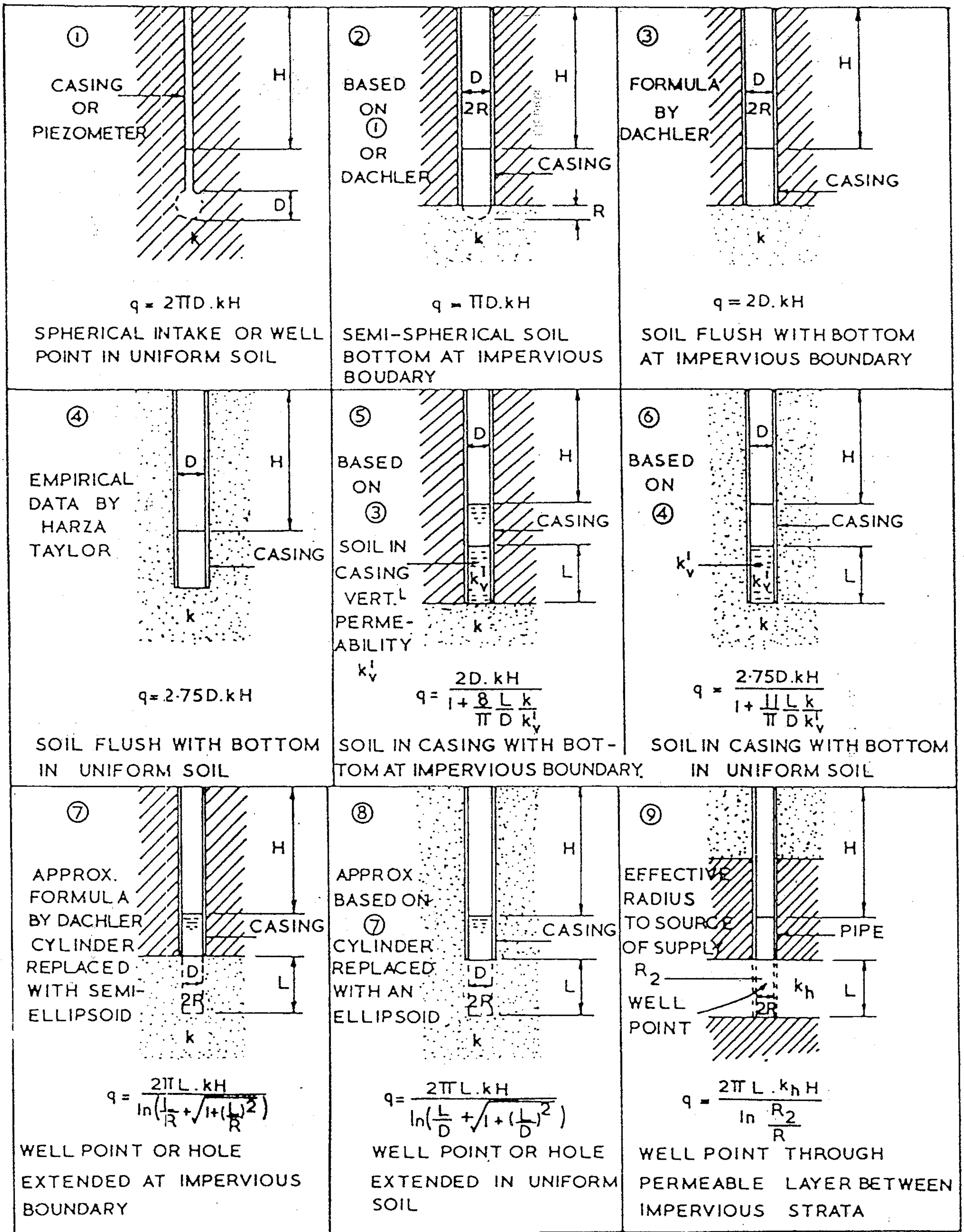
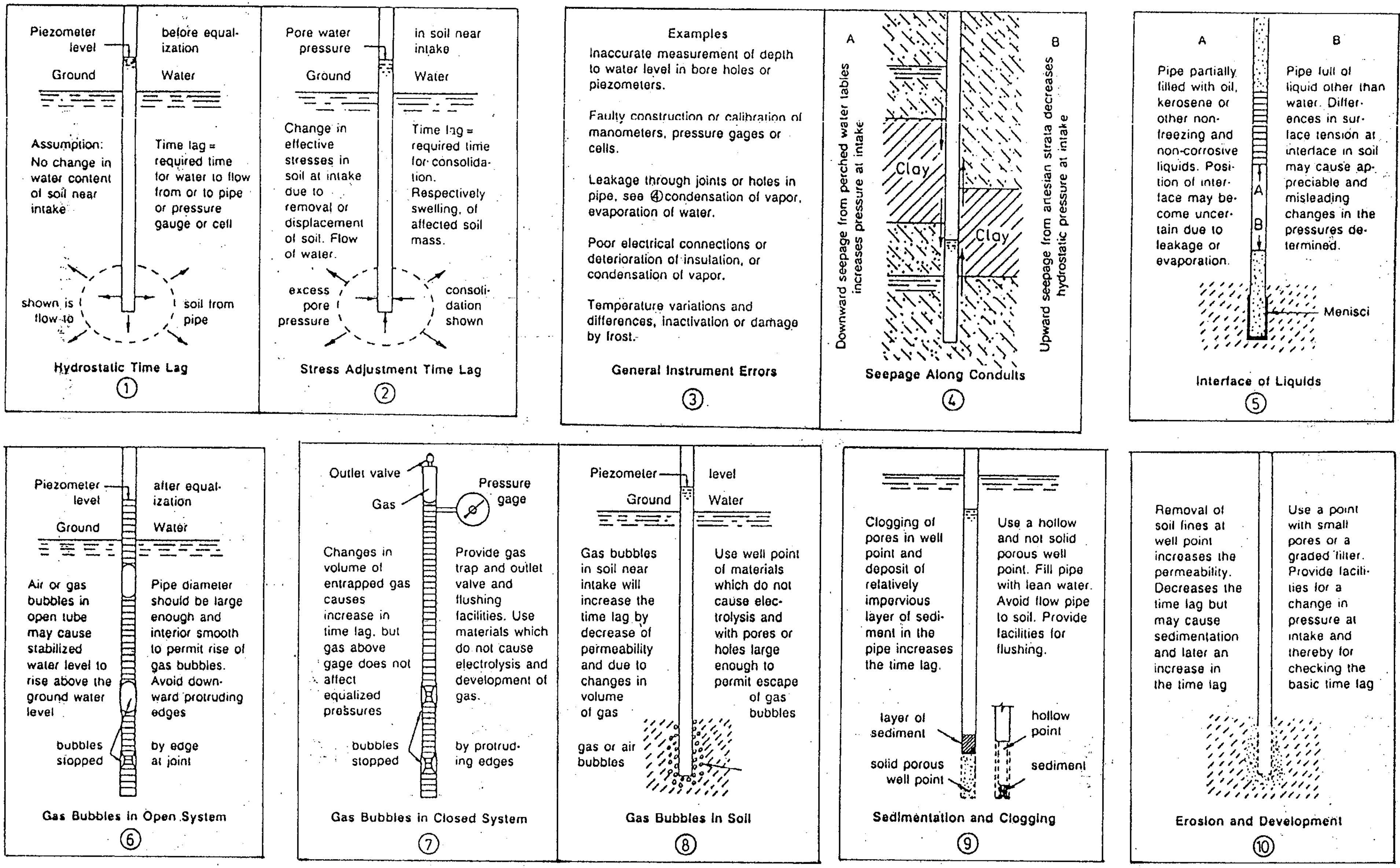


q = RATE OF FLOW IN CM^3/SEC, H = HEAD IN CM, k = COEF. OF PERMEABILITY IN CM/SEC, ln = LOG_e, DIMENSIONS IN CM.

Inflow and shape factors for piezometers (Hvorslev,1951)



Sources of error in groundwater pressure measurement (Hvorslev,1951)

Experimental Findings of Shear Strength Parameters of some problematic Soil deposits in Sri Lanka

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1. Introduction:

Soils are fundamentally composite of solid, water and air. These are naturally combine with concept of weight volume relationship which can be divided in to appropriate relations of fundamental soil mechanics of void ratio, porosity, water content, degree of saturation, bulk density and the physical means of specific gravity. The strength of soil is determine by various means of mechanisms involving in testing in compression, torsion and tension or extension. The minimum parameters are interesting in the assessment of shear strength determination, are cohesion intercept and the angle of internal friction. These appropriate parameters always depend on loading, confining pressures and rate of strain subject to the failures. However, non homogeneity of soil deposits bring the problem of inequality of two different test localities or more. Also similar situation occurs some problematic soils terrains such as in soft clays, peat soils, organic or marine deposits, leach lateritic soils and colluvium. The soil parameters of a soil containing in-situ discrete boundary shear is showing remarkable difference as in the case of landslides. The findings of residual strength of discrete boundary shears are also been considered in the paper. The unavoidable misleading results due to lack of observations on results and giving more attention on the preliminary assessments of soil is an important mistake made in interpreting problematic soils. Therefore, the preliminary stages of sampling of soil, extruding, sample preparation for testing and methodology available for the advanced interpretation of testing and analysis is considered as key references in the paper.

2. Some Problematic Soil Terrains in Sri Lanka

When soils are subjected to differential foundation loading cum stresses in a complex form, soil terrain is generally represented as difficult ground condition or problematic terrain as compared to other deposits. Therefore, relevant methodology of is required for testing samples because, ground improvement methodology is depended on the accuracy of investigation of soil. Loose sand deposits, organic clays, peat deposits , expansive soils, colluvium and discrete boundary shear containing soils are definite examples of such problem oriented soil in construction designs. The nature of representative distribution of these soil deposits are shown in Table 1 of the Appendix A..

Fig. 1

Visual appearence of soft clay showing slurry form during sampling.

Fig. 2

Stiff marine clay showing molding appearance and species of seashells.

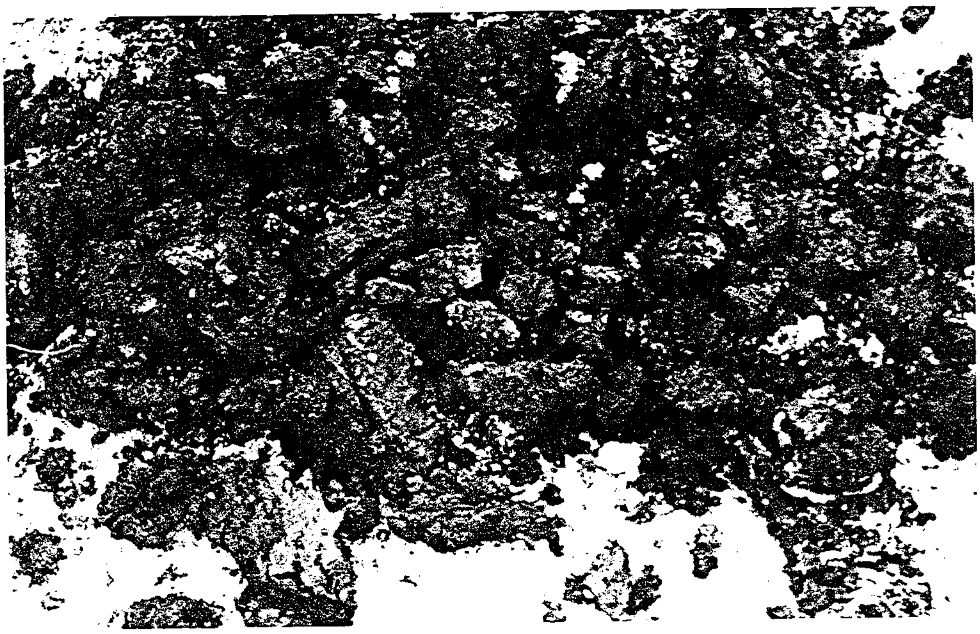


Fig. 3

Organic clay (peat) very loose appearence and some instances free water contain at the time of recovery.

3. Undisturbed Soil Sampling:

The undisturbed soil sample represented a minimum physical state of in-situ form with or without subjecting to environmental loading from the overburden or pore pressures or both. In a remolded sample the in-situ arrangement is always changed(i.e. the intergranular arrangements of soil particles). Some of the laboratory tests do not require UDS samples such as classifications, specific gravity measurements, soil indices, organic and inorganic parameters. Any way disturbed soil samples should contain actual composition of soil moisture, gradation and any other particles, as in the case of the collected site. However, any possibility of minimum disturbances to a soil sample, is strongly recommended. In order to obtain most relabel parameters of in-situ condition of testing at the laboratory, the high quality UDS soil samples are required. Nowadays, large number of mechanical and electrical devices are available for the sampling of undisturbed soil (UDS), through auguring or open pit excavation methods. Regular maintenance and use of best suited method of collection of the sample is equally important in the case of use of mechanical devices. If the quantity available for the test is lesser than the requirement of testings, then conventional approaches will not be employed and advanced method of testings are encourage to obtained better test results.

4. Visual Observations & Significant Variations of In-situ Moisture Content in Organic Soils

In problematic soil terrains visual identifications will help to understand soils in a great deal. Therefore, any significant visual observations should be recorded on each sample at the recovery from shelby tube or before any testings. Any intrusions of solid particles, such as sea shells, timber particles and any signs of in built shearing planes should be recorded. It is advisable to obtain photographs of the sample (color) in order to avoid any mis-recordings. Fruitful observations should contain, color, order, spots, particle size, particle orientations, sample disturbance, free water, and any other solid intrusions.

5. Moisture Content Measurements:

In-situ moisture content will influence to understand non-homogeneity of deposits. The condition of clay soil can be greatly influenced by changes of moisture content and softening of soil with significant presence of free water.

Based on the laboratory experiences of soil testings of high moisture containing soil, it is advisable to conduct additional testings such as organic content, in-organic content, clay mineralogy in order to understand the effect of variations of other soil parameters on shear strength, consolidation and soil permeability.

Fig. 4

Earth failure on residual formation.

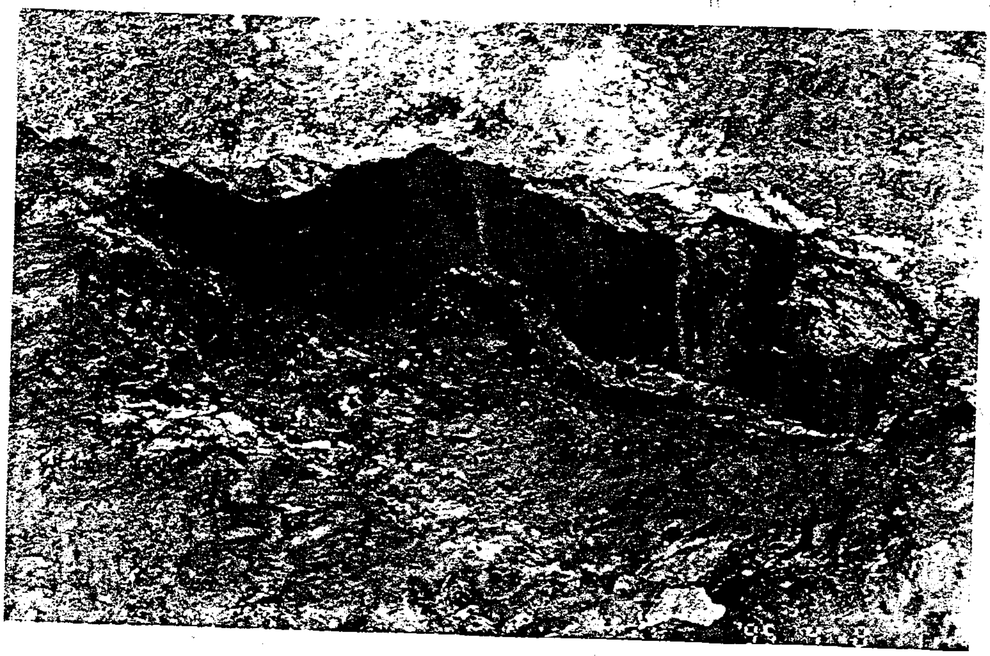


Fig. 5

Earth failure along descrete boundary of shear (Watawala Earthslide)

Fig. 6

Hetrogenity of soil always gives clear evidance of critical condition nor the reliable strength parameters can obtain.

Table 1: **Significant Observations & In-Situ Moisture Content in some Organic Clays and Marine Deposits**

Depth of Horizon	Visual Observations	in-situ moisture content variations within a 1m long single shealby tube %
0.4 - 1.5	black color, soften soil, free water contain at the time of recovery - probably a organic clay	175 190 184 362
0.7 - 1.5	soil contain highly decomposed timber portions,black colored, soft material containing organic form of species, drying well - peat or dumped earth	240 420 498 520
1.5 - 2.4	highly saturated, gray in color,very soft form of clay, slurry form appearance - soft clay	126 120 108 140 142
3.0 - 3.9 5.0 - 7.2	dark gray clayey soil with intrusions of sea shell - marian clays	123 156 166

Some observations of significant variations has been made in the boundary shear plane samples at the interface between boundary of residual soil and the colluvium.

The moisture content test as per the ASTMD 2216-90 code specify that drying oven temperature of 110°C is too hot for certain organic soils and soil containing appreciable amount of gypsum or some other minerals. This is because when soil contains gypsum the molecular bond water crystallization may tends to loose at temperatures above 100°C. Therefore, ASTM suggested that oven temperature of 60°C is suitable for drying of these types of soil to avoid any mis-interpretation.

Table 2: Visual Observations of Discrete Boundary Shear Plane associated mineral clay layers: thickness 1cm - 1.2m

Approximate Depth at recovery m	Visual Observations	Moisture Content variation in 300* 300 box sample
1 to 3.5	very stiff light green color clay, white patchers, at the upper part of the plane of shear. very thin shinning luster due to muscovite mineral - slickenslide nature	38 to 44
1 to 3.5	very thin stiff brown color, clay layer with hard stratum. soil type within the range of clayey silty sand. very thin shine luster due to muscovite mineral - slickenslide nature	26 - 36

Table 3: Moisture Content variations in different soil types of the country

Soil Deposit	Moisture Content %	Location
Residual Soils	25 - 35	hill country
Loose sand	25 - 32	Colombo; uniform sand or mixed sand
Peat and Organic Soil	140 - 550	Muthurajawela and Southeren marshy areas
Marine Deposits (marine clays)	100 - 145	Matara, Southern marshy land areas
Soft Clays	120 - 160	Southern marshy land areas

Expansive Clays	8 - 30 10 - 33 10 - 15 4 - 27 8 - 12	Annuradapura Mahiyangana Dabulla Buttala Kataragama

6. Shear Strength Characteristics

Conceptually, triaxial compression tests are specified in the BS1377, for the determination of effective and total shear strength parameters of specimens with saturated soils which has been subjective to isotropic consolidation in the case of effective strength measurements and then sheared in compression , under constant confining pressure while maintain a constant rate of deformation.

The difference in the observed shear strength parameters during various testing arrangements, and what can be expected from the soil classification indices are somewhat important in the laboratory experiments. Example, the some literature reviles that characteristic variation of residual soil comparatively less than that of problematic soils deposits in the country. This is one of the example of limitation of the use of classification system in residual soils without considering other factors such as degree of weathering etc. The Table-4 shows that differences of observed shear strength parameters in residual soil.

Table 4: The Effective Peak Shear Strength Parameters of Residual Soils (Senevirathne;1993)

British Soil Classification	Gravel by Weight %	Cohesion Intercept(kPa)	Angle of Internal friction (°)
SFI - SFH	13	5	38.6
MVS	5	11	32.7
MIS	2	33	30.7
MIS	1	20	35.0
MVS	3	21	36.2
SMI	3	0	39.6
MHS	1	8	36.4
MHS	8	24	36.2

Some of the recent findings of the soil variations of classification parameters of residual soils in Horana, Sooriyakanda, Ratnapura, Kabaragala, and Kandy are recorded in the Appendix B

The undrained and drained shear strengths of parameters are significantly vary specially in organic clays, marine clay deposits and in some cases loss of strength is significant than the expected. In order to obtain fruitful observations same tested samples were subjected to find an organic contents and the Atterburg limits. The experience showed that liquid limit is remarkably lower than the in-situ moisture content. This type of data interpretations clearly indicate that the organic content showing a reflection of the incapability of any guess work of the interpretation of the average strength of organic and marian clay deposits. Some variations of undrained cohesion and effective shear strength parameters are shown in the Table 5 and Table 6.

Table 5: Some Results of average Undrained Strength of Organic Clays and Marine Deposits in Southern Marshy Areas of the Country

Soil Deposit	Depth of Horizon	Average Undrained Strength of soil C_u
organic clay organic content: 12% to 24%	.5m to 2.5m	2.9 - 10.3
marine clay organic content: 10% to 15%	.4 to 1.5m	8.5 - 22.8
very soft clay	.9 to 1.6	0.0 - 4.6

Table 6: Some Test Results of Effective Shear Strength Parameters of Organic Clays and Marine Deposits in Southern Marshy Areas of the Country

Soil Deposit	Depth of Horizon	Cohesion Intercept(kPa)	Angle of Internal friction (°)
organic clay organic content: 12% to 24%	.5m to 2.5m	0 - 16	9°to 21°
marine clay organic content: 10% to 15%	.4 to 1.5m	2.9 - 18.0	9°to 29°
very soft clay	.9 to 1.6	1 - 1.2	----

7. Discrete Boundary Shear Strength of an Earthslide

Slope stability analysis of an earthslide is ideally suited in deterministic approaches coupled with reliable contents of geometry of boundary shears, shear strength variation and pore water pressure on the sliding surface. The Watawala Earthslide is conclusively known to be repeated failure itself, year after year from 1920s and well developed boundary shear will provide a field example which stimulated the importance of shear strength on a high degree particle oriented discrete boundary shear surface. The possibility of determination of loss of strength through progressive failures or ample evidence of residual shear strength on a slickenslide shear surface has been appreciated during the investigation programme. But quantification of its effect has been hampered by limitation of undisturbed soil samples on a discrete soil boundary. Such an effect might be reduced by comparing the results of Ringe Shear test or Field Direct Shear test results. The extensive study reported that the shear strength parameters found by laboratory determinations; $c' = 4kPa$; $\phi_r' = 14° \pm 1°$ and those found by back analysis $c' = 0$; $\phi_r' = 14° \pm 1°$; or the values of Ring Shear test results which represent the lower bound residual strength of $c' = 0$; $\phi_r' = 13° \pm 1°$. Many of insitu field Direct Shear test represents $c' = 0$; $\phi_r' = 16° \pm 1°$ which indicate that the level of acceptable risk for the assessment of an overall stability of the earthslide was not sufficient with limited tests.

8. Use of the Geotechnical Digital System for Triaxial testing

A newly established Geotechnical Digital System for triaxial testing can be adopted for precise controlling of pressures ,data acquisition and data interpretation. Test is usually carried out using conventional method of testing with a sample diameter of 36mm to 50mm, subjected to a different confining pressers up to 2000 kPa and the method of testing which is precisely controlled by digital and computer interface.

Further, one 50 mm diameter UDS specimen tested according to the multistage quick undrained triaxial test with constant strain test mode of GDS triaxial apparatus. Also another sample of the same soil was subjected to a slow rate of strain of 1% / Hr and comparison of results are given in Table-7.

Data interpretation was based on number of graphical plots such as Axial strain vs Deviator Stress, Axial strain vs Porewater pressure , and p' vs q' plots (where $p' = (2\sigma_r + \sigma_1)/3$ and $q' = (\sigma_1 - \sigma_r)$).

Detail interpretations of results were annexed at the Appendix - C of the paper and the apparatus as shown in the Fig .

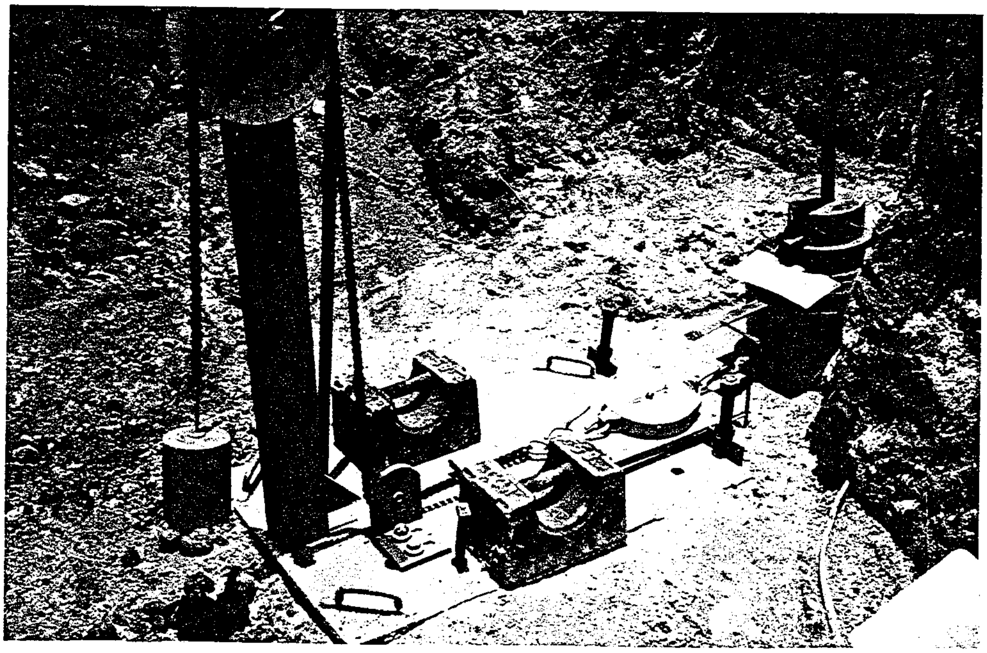


Fig. 7

In-situ Field Direct Shear Apparatus setup on Discrete Boundary Shear Plane at the Watawala Earthslide

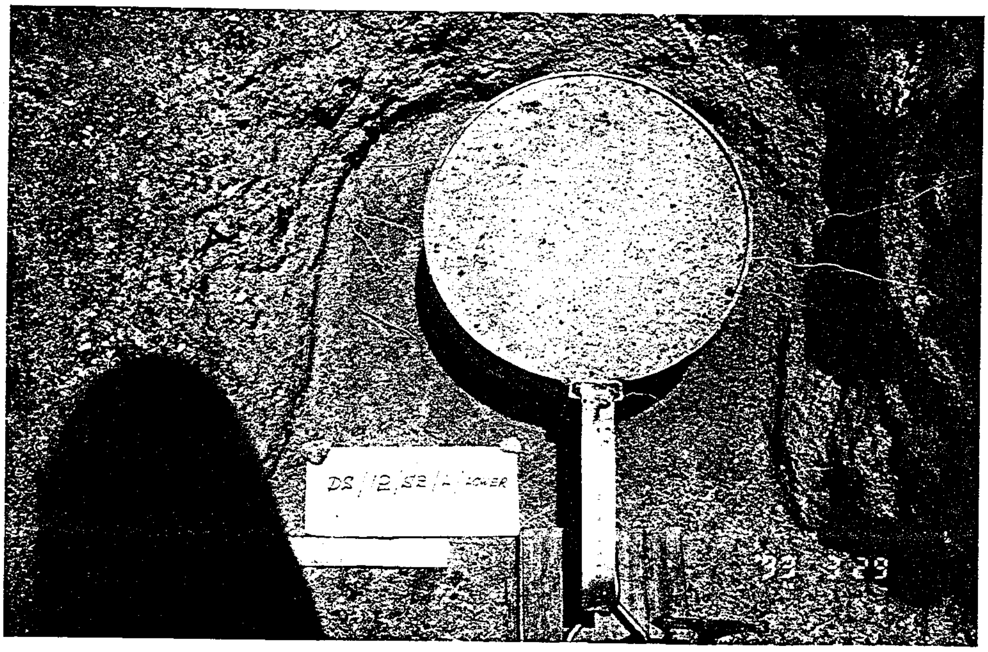
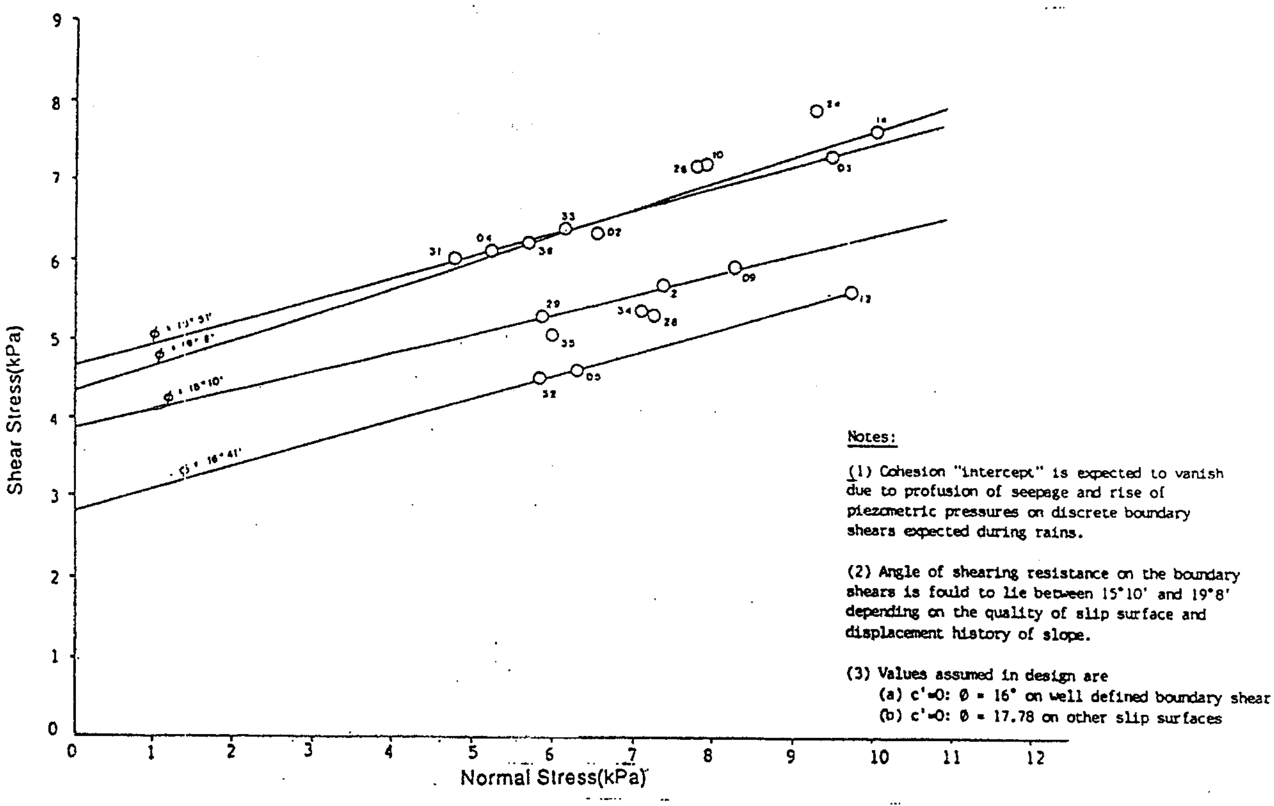


Fig. 8

In-situ preparation of 300 mm diameter sample on the plane at shear (before applied load)

Fig. 9

Just after shearing shining tuster indicates the slikenslide nature of boundary shears.



WATAWALA EARTHSLIDE - Insitu Direct Shear Tests on Slip surfaces

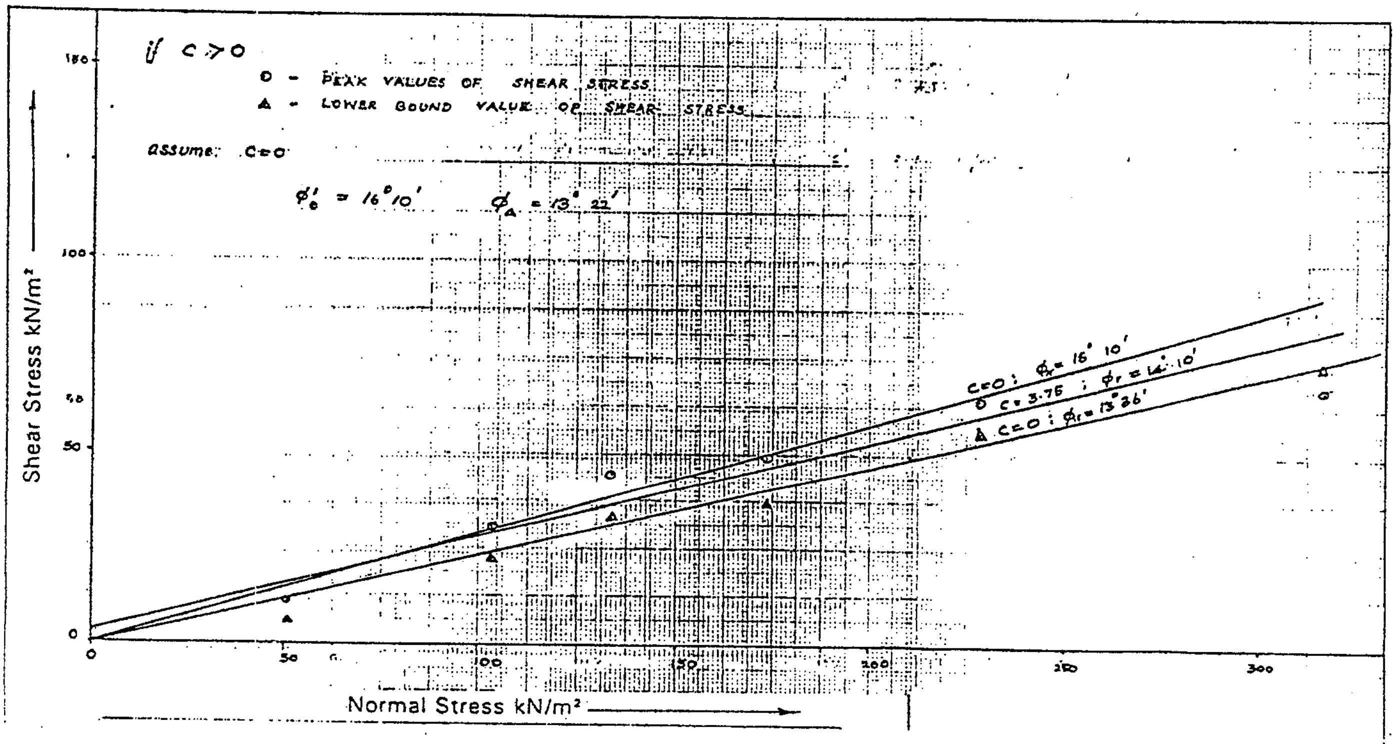


Fig. - Laboratory Direct Shear Test results on Discrete boundary shear; UDS samples

(WATAWALA EARTHSLIDE)

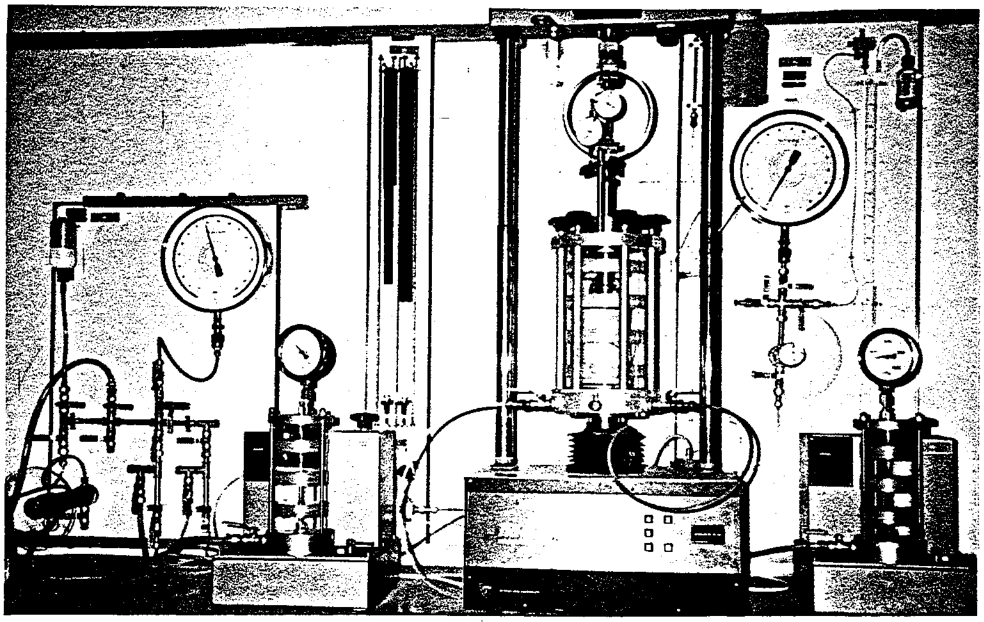


Fig. 13

Conventional Triaxial Apparatus setup

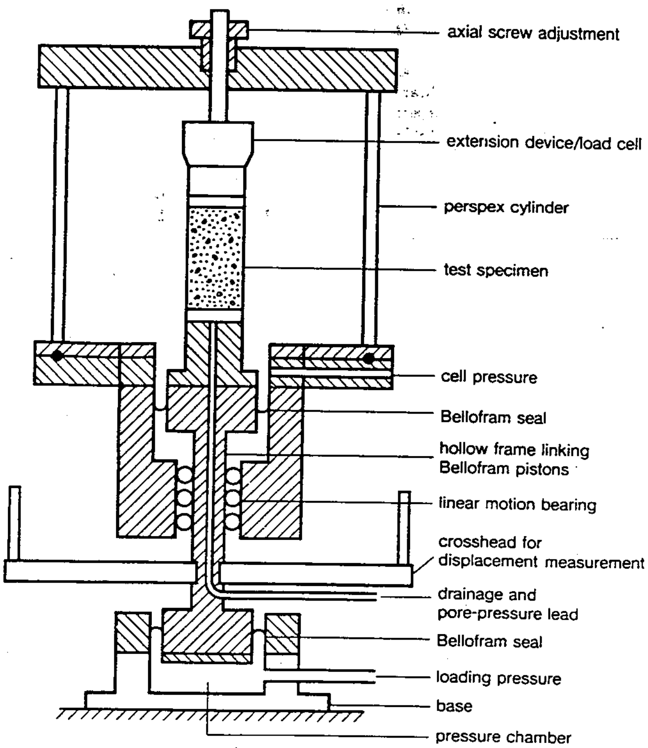
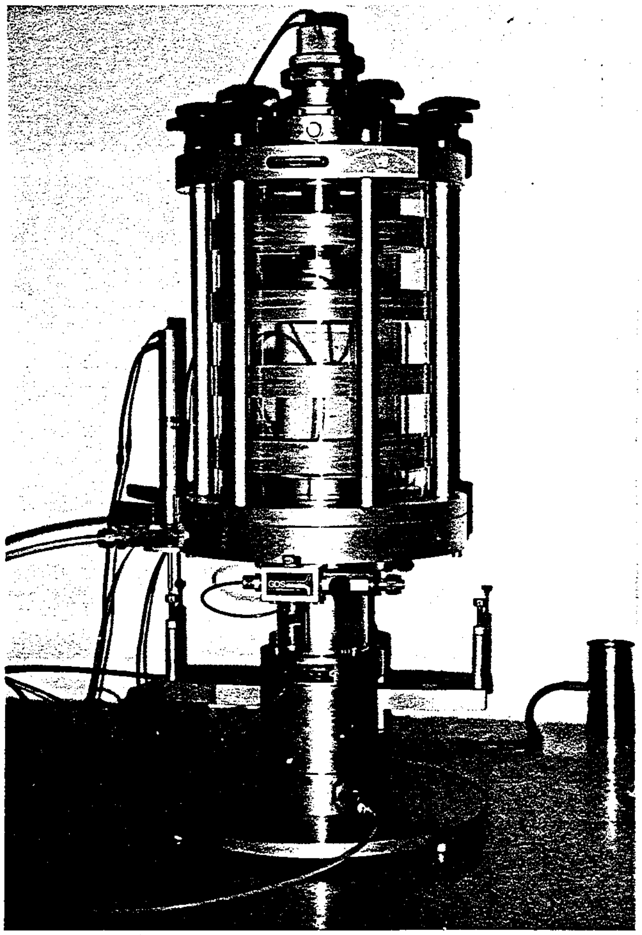


Fig 14. Diagrammatic representation of the Bishop & Wesley Stress path cell.



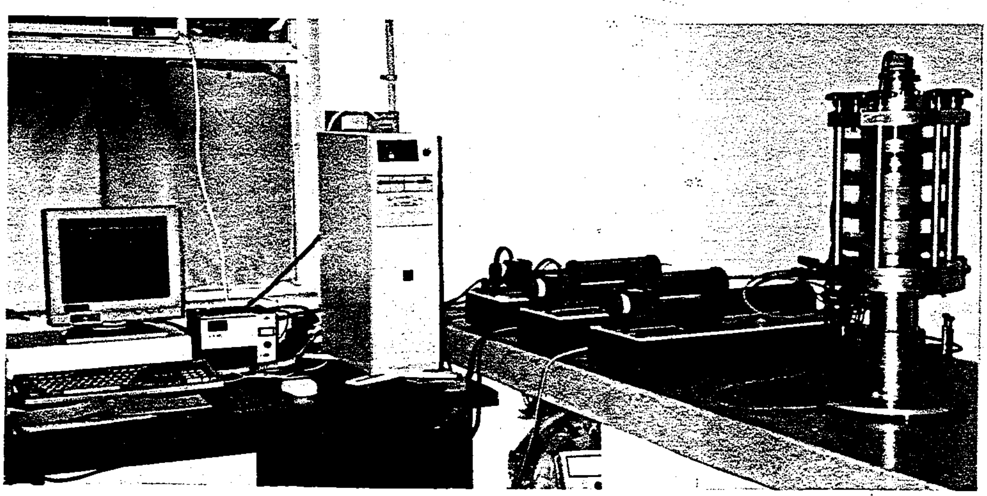


Fig. 15

Geotechnical Digital System (UDS) Triaxial Setup at the Geotechnical Engineering Laboratory, NBRO.

Table 7: Results of Unconsolidated Undrained Triaxial Test which was subjected to a different compression according to BS 1377:part7;1990.

Unconsolidated Undrained Triaxial test - GDS	Undrained Shear Strength kPa
Constant rate of strain of quick test and single interpretation; B=100 %	9.12
Multistage quick undrained triaxial test; B=100 %	9.25
Multystage slow strain undrained test; 1% per hour strain loading test; B=100 %	5.36

9. Conclusion:

This reported experience is shown considerable value as if it increases the level of confidence to a considerable stage as the results are overall determination of shear strength using undisturbed soil samples.

10. Acknowledgements :

The authors would like to thank the project team of the Landslide Studies and Services Division, National Building Research Organisation (NBRO) for their association and help extended at various stages. The analysis reported in this paper was pursued as a part of the Landslide Hazard Mapping Project, (SRL/89/001),Phase 1, implemented by the Government of Sri Lanka, executed by the UNCHS (Habitat), Nairobi, and funded by the UNDP, Colombo. It is being published with their permission. The views express in the paper are however those of the authors only.

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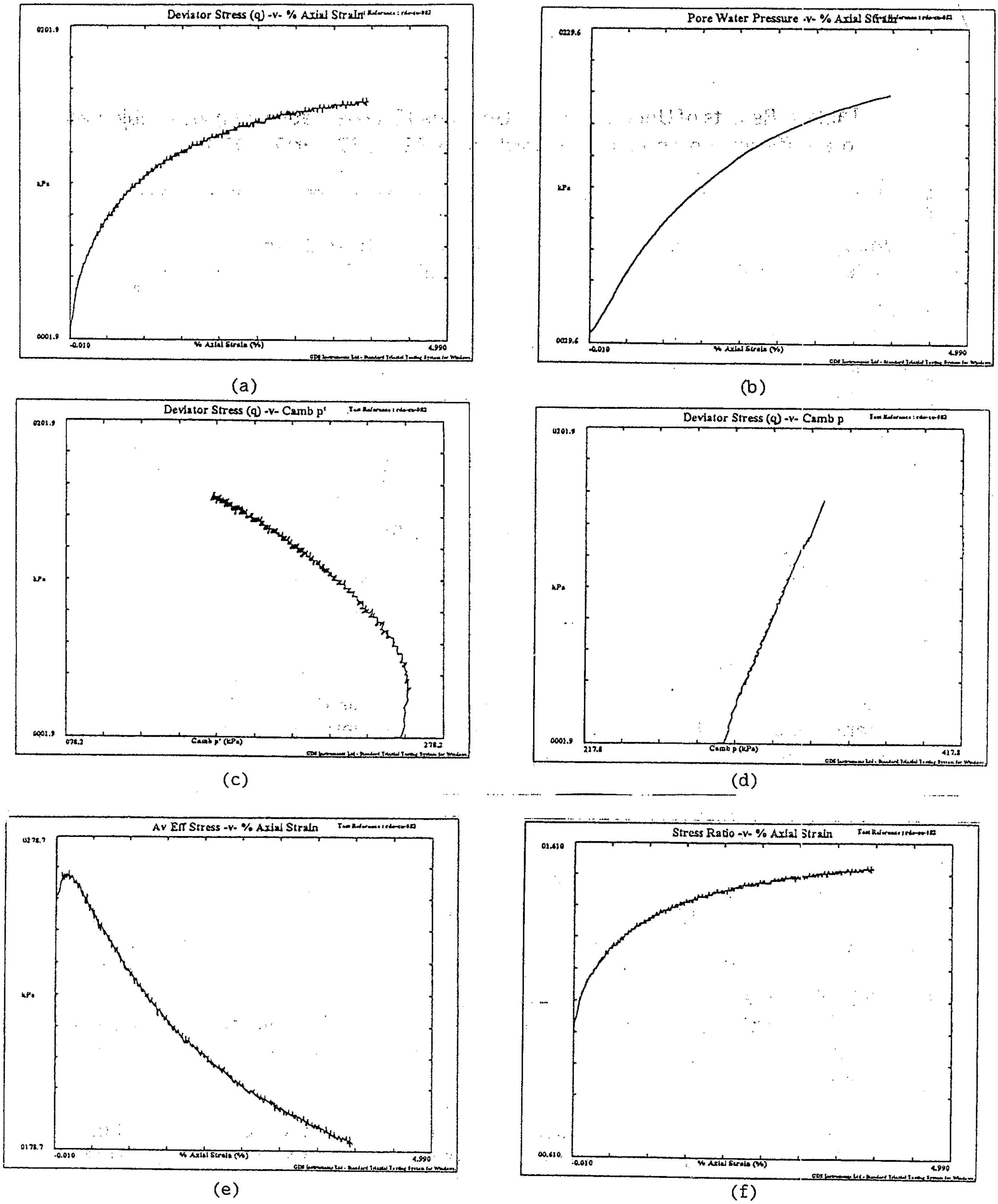

Geotechnical Digital System outputs for Consolidate Undrained Test;

(a) Deviator Stress Vs Axial Strain
(b) Pore Water Pressure VsAxial Strain
(c) Deviator Stress Vs Camb p'
(d) Deviator Stress Vs Camb p
(e) Av Eff Stress Vs Axial Strain
(f) Stress Ratio Vs Axial Strain

13. Reference

Manual on Ground Improvement Guidelines with reference to Colluvium, loose sand deposits, Expansive soils and peat(1995) : Dr.B.Indrarathne, K.Rajarathnam, S.Karunagoda, D.A.Samaranayake & S.Jayasekara; Tokten Programme; Internal Publication; Landslide Studies & Services Division, National Building Research Organisation

Senevirathne, H.N.(1993); Engineering Behavior of Tropical Residual Soils; Proceeding of the seminar on Geotechnical Practices in Difficult Ground Condition, pp21

APPENDIX A

Table 1: Some Ground Condition Associated Soil Deposits in Sri lanka.

Soil Type	Location	Remarks
loose sand deposits Type - SW/SP/SM/SM-ML SPT (N Value) - 0-10 Porosity 0.46 - 0.40 Void ratio 0.85 - 0.67 Water content % 32% - 25% Unit Wt dry g/cm3 1.43 - 1.59 Unit Wt Sat g/cm3 1.89 - 1.99 Skin friction 10km/m^2	Kollupitiya Bambalapitiya Borella	
Expansive Clays Type - CL/CI/CH/SC SPT (N Value) - 22- >50 Moisture content 10% - 33% Liquid limit 30% - 66% Plastic limit - 18% - 33% Plasticity Index - 6% - 50% Silt & clay% 20% - 86% Swell pressure(kg/cm) 0.1- 3.2	Mahiyangana	These soils contain essentially platy minerals like Montmorilonite and Vermucalite which have an affinity for water
Peat and Organic Soils Moisture content 100% - 600% Liquid limit 60% - 20% Plastic limit - 30% - 70% Specific gravity - 0.7 - 2.3 Insitu void ratio - 1.0 - 4.0 Coefficient of consolidation (cv) 10^{-3} - 10^{-4} cm/sec. Coefficient of volume compossibility (M_v) 0.05 - 0.7 SPT n volume - 0-2	Peliyagoda	Peaty soils found in low lying marshy areas and flood plans Differential and excessive settlement is the principal problem confronting the geotechnical engineers dealing with the peaty soils

Leached Laterites Moisture content 16% to 49% Liquid limit 33% to 90% Plasticity index 5% to 59% Clay fraction 15% to 45%		color of lateritic soils varies from very dark red,reddish brown and yellow or pink. Laterites are of low to medium plasticity. The leached laterite having loss of shear strength due to chemically disintergration
Colluvium Deposits Moisture content 28% - 44% Liquid limit 30% - 70% Plastic limit - 24% - 44% Specific gravity - 2.5 - 2.8 Peak shear strength - 22° - °29 Cohesion (kPa) - 7 - 14	Watawala	colluvum deposits are associated with the occurrence of landslides in the central hill county districts of Sri Lanka
Shear zones Moisture content 28% - 44% Liquid limit 55% - 70% Specific gravity - 2.5 - 2.8 Peak shear strength - 14° - °19 Cohesion (kPa) - 3 to 7	Watawala	prominent in old slow to medium earth slide