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SOME SPECIAL FEATURES IN THE GEOTECHNICAL ASSESSMENT OF A SITE FOR THE CONSTRUCTION OF BUILDINGS IN MARSHY AREAS

by

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Abstract

The presence of problematic soils in marshy areas poses a challenge to the engineers who build on them. Taking a case study of such a site where extensive studies were carried out, special features of such sites which affect the design of foundations are discussed. Vane shear tests have been done and their usefulness in investigating marshy areas is evaluated. It is shown that special laboratory testing procedures have to be used for studying the consolidation characteristics of the peaty soils which are present. Further, because the available theoretical models for predicting field settlements in peat are either inappropriate or too complex, simplified procedures have been evolved based on settlement studies of buildings already constructed on the adjacent site. The coefficient of subgrade reaction is shown to be an additional parameter which should be established for the design of shallow foundations in marshy areas. For this purpose, the plate bearing test is not suitable because these sites contain layers of different soil strata. An alternative method based on the determination of the average modulus of elasticity of the soil is proposed.

1.0 Introduction

As a result of major development projects in the industrial, commercial, and housing sectors during the past decade, much of the 'good' ground in and around the city of Colombo has been utilized and it has now become necessary to consider the planned development of the low lying areas. A study of the Greater Colombo area has shown that already some 150 hectares of marshland have been reclaimed; and after incorporating a stormwater drainage scheme which uses the marshland as large retention areas that absorb storm flows and reduce the peak flows to be discharged downstream, a further 120 hectares of marshland could be released for development [NORTH (1989)].

The marshy areas often contain peat and other organic clays which show high compressibility and low shear strength. Therefore, the design and construction of structures on these soils pose a challenge to the civil engineers. A good site investigation is absolutely essential for this purpose. This paper deals with some special features that were used for the geotechnical assessment of such a site.

2.0 Project Description and Project Site

The project is the Open University Building Complex - Stage IIA; and the site is adjacent to the Kirulapone Canal at Nawala, and it has been reclaimed from its low lying state by filling. Some of the buildings which are to be constructed in Stage IIA of the Building Complex are shown in Fig. 1. The three Administration Buildings and the Directorate Building shown in the figure have been designed as 3-storeyed structures, whilst the Library Building is a 2-storeyed structure and the S.C.R. Building is a single storeyed structure. (The other buildings to be constructed in Stage IIA are outside the location area of Fig. 1).

Stage I of the Building Complex has already been completed on the other side of the canal. In Stage I there are a few 2-storeyed buildings, but the majority of the buildings are single storeyed structures.

3.0 Field Investigations

The field investigations were planned to obtain the information required for the design of foundations as

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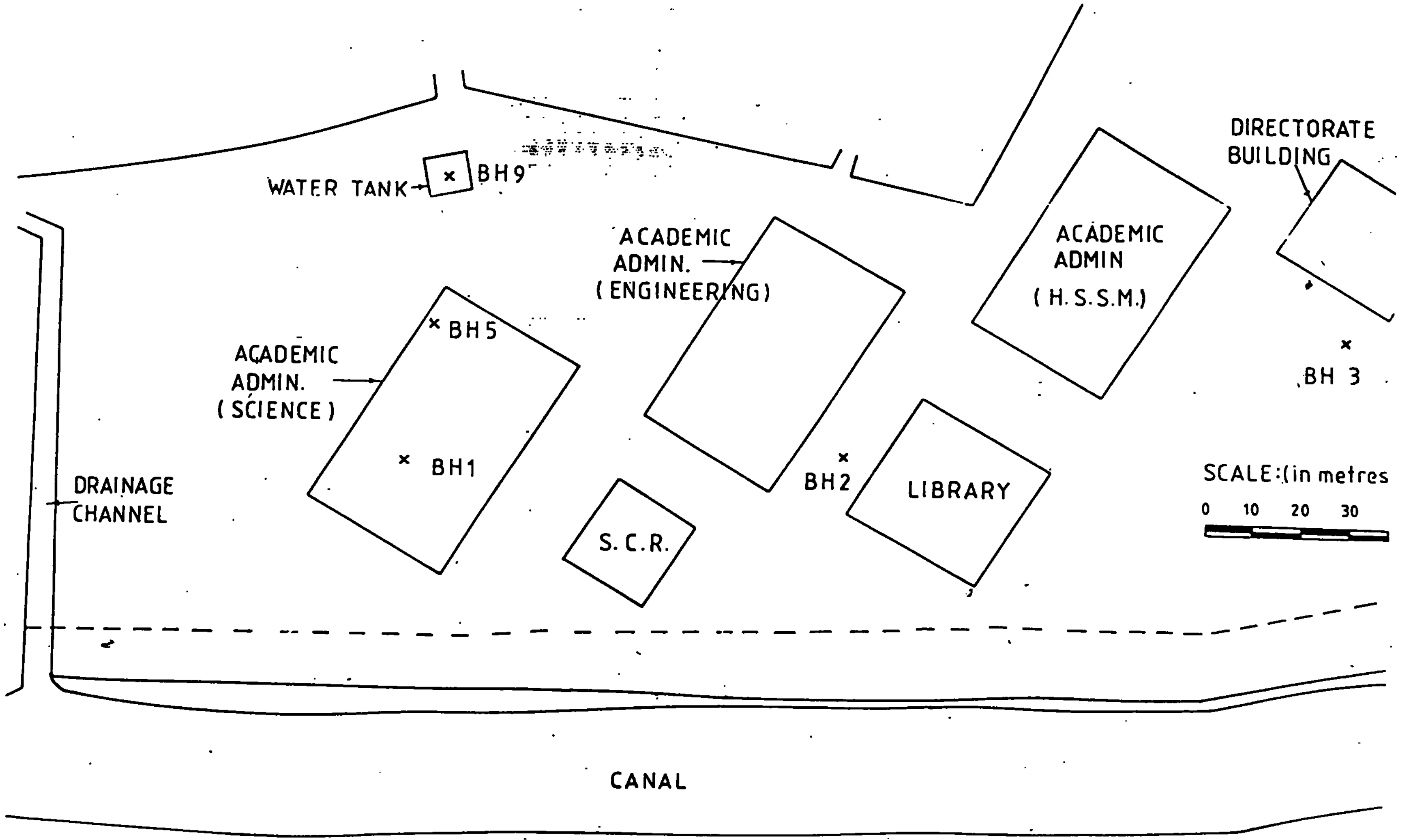

Fig. 1 — Layout of Boreholes

economically as possible. Nine boreholes were advanced at the site, four of them upto rock and the rest to a depth of 10.0 m. The location of 5 of the boreholes are shown in Fig. 1, while the other 4 boreholes are outside this area but within the project site. In terms of cost, the total cost of the investigation was about 1.8% of the estimated cost of the foundation.

In the planning of the boreholes, consideration was given to the fact that

i. the site covered a large area in which a number of buildings were to be constructed; and

ii. the site would contain some peat layers. (A study of some of the other site investigation reports in the area showed the presence of two peat layers).

Therefore, one set of boreholes was located approximately parallel to the canal, while the second set of boreholes was approximately perpendicular to the canal.

3.1 Sub-soil conditions

The cross-sections across the boreholes are shown in Figs. 2a and 2b. From these results, the sub-surface in the marsh can be categorized as the M-II sub-series defined by SENANAYAKE (1986). In this sub-series, the natural soil has four distinct horizons: peaty soil followed by a transported sand/silt followed by a second peat layer below which is sand or some other hard stratum. As observed by Senanayake, the peat in Horizon 1 was easily identifiable with its well preserved vegetable remains, but the vegetable matter in Horizon 3 had undergone decomposition to a much higher degree so that its biological identity was more or less lost. In the latter case, mineral soil constituents, specifically clays, had intermixed with the peat making it difficult for classification. However, for a specific site investigation, this poses no problem as the required soil characteristics can be obtained from laboratory testing.

Another feature of this site, which is found to be typical of many marshy areas investigated by the Author, e.g. at Maligawatte, is the rapid variation in the thicknesses of the different soil strata. For example, the thickness of the peat layers which is a maximum in the vicinity of boreholes BH 4 and BH 7 (the total thickness of the 2 peat layers here is about7m), is found to change rapidly across the site and it is virtually zero in BH 9. This has a significant effect on the design of foundations as the peat layers are highly compressible and large variations in their thickness give correspondingly large variations in the settlements of the structures.

Since pile foundations are an alternative which has to be considered, especially for buildings of 3-storeys or higher, the depth to rock has to be determined in the site investigation. The results of Figs. 2a and 2b show that there are large variations in the depth to rock, (again a common feature of the marshy areas), and hence when piles are to be used the length of piles is expected to change significantly across the site. Therefore, the type of piles used should be one in which it is easy to change the pile length; eg. cast in-situ bored piles.

From the above features it is clear that in a site investigation of a marshy area, a sufficient number of boreholes must be done to ascertain the variations in soil profile across the site. Hence the cost of such an investigation will be greater than the corresponding cost at a site where the soil conditions are more uniform.

3.2 Vane Shear Tests

Two of the boreholes (BH 3 and BH 7) were done also for the purpose of carrying out vane shear tests, whilst Standard Penetration Tests and taking of undisturbed samples were done in the other boreholes.

In taking undisturbed samples from the peaty soil, the danger exists that

i. retaining the soil in the sample tube may be difficult if the soil is very weak; and

ii. even when the soil is retained in the tube, changes in natural moisture content could occur across the sample. Many peaty soils have a high permeability, and there is possibility of drainage under gravity in the sample tube.

Since changes in moisture content and structural disturbance of the soil texture are found to significantly affect the shear strength of the soils, it was decided to supplement the laboratory shear testing of the peaty soils with in-situ vane shear tests. Considering the magnitude of the site investigation, this was possible and 2 out of the 9 boreholes were used for carrying out vane shear tests.

A comparison of the laboratory triaxial test results and the field vane shear test results at this site showed that the two sets of results were reasonably comparable. A more definite conclusion was not possible because the results indicated relatively large variations in the strength of the peat both in location and in depth. The results of the vane shear tests in BH 7 given in Fig. 3 show some typical variations of the shear strength with depth.

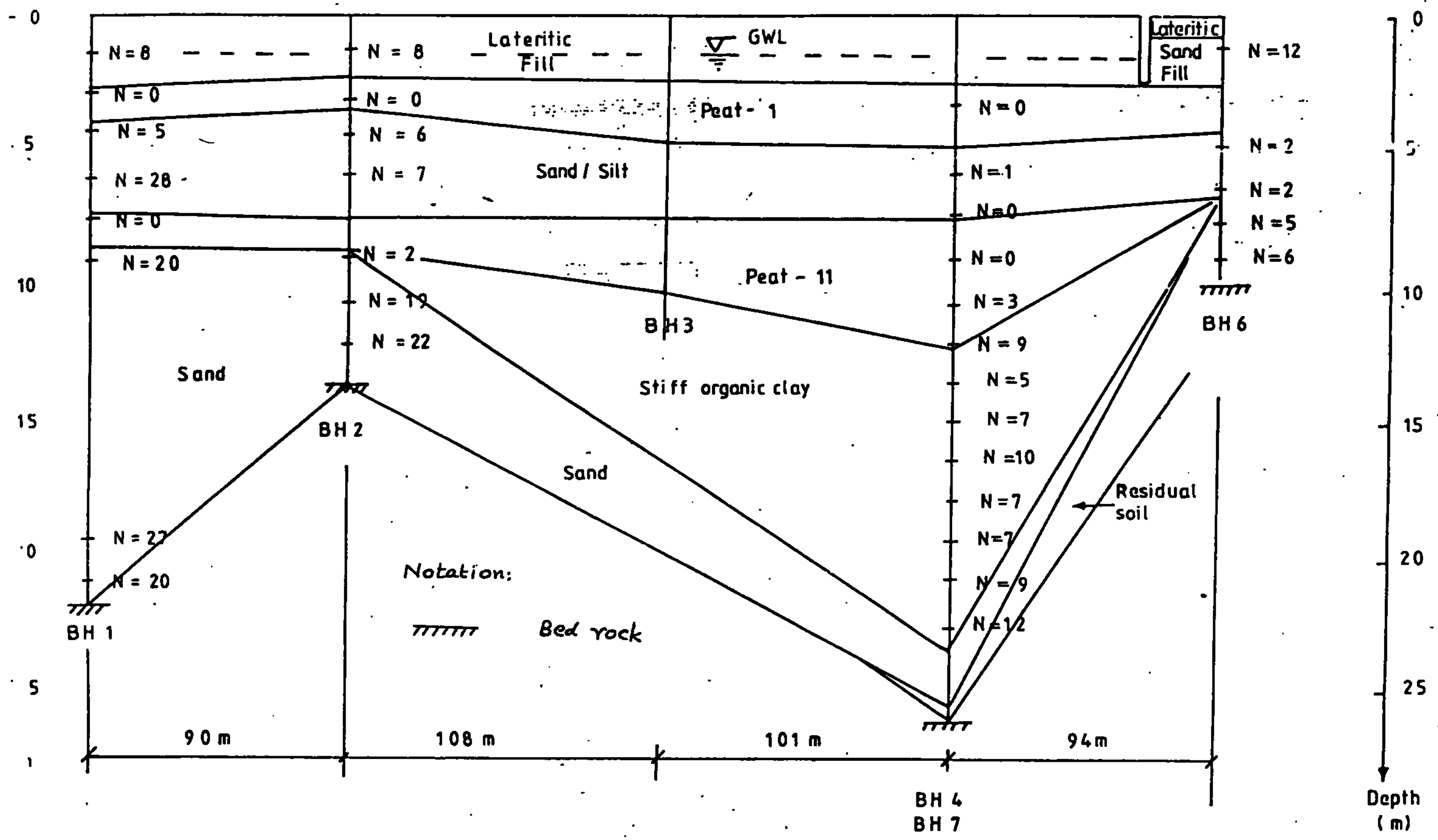

Fig. 2a - Cross section across Boreholes BH1 - BH2 - BH3 - BH4 - BH7 - BH6

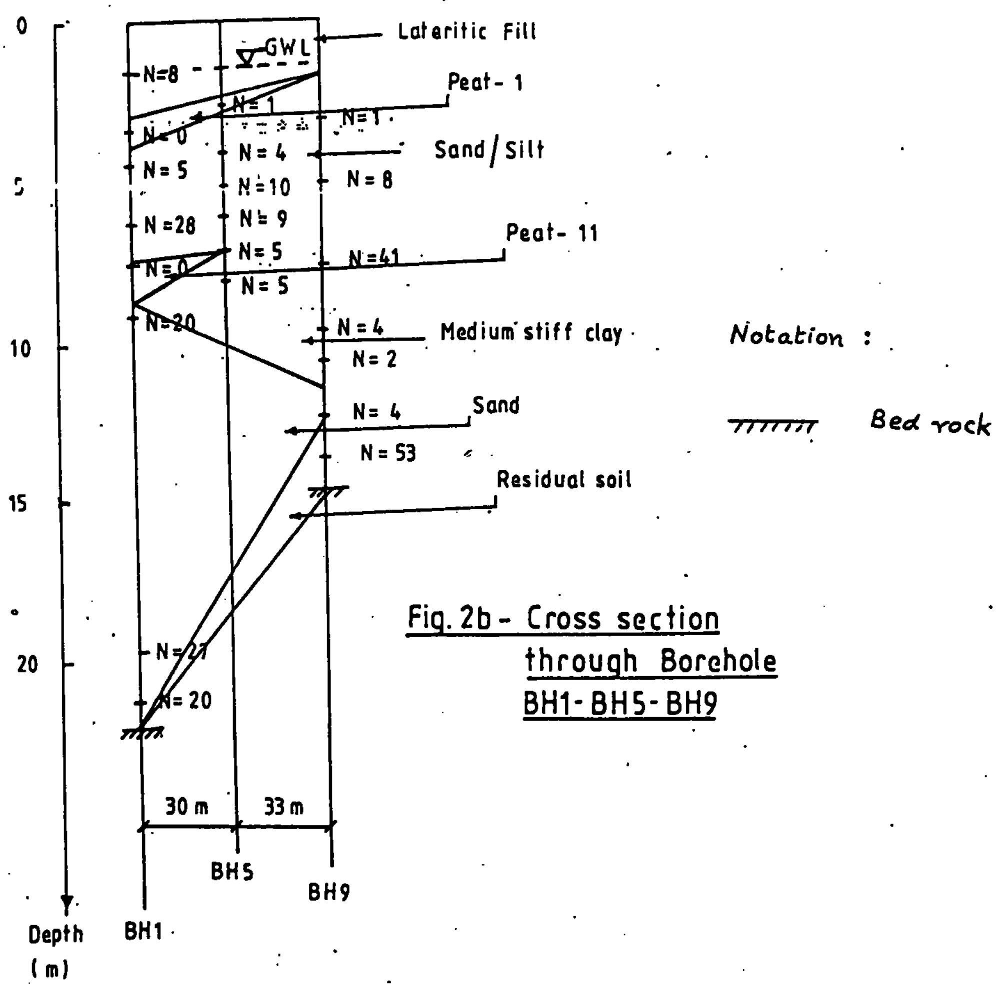

Fig. 2b - Cross section through Borehole BH1-BH5-BH9

4.0 Ultimate Bearing Capacity

Using the results of both the laboratory and field shear tests, it was found that the undrained cohesion

a) in Horizon-1 (called Peat-I) varied between 25 and 45 kN/m²;

b) in Horizon-3 (called Peat-II) varied between 25 and 100 kN/m².

(It should be noted that these peat layers have already undergone pre-consolidation due to the load of about 2.5 m of compacted lateritic fill which had been placed about 2 years previously).

Taking even a conservative value of 25 kN/m² for the undrained cohesion, the ultimate bearing capacity is of the order of 140 kN/m². From subsequent analysis (in Section 5) it is seen that the foundations are more likely to fail by excessive settlement rather than by shear failure of the soil.

5.0 Settlements of Structures

A study of the soil strata given in Figs. 2a and 2b show that there are 3 compressible layers which have to be studied in relation to foundation settlement. These are the two peat layers and the loose sand/silt layer.

5.1 Consolidation settlement of the peaty soils

The standard method of predicting consolidation settlements in the field is to

i. carry out a laboratory test on thin undisturbed samples of clay (about 20 mm thick); and

ii. Use the theoretical model of Terzaghi for predicting the field settlement from the laboratory results.

In this study, changes were made to both these procedures.

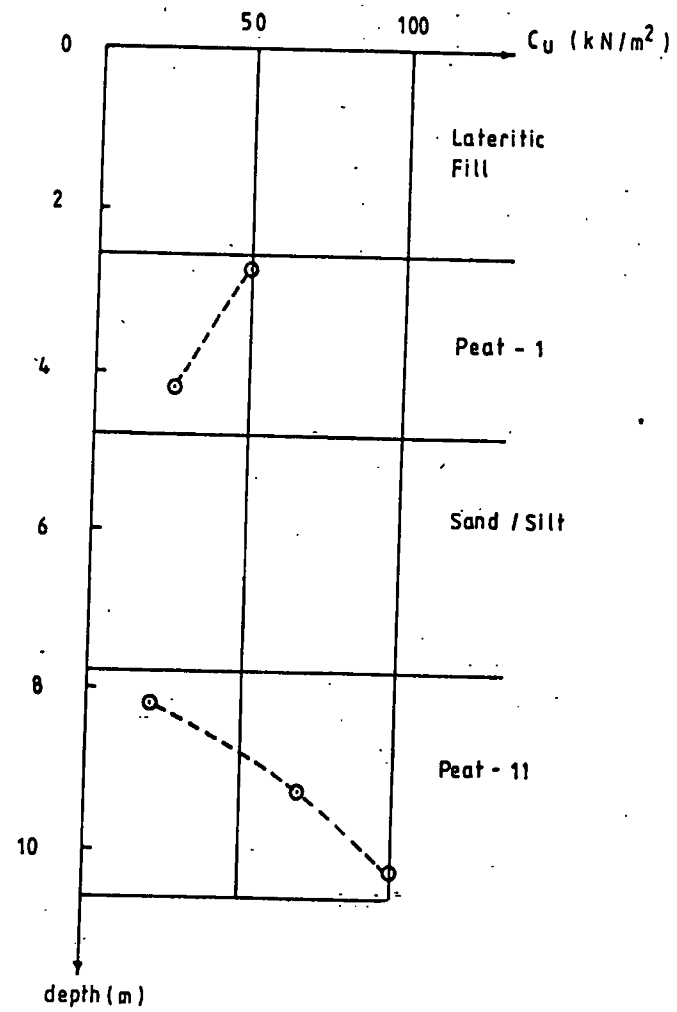

Fig. 3 - Vane shear test results in BH7

5.1.1 Laboratory consolidation tests

Since peaty soils have large secondary consolidation settlements, both the primary and secondary consolidation characteristics have to be determined. This is usually done for 4 stress increments; eg. (0-25), (25-50), (50-100) and (100-200) kN/m^2. In the standard method, the primary characteristics are determined by keeping each stress increment for 24 hours. If the secondary characteristics are also required, then each stress increment is kept on for a longer duration.

When the secondary settlements are large (as in peaty soils), the secondary settlements of the preceding stress increments affect the settlement readings of subsequent increments. Therefore, the following modified testing procedure was adopted.

The primary characteristics were determined from 'multiple increment loading tests' in which each stress increment was kept on until about 95% of the primary consolidation was completed. For the samples tested from this site, this usually varied from about 4 minutes to 10 minutes.

The secondary characteristics were obtained from 'single increment loading tests' where each sample was tested only for one stress increment. Although some tests were carried out for 72 hours, it was found that for these soils a test duration of 24 hours was sufficient to obtain the necessary secondary characteristics.

5.1.2 Theoretical models for consolidation

a) Primary consolidation

It is now recognized that the conventional Terzaghi model is not applicable to peaty soils because these soils undergo large strains, they have a highly variable coefficient of permeability, and the solid particles themselves are compressible. Theoretical models which take these variations into account have been developed by BERRY AND POSKITT (1972), MESRI AND CHOI (1985). However, the equations are very complex and a large amount of computer power is necessary for their solution.

b) Secondary consolidation

It has been observed that secondary consolidation in the laboratory follows the equation

$$s_t = s_o + c_s.H.\log(t/t_o)$$

where s_t is the settlement at time t,
s_o is the settlement at time t_o,
H is the thickness of the sample, and
c_s is a constant.

It has also been pointed out by TERZAGHI (1953) that secondary compression in peat takes place not only due to the gradual re-adjustment of particles to a more stable position, but also due to gradual lateral displacements (shear distortions). The latter displacements are absent in the laboratory consolidation test, and hence the settlements in the field will be more than that predicted from the laboratory test.

5.1.3 Prediction of field settlement

Perhaps the most telling comment of the inadequacies of existing theories was that by LAMBE (1973) in his Rankine Lecture, when after an extensive analysis of prediction versus observed behaviour came to the conclusion that "simple predictive techniques frequently yield better estimates than more sophisticated methods".

The best known example of the long term settlements are those occurring beneath the dykes in Holland, many of which are founded on peat. The Dutch have kept records of these settlements since 1880, and BUISMAN (1936) showed that again the settlement had a straight line relationship with the log of time.

The settlements of a number of buildings in Stage I of the complex at Nawala have been monitored. A typical settlement curve for one of the blocks is shown in Fig. 4a, and the log of a borehole situated close to the block is given in Fig. 4b. It is seen from Fig. 4a, that the entire primary settlement has been completed in about 90 days, and the long term settlement follows the straight line relationship observed by Buisman. The coefficient of secondary consolidation c_s as determined from Fig. 4a is 11.0×10^{-3}, and as expected it is higher than the value determined from laboratory testing which was 5.0×10^{-3}. This clearly demonstrates the inadequacy of the determination of c_s from laboratory consolidation tests for the prediction of settlements in the field in peaty soils.

Therefore, the following simplified procedure was followed for predicting the field settlement:

1) The ultimate primary settlement was computed using the characteristic m_v. (m_v is the coefficient of volume decrease defined as [(v/v)/). The value of this parameter was determined as
1.7×10^{-3} m²/kN for Horizon-1 (Peat-I); and
0.2×10^{-3} m²/kN for Horizon-3 (Peat-II).
2) The time for the end of primary consolidation was computed using the (t/H^2) law.
3) The secondary settlements were computed using a value for c_s of 11.0×10^{-3}.

5.2 Settlement of the sand/silt layer

The settlement of the sand/silt layer was determined by the standard method of relating settlements to the average N-value.

5.3 Total settlement

The total settlements would depend also on the magnitude of the structural loads to be transferred to the ground. For example, Table 1 has been prepared for the building Academic Administration (Science) assuming a raft foundation.

Bearing pressure (kN/m^2)	Settlement (mm)	
	at end of primary consolidation	after 25 years
20	42	74
30	63	95
40	84	116

Table 1

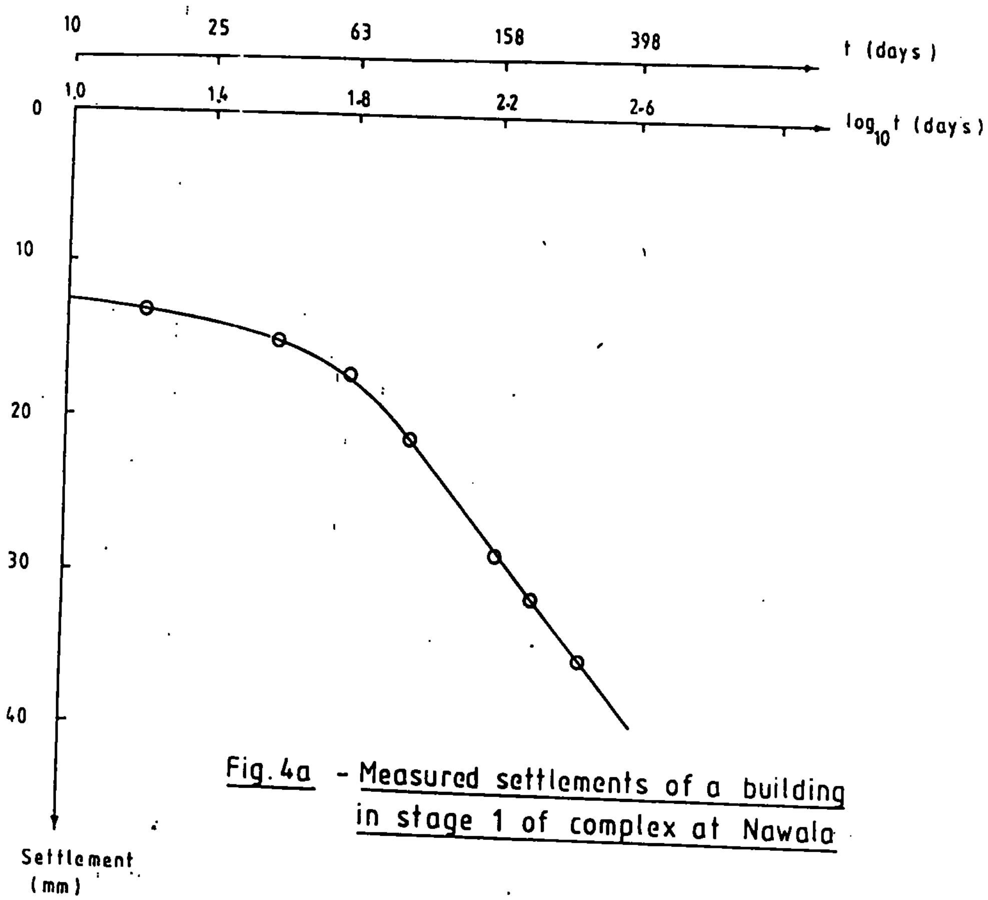

Fig. 4a - Measured settlements of a building in stage 1 of complex at Nawala

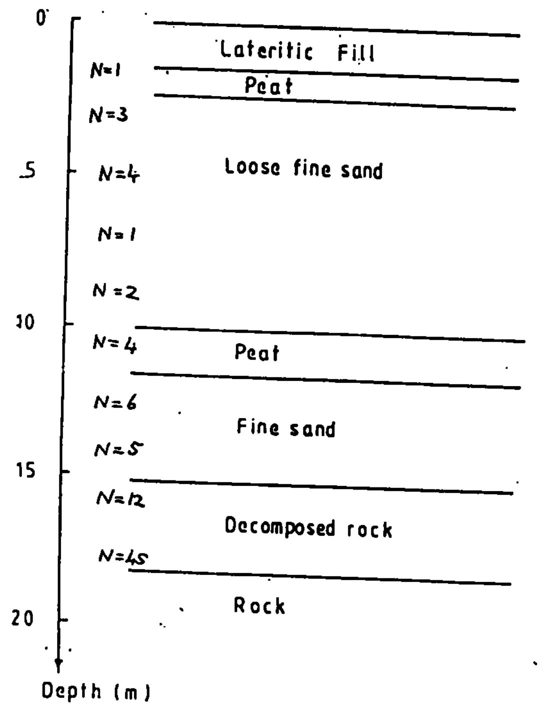

Fig. 4b - Log of borehole situated close to building monitored in Fig. 4a

5.4 Allowable settlement

LEONARDS (1987) recommends limiting total settlements to 50 mm in sands and 75 mm in clays. In a study of several buildings in the low lying areas of Colombo, TENNEKOON, SIVAKUGAN AND LAKSHMAN (1988) find that the limiting settlements could vary appreciably depending on the overall stiffness of the structure in relation to the ground. For example, the buildings of Stage I of this complex were also monitored and some of the buildings showed signs of distress at settlements as small as 46 mm. The stiffness of these buildings relative to the ground was found to be small because

i) the buildings had large dimensions in plan; and
ii) there was little brickwork between floors to provide stiffness to the structure.

TENNEKOON, SENANAYAKE AND AMERATUNGE (1989) have proposed a method for checking the safety of a structure from settlement considerations. This method has been obtained from the study of existing buildings in the marshy areas. However, in all the buildings which were studied, the foundations have consisted of either individual pad footings or strip footings. Therefore, it is expected that when raft foundations are used, the allowable settlements would be more than given by the method of Tennekoon et al.(1989).

6.0 Parameters for Design of Shallow Foundations

6.1 Allowable bearing capacity

The usual parameter expected from a site investigation for the design of shallow foundations is the allowable bearing capacity. As discussed in the preceding sections, the allowable settlements rather than safety from shear failure will often govern the magnitude of the allowable bearing capacity.

6.2 Coefficient of subgrade reaction

When designing buildings in marshy areas, it is necessary to incorporate the effect of soil-structure interaction; i.e. it is necessary to be able to evaluate the redistribution of stresses and moments as a result of foundation settlements. One of the simpler methods of doing this for combined footings and raft foundations is by using the "coefficient of subgrade reaction method" given by BOWLES (1968). This requires the determination of the coefficient of subgrade reaction (k_s) of the soil.

The standard method of determining k_s is by carrying out a plate bearing test. Considering that the plate is often only 30 cm square and that the marshy areas have several layers of soil which would be affected by the foundation, it is clear that the plate bearing test can at best provide k_s only for the layer at which the test is done.

An alternative method which was used at this site was to estimate the modulus of elasticity (E_s) of the different layers, and then evaluate the average E_s for the site for a given width of foundation. This method of calculation has been shown by TENNEKOON, SIVAKUGAN AND LAKSHMAN (1988).

In evaluating the modulus of elasticity of the different layers, the E_s for the peat layers was determined from the triaxial test results, while the E_s for the sand/silt layer was obtained from the average N-value using the charts given by Menzenbach and reported by TOMLINSON (1976). The coefficient of subgrade reaction can then be calculated using the empirically established relationship of Vesic reported by BOWLES (1974), which is

$$R_s \quad \frac{E_s}{B(1 - ^2)}$$

where B is the width of the foundation,
and is Poisson's ratio of the soil.

7.0 Conclusions

It is shown that one of the features of this site is the rapid variation in thickness of the different soil strata. These large variations in the thickness of peat and other compressible soils give correspondingly large variations in the settlement of structures which will significantly affect the design of foundations.

Pile foundations will be inevitable when the expected settlements are excessive. It is shown that another feature of this site is the rapid variation in the depth to rock. Therefore, if piling is required, then the type of piles used in such cases should be one in which the pile length can be easily changed.

These large variations in thickness and depth to rock mean that more investigation will be required in these areas than in sites where the soil conditions are more uniform.

Vane shear tests are shown to be useful in supplementing the laboratory shear tests in very soft cohesive soils. However, analysis has shown that settlement criteria rather than shear failure of the soil are likely to govern the allowable bearing capacity.

It is shown that special testing techniques are needed to study the primary and secondary consolidation characteristics of peaty soils. 'Multiple increment loading tests' are used for the determination of the primary characteristics. In these tests, a stress increment is kept on only until about 95% of the primary consolidation is complete before the next stress increment is put on. For the 20 mm thick laboratory samples, this was found to be quite small - it varied between 4 and 10 minutes. The secondary characteristics are determined from 'single increment loading tests' where each sample is tested for only one stress increment. It was found that a test duration of about 24 hours was sufficient to obtain the necessary characteristics.

When considering the theoretical models of consolidation to be used to predict field settlements, it was shown that the available models are either inappropriate or far too complex for design work. Reference was made to the work of LAMBE (1973) who found that simple predictive techniques frequently gave better estimates for settlements than more sophisticated methods.

Field measurements made in buildings of Stage I of the complex showed that primary settlements were complete in about 90 days while the secondary settlements followed the straight line relationship first observed by BUISMAN (1936). Using these results a simplified procedure was used for predicting the field settlements.

In the determination of allowable settlements for buildings in marshy areas, it is shown that the simple guidelines given by Codes of Practice on limiting total settlement are not applicable and it is more appropriate to work with more refined procedures involving the determination of the stiffness of the structure relative to the ground.

When considering the design parameters for shallow foundations, it is shown that in addition to the allowable bearing capacity it is necessary to evaluate the coefficient of subgrade reaction (k_s). This latter parameter is used in some of the simpler methods of soil structure interaction analysis for the design of foundations. It is shown that the commonly used plate bearing test is inappropriate for the determination of k_s for foundations in marshy areas. An alternative method is proposed where k_s is determined from an empirical relationship relating it to the average modulus of elasticity of the soil.

8.0 Acknowledgements

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