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CONTENTS

• ***Use of Nailing Technique for Stabilization of Slopes*** Mettananda, D.C.A., Premarathne, B.D.H., Priyantha H.M.S. University of Moratuwa	1 - 4
• ***Optimum Geometrical Shapes for Gravity Retaining Structures*** De Silva L.I.N., Hettiarachchi, J.W., Dhammika A.C.P.C.P. University of Moratuwa	5 - 8
• ***Introducing Geo Pro Version 1.0A for 3D Modelling*** Wijesekara, G.N., Wickramasinghe, A.M., WickramanayakE, A.N. University of Moratuwa	9 - 12
• ***A Geotechnical Analysis of a Shallow Foundation Systems Using a Spreadsheet*** Adrian Swampille Open University of Sri Lanka	13 - 16
• ***Subsurface Investigation Using Electrical Resistivity Method at Engineering Faculty Premises*** Jayanathan M., Tharmendira, R.I., Thevaragavan, M., Jayawardena, U. De S. University of Peradeniya	17 - 20
• ***Soil Nails and their Pullout Resistance*** Chandrakumar, S., Krisnagopalan, N., Kugan, R., Fernando, G.S.K. University of Peradeniya	21 - 24
• ***Computer Program for cost analysis of Earth Retaining Walls*** Jeyisanker, M., Kalyani, N. University of Peradeniya	25 - 28
• ***Classification of Sri Lankan Residual Soils*** Aravinthan, R., Aravinthan, K, Baskaran, K., Fernando, G.S.K. University of Peradeniya	29 - 32
• ***Engineering Properties of Sri Lankan Residual Soils*** Gunaranjan, G.J., Thivakaran, M., Fernando, G.S.K. University of Peradeniya	33 - 36
• ***Bearing Capacity of Strip Foundation on Ground Improved by Sand Column and Sand Replacement*** Sivakumar, R., Sivakumar, T., Logitharan, R.L., Kurukulasoriya, L.C. University of Peradeniya	37 - 40
• ***Control of Differential Settlements in Foundations*** Sathananthan, I., Sivakanthan, S., Fernando, G.S.K. University of Peradeniya	41 - 44

USE OF SOIL NAILING TECHNIQUE FOR STABILIZATION OF SLOPES

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ABSTRACT

Soil Nailing technique is widely used in many countries to stabilize slopes and embankments with many proven advantages including low cost. This research was aimed at studying the behavior of soil nailed structures in Sri Lankan conditions and to find the improvement achieved in the factor of safety.

Limit equilibrium methods were incorporated in the analysis for the design of soil nailed structures. Two analytical models were developed to analyze the stability of the nailed soil mass using the Bishop's method and the Janbu's simplified method. Pullout resistance of nails, embedded in Sri Lankan soils were determined in the laboratory. Laboratory model slopes were then constructed and loaded to failure. Data relating to the failure surfaces were back analyzed using the analytical models. In the final phase, an attempt was made to design soil nail arrangements for some critical slopes in the hill country.

Model test results indicated that a remarkable increase of the stability could be obtained by soil nailing. In addition, it revealed that the pullout resistance of a nail is greatly influenced by the moisture content. Also, disturbances during driving the nails reduced the pullout resistance of the driven nails.

1. CURRENT STATE OF THE ART OF SOIL NAILING TECHNIQUE

Soil Nailing is a practical and cost-effective technique to stabilize slopes and excavations through the introduction of reinforcement into the soil mass. Nails, generally made of steel, are installed in a cut face in original ground on a pattern which may or may not be varying with wall height; installation process may be driving using a hammer or placing in a pre-drilled hole and subsequent grouting. The nails are connected by steel mesh or shotcrete facing to hold soil near the cut face in place between them. Sometimes precast or cast-in-place concrete facades or seeded mats are used to improve the appearance of the facing for permanent facilities. Seeded mats are the latest development, and it has become very popular today due to its ability to blend with the environment. Corrosion resistance is achieved by the nail-grout interaction or by hot dip galvanizing. Several proprietary nail systems, which can resist corrosion, have also been developed in the recent past. The main difference between nailing and other ground reinforcing techniques is that the nailing is done for in-situ ground in a top down process, while the other reinforcement techniques are used to reinforce new constructions in a bottom up process (Figure 1).

The nails strengthen the ground by helping the soil to resist deformations. As the ground deforms, the nails share the load with the soil, gradually becoming more stressed in tension from an initially nominal stress level at installation. As the soil deforms, the nails become tensioned to the extent that is necessary to arrest the deformations and stabilize the soil. The nails are therefore a technique of passive stabilization unlike tiebacks, which are pre-tensioned at installation.

The first soil nailed wall available in literature was the cut slope stabilization in Versailles, France (1970). Thereafter, it gained popularity in many other countries such as UK, USA, Australia, Hong Kong, Japan etc.

Soil Nailing has been used successfully in temporary and permanent applications, in new and remedial construction, and in rural and urban settings. The categories of applications can be identified as excavation support, stabilization of slopes and retaining wall repair.

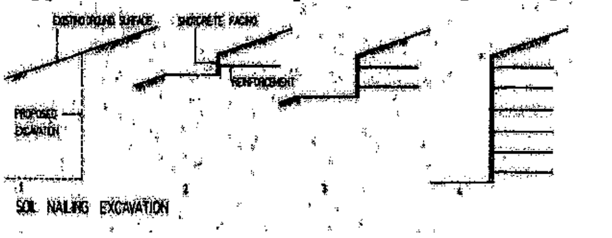

Figure 1: "Top down" construction sequence in soil nailing (after Bruce et.al. ,1986)

Main advantages associated with the technique are the low cost, fast construction rate, design and construction flexibility, use of light construction equipment and the environment compatibility. However, soil nailing technique has some problems in soft clays, due to lower shear strength parameters.

2. ANALYTICAL MODELS

The existing design methods for soil nailed retaining structures can be broadly divided into two categories, viz. Limit Equilibrium Design Methods (or Modified Slope Stability Analysis), and, Working Stress Design Methods. Slope stability analysis procedures have been modified to consider the shearing, tension, or pull-out resistance of the inclusions crossing the potential failure surface. Some approaches reported in literature, such as Davis method, French method etc., utilizes limit equillibrium techniques involving various failure mechanisms such as slices [ASCE, 1997]. However, in some approaches, a modified wedge method had also been used [Johnson et.al.,1995]. The available commercial packages for soil nailing designs are extremely expensive. Furthermore, they do not possess the flexibility required in a research project. Therefore, for the purpose our research, two analytical models were developed using modified slices methods. This will enable the modifications that would be needed under special Sri Lankan conditions.

The model developed using the Bishop's method can be used to evaluate the circular failure surfaces, and, the derived equation is given as,

$$F = \frac{\sum \left\{ \left[c'\Delta x_i + \left(W_i + Q_i - u_i \Delta x_i + T_N \sin\alpha \right) \tan\phi' \right] \left[1 / M_i(\theta) \right] \right\}}{\sum \left[\left(W_i + Q_i \right) \sin\theta_i - T_N \cos\left(\theta_i + \alpha \right) \right]} \quad \text{...(1)}$$

where,

$$M_i(\theta) = \left(\cos\theta_i + \frac{\tan\phi' \sin\theta_i}{F} \right) \quad \text{..........(2)}$$

In these equations, c and ϕ represent soil strength parameters. u represents the pore pressure, W and Q represents the weight and the surcharge on a particular slice, while θ is the angle of the tangent of the slice to the horizontal. T_N is the mobilized tensile force in a nail, and α represents the nail inclination to the horizontal. Δx represents the slice width and F is the factor of safety.

The mobilized force in the nails is the lesser of the pullout resistance and the tensile strength of the nail. It has to be emphasised here that only the tension in a nail is taken into consideration.

The model developed using Janbu's simplified method can be used to analyse the non-circular failure surfaces as well. The derived equation is given by,

$$F_0 = \frac{\sum \left\{ \left[c'\Delta x_i + \left(W_i + Q_i + T_N \sin\alpha_i - u_i \Delta x_i \right) \tan\phi' \right] \frac{1}{n_\theta} \right\}}{\sum \left[\left(W_i + Q_i \right) \tan\theta_i - T_N \frac{\cos(\alpha + \theta_i)}{\cos\theta_i} \right]} \quad \text{......(3)}$$

where,

$$n_\theta = \cos\theta \left(\cos\theta_i + \frac{\tan\phi' \sin\theta_i}{F_0} \right) \quad \text{..........(4)}$$

All parameters are similar to those in the Bishop's model. After obtaining F_0 from the above equations, the modification factor f_0 is obtained from the charts derived by Janbu, and the final factor of safety is given by,

$$F = f_0 F_0 \quad \text{.....................(5)}$$

Two spreadsheet programs were developed for the calculations, based on the above models. Each spreadsheet has three worksheets. They are named as; Input/ Output Worksheet, Calculations Worksheet and Nail Force Worksheet.

The input/ output worksheet provides a space for the user to enter input data corresponding to a selected failure surface. The nail force worksheet reads data from input worksheet, and, calculates the resistance provided by nails, which pass through that particular slice. The spreadsheet calculates allowable loads due to both tensile strength of the nail and pullout resistance, and reads the minimum value out of them. The calculation worksheet reads data from other two worksheets, and, performs the calculation to find out the factor of safety for the selected trial failure surface. This factor of safety is then transferred to the input/output worksheet, so that the user can directly read it from the input/output worksheet.

To obtain the dimensions of the slices for the trial failure surface considered, another model was developed using AUTOCAD. Through this model, once the slope outline is drawn, any number of failure surfaces can be drawn, and the dimensions could be obtained using some fixed slice boundaries within a very short time period (5 minutes).

3. PULLOUT TESTS

Pullout resistance tests were conducted on the soil nails prior to the model studies in order to decide on a suitable nail diameter for the model studies, and to find out the bond coefficient available under the test conditions. Tests were done both with a sandy fill and with a conventional lateritic fill. The soil compacted to a required density inside a box was expected to represent the natural soil.

The test box had dimensions 0.3m x 0.4m x 0.3m, and, the boundary stresses were applied from the top. The effective length of a test nail was 0.5m. The pullout load was applied through a cable attached to the nail (Figure 2).

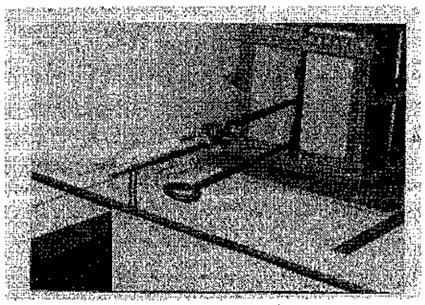

Figure 2: Test Apparatus

Tests on sand were performed for both 10mm and 12mm diameter reinforcing bars. Both plain bars and deformed bars were tested. As the tests on sand showed that, not much increase in pullout resistance could be obtained by using 12mm bars, the tests on lateritic soils were limited only to 10mm bars. The vertical surcharges of 20kPa, 40kPa, 60kPa and 80 kPa were used in the tests.

The equation used to calculate the pullout resistance is given by,

$$F = f_b * \pi D l (c' + \sigma' \tan\phi) \quad \text{.............(7)}$$

where, D and l are the diameter and length of the nail, and f_b is the bond coefficient.

The properties of soils were obtained using direct shear tests, sieve analyses, Atterberg limit tests and Proctor compaction tests. Sand was found to have $\phi'=32^0$, and, lateritic soil, had c' = 20 kPa and $\phi'=36^0$. Maximum dry density was 1770 kg/m^3, at an optimum moisture content of 16%.

With deformed bars, pullout resistances were expected to be greater than that of the plain bars due to the surface roughness. But, the observations were contradicting to this, and, the conclusion that could be arrived is that there had been more disturbances during the driving of deformed bars. Similar observations were made by other researchers as well [Raju, 1996]. Hence, it was decided that the deformed bars are not suitable for the test.

The experimental values obtained for the bond coefficient were 1.5 for sand and 1.6 for lateritic soil.

4. MODEL STUDIES

Model studies were performed with the objectives of identifying the failure loads and failure patterns with and without nails for both sands and lateritic soils, and, of verifying the developed analytical programs using the model study data. Although the Soil Nailing is a technique that is used to improve the stability of existing natural slopes, a slope was formed by compacting the soil, for the model studies. Models were constructed inside a perspex box having dimensions 1.2m x 1.2m x 1.0m.

For sand models, sand was placed in 50mm layers and the density was maintained at 1700 kg/m^3. A moisture content of 10% was maintained to facilitate compaction. Alternate layers were coloured in black, in order to visualize the failure surface easily. Loading was applied by jacking against a loading frame. The loading arrangement is shown in figure 3.

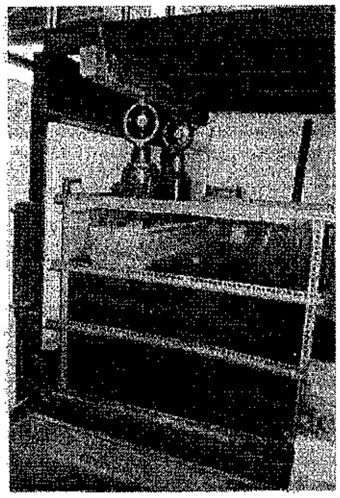

Figure 3: The loading arrangement

First sand model was constructed with a slope of 33^0, and the top surface area of 1.2m x 0.4m. The ultimate failure load was 79 kPa. Since this was a higher surcharge than expected (which was around 3 kPa), the next model was prepared with a slope of 45^0, and, a top surface of 1.2m x 0.6m. It was nailed with 16 no. of 0.5m long, 10mm plain reinforcing bars, which were spaced 0.1m vertically, and, 0.3m horizontally. Nails were driven at 12^0 to the horizontal. The facing consisted with a chord mat placed in between some steel strips. The face, after nailing is shown in figure 4. When the model is loaded, it could withstand a surcharge of 180 kPa before failure. Also, it was observed that the nails had been bent considerably.

In both occasions, considerable outward movements could be observed on the slope face. Failure surfaces could be reproduced by taking sections of the model and they were back analysed using the analytical models. One failure surface is shown in figure 5, and a section showing the outward movement is shown in figure 6.

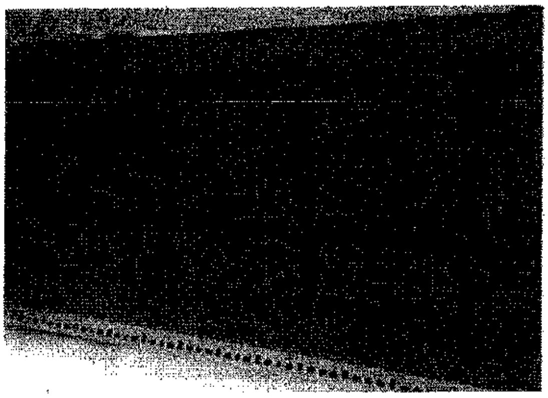

Figure 4: Facing Arrangement

Figure 5: Failure surface for sand model without nails

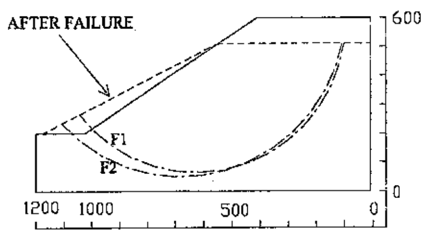

Figure 6: Slope Geometry before and after failure

In the lateritic models, the density was controlled by applying a measured compaction effort to a measured volume of soil. Moisture content was maintained at 16%-the optimum. The model without nails was compacted using "Proctor equivalent energy", and the slope angle was 45^0. The model was saturated before loading, and, loaded upto a surcharge of 480 kPa. Some surface cracks were seen. A shallow failure surface propagated in front of the load. However, further loading was not possible due to the limitations of the proving rings. Hence, the next model (with nails) was compacted using a lesser compaction effort (35% proctor), with the view of obtaining a failure within the practical loading range. The slope angle was also increased to 60^0. Similar nailing arrangement as in sand model was used, and, the model was saturated before loading. When loaded, it could withstand a surcharge of 500kPa, and. even the local failure surface could not be observed, except for few local cracks.

5. BACK ANALYSIS OF RESULTS

The failure surfaces (F1 & F2) obtained for the sand model without nails were circular, and, hence, back analysed using the model that used the Bishop's method. A parametric analysis was performed for a range of shear strength parameters, $c' = 0$ -2 kPa, and $\phi' = 32 - 34^0$. The results are presented in table 1, and, it could be observed that a reasonable estimation is achieved for $c' = 2$ kPa, and $\phi' = 32^0$.

Slip Surface	q (kPa) For $c'=0$, $\phi'=32^0$	q (kPa) For $c'=1$ kPa, $\phi'=32^0$	q (kPa) For $c'=2$ kPa, $\phi'=32^0$	q (kPa) For $c'=0$, $\phi'=34^0$	q (kPa) For $c'=2$ kPa, $\phi'=34^0$
F1	40	60	85	50	110
F2	30	50	60	40	79

Table 1: Results of the back analysis (Sand model without nails) – (see figure 6 also)

This supports the presence of some apparent cohesion in wet sand. Similar observations had been made by some other researchers also.

Failure surface for the nailed model (sand) was non-circular, and, hence, back analysed using Janbu's model. The same failure surfaces were back analysed for both with nails, and, without nails conditions. Again, a range of parameters were considered, and with $c' = 2$ kPa, and $\phi' = 32^0$, the improvement that could be expected in the presence of soil nails is 400% (table 2).

Slip Surface	q (kPa) For $c'=0$, $\phi'=32^0$	q (kPa) For $c'=1$ kPa, $\phi'=32^0$	q (kPa) For $c'=2$ kPa, $\phi'=32^0$	q (kPa) For $c'=0$, $\phi'=34^0$	q (kPa) For $c'=2$ kPa, $\phi'=34^0$
with	20	50	100	50	180
without	4	15	25	5	30

Table 2: Results of the back analysis (Sand model with nails)

However, the experimental ultimate load is even higher, and it showed an improvement of 700%. With the observation of bent up nails, this was explained as the utilization of the bending stiffness of the nails.

For lateritic models, the local failure surfaces for the model without nails could not be analysed using the models developed. However, it could be clearly observed that the model with nails could resist the shallow failure surface although having a lesser degree of compaction.

6. APPLICATION TO SOME REAL SLOPE PROBLEMS IN SRI LANKA

Sri Lanka has been experiencing number of landslides in many parts of the central and southwestern regions. Some of the major slides taken place recently are Watawala slide, Naketiya slide, Hela-Uda slide (Ratnapura), Beragala slide and Haldummulla slide.

Almost all the landslide events reported were taken place after some heavy rain. It indicates that the slopes that are stable in dry weather have become unstable, once high pore water pressures are developed. Also the accumulation of small deformations taking place over a period of time will reduce the peak shear strength parameters to the residual values. This will eventually lower the factor of safety of the slope. Since there is a high possibility that shear strength parameters have been lowered to residual values, there should be some external support systems to stabilize those slopes. Soil Nailing gives a means to provide this support through its passive inclusions.

Major difficulty faced here was in obtaining of past records. In the absence of direct records, some analysis was done based on the best possible information that could be derived from the fewer data available. Hence, the results presented here are approximate.

Two cases, Watawala, and, Beragala, were analysed. Watawala slide had some deeper failure surfaces, and, required long nails in the order of 30m to stabilize using the soil nailing alone. Since the maximum nail length reported in literature is 23m, the most economical solution may be to improve surface and subsurface drainage, while using soil nailing to stabilize shallower parts. However, in Beragala slide, it was found that, a part of the slide could be stabilized using a nail arrangement with a vertical and horizontal spacing of 1.2m x 1.2m. Proposed nails were of lengths 12m and 18m, with an angle of 15^0 to the horizontal.

7. CONCLUSIONS

From the sensitivity analysis, it was shown that, soil nailing could improve the factor of safety of a slope under both dry and wet conditions. Model studies were done to verify these observations. Due to the practical limitations, the tests were limited to driven nails only. Pullout resistance tests showed that the pullout resistance provided by the driven nails could be greatly reduced by the disturbances caused during driving. However, this can be overcome by the use of drilled and grouted reinforcement.

In the study of actual case histories, it was observed that most of the limited data available were corresponding to deep seated failures. Hence, they could not be stabilized using soil nailing along. The most economical solution will be to use nails in combination with drainage. However, results of the Beragala slide showed that shallow failures could be stabilized satisfactorily using nailing.

ACKNOWLEDGEMENTS

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Optimum geometrical shapes for gravity retaining structures

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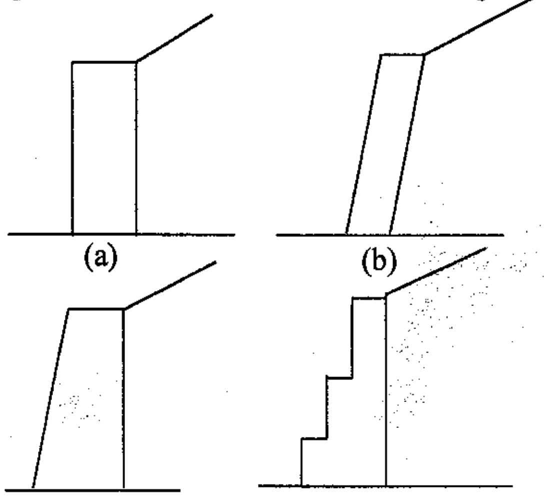
ABSTRACT: Gravity retaining structures are widely used to support soils that are not stable on their own. They are constructed with various geometrical shapes. The cost of the retaining structures increase rapidly with the retained height and it would be very beneficial to find the most optimum shapes for gravity retaining structures. The main objective of this project is to find optimum geometrical shapes of gravity retaining structures. Four basic shapes with the same cross sectional area are selected for this purpose. Any geometrical shape currently in use can be approximated with one of these four shapes.
Two Excel Spreadsheet programs were developed for the analysis and the design of a gravity retaining structure with any geometrical shape. These programs are capable of handling any ground water condition. These spreadsheet programs were verified by carrying out small model tests in the laboratory. Subsequently, that was applied to known case histories. Finally, a finite element analysis was carried out using SIGMA/W to investigate the deformation characteristics of different geometric shapes.

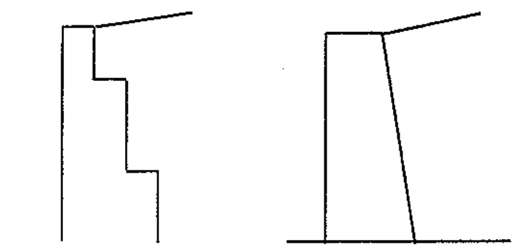
1. BACKGROUND

Gravity retaining walls provide support to the unstable soil mass through their weight. There are several different types of gravity retaining structures based on the material used and method of construction. The most appropriate type of wall depends on the ground conditions, the available technology and time periods available for construction. In the early stages of the development brick masonry or random rubble masonry with mortar joints and dry rubble masonry walls were used widely. Thereafter mass concrete was also used in the construction. At present "gabion walls", "crib walls" and "interlocking modular block walls" are getting more popular because they are easy to construct and can handle differential settlements and lateral movements effectively compared to the conventional walls made of masonry or mass concrete. Furthermore, they are effectively permeable structures and prevent the developments of high pore water pressures behind.

Different geometric shapes that the gravity retaining structures can assume can be summarized as in the Figure 1. The geometry with the back batter (Figure 1(b)) can be constructed only with more flexible gabion walls, crib walls or inter locking modular walls. In such cases the back filling could be done in parallel with the construction of the retaining wall.



(c) (d)



(e) (f)

Figure 1 – Different geometrical shapes for gravity retaining structures

2. DEVELOPMENT OF THE SPREADSHEET PROGRAM

2.1 Evaluation of the force on the wall

The objective of the analysis is to find out the critical design force exerted on the wall under the existing loading conditions. The most general analysis could be carried out with the trial wedge approach. Different types of geometries of the wall and back fill with different types of pore water condition can be analyzed by the trial wedge approach. Hence an analytical method was developed using Excel spreadsheet based on the trial wedge approach. In this method of analysis the equilibrium of the trial wedge of soil on the verge of failure was considered. In the computation of earth pressures the shape (e) was approximated by the shape (f). Thereafter resolving the forces horizontally and vertically the design force on the wall can be found for a given trial wedge angle. The wedge angle was varied and the resulting variation of the force on the wall was plotted through the spreadsheet program to find the critical design force.

Analysis can be done according to either the former British Code CP2 or the new Code BS 8002. In the former case the peak shear strength parameters are used and in the latter case mobilized shear strength parameters are used along with the minimum obligatory surcharge.

2.2 Design of the walls

After determining the design force exerted on the wall, it should be designed to withstand this force, having a sufficient safety margin on overturning, sliding and ensuring that bearing pressures are not excessive, if the design is done based on CP2. In the context of a design done according to BS 8002 the resisting forces or moments should be greater than the disturbing forces or moments. Thus the factor of safety values should be grater than unity. Also the allowable bearing pressure should not be exceeded.

Therefore, after taking a trial cross section for the wall, the factors of safety against sliding and overturning were checked and the maximum bearing pressure on the soil was evaluated. The developed spreadsheet programs were initially verified by the results obtained through small scale laboratory tests. Subsequently the program was used to back analyse Burgoyne's walls (1834) and the results were compared with the numerical analysis carried out by Harkness. et al (2000).

3 TESTING OF SMALL MODELS

Two small gravity wall models were tested in the Laboratory. These model tests can also be used to illustrate the outward wall movement during back filling and the development of active mode of failure behind the wall. Two gravity walls with equal cross sectional area were made using plywood. The width of the model was 155mm and the height of the models was 300mm. Tests were done in a special retaining wall model set up in the soil mechanics laboratory. Those Two models were made as hollow boxes and by filling those hollow units with materials of different densities; gravity walls of appropriate weight could be obtained. Those two models could be used to obtain four geometric types of gravity retaining structures (A), (B), (C), (D) by interchanging the back and front.

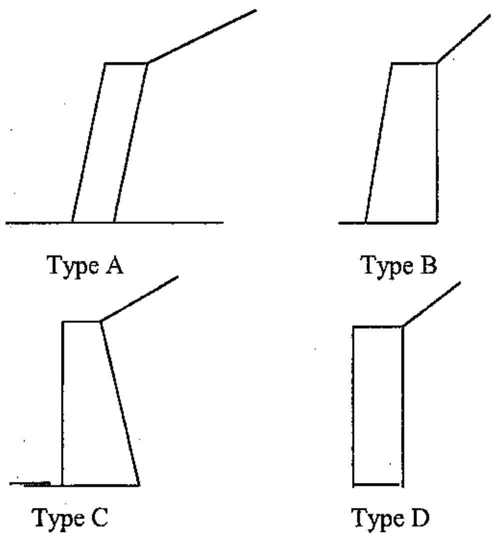


Figure 2 – Different models tested

3.1 Model testing procedure

First, Sand was mixed with a black-dye to obtain a black-coloured sand. Then the model wall was placed inside the apparatus at the correct location. The width of the wall was made slightly smaller than that of the box to minimize the effect of any end restraint. A cardboard sheet was pasted at the back of the retaining wall at its edge to prevent falling out of sand from the sides. (Figure 3) The wall was initially propped using the two props provided. A wooden block was inserted at the top to prevent bulging of apparatus during back filling.

Then the back of the wall was filled with alternative layers of black and white sands. Sand filling was carefully done to obtain the same density throughout the backfill. This was done by allowing the sand to fall freely from a constant height throughout the filling operation. The density of the back fill behind the retaining wall was found to be 15.14 kN/m^3.

Thereafter, the hollow model wall was filled with sand and the two props were released slowly to see whether any failure would take place. If there is no failure the weight of the gravity wall was reduced by gradual removal of the sand inside the model wall. Failure surface developed could be identified by observing the layers in the backfill .The soil behind the wall was seem to move outward creating a failure wedge of soil behind the wall. This was shown very clearly by the alternate sand layers of different colours. Finally, the angle of the wedge, the weight of the wall and the lateral deformation were measured. The initial position of the wall when it is supported by the props was marked on the Perspex wall so that the final displacements could be measured. Same procedure was repeated for all four different wall geometries.

It was necessary to determine the shear strength parameters for the sand and the interface parameters for the sand wall interface for the subsequent back analysis. Direct shear tests were performed to determine the soil strength parameters and interface shear tests were performed to find the plywood/soil interface properties. Friction angle of sand was determined as 26^0 and the interfaces shear angle between sand and plywood was observed as 22^0.

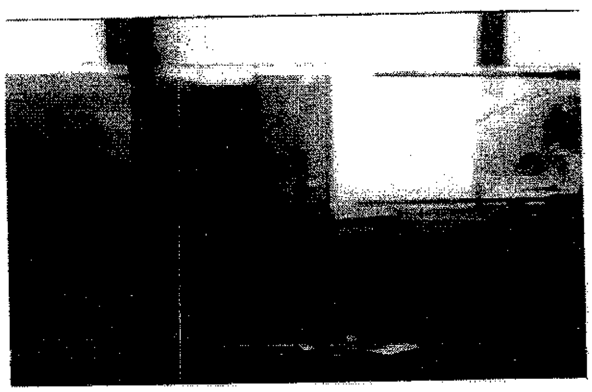


Figure 3. Model Wall D after failure

3.2 Back analysis of model tests

The small models tests were analyzed using the developed spreadsheet program, and the results were compared with the actual test results. Using the spreadsheet program, the weight of the wall required to make the F.O.S against sliding equal to one was found. This was obtained by changing the unit weight of the material inside the model until the factor of safety against sliding approaches one (Based on the old British Code CP2). This approach was justifiable as in all cases the walls failure took place through sliding. The experimental data on walls at the failure state for wall types A, B, C and D are compared with the results obtained by back analysis through the Spreadsheet program in Table 1. There is a reasonable close agreement. Both the model test results and the spreadsheet results showed that wall type A is the most optimum requiring the smallest weight.

Wall type	Weight of the wall required to cause failure by sliding (kN/m)		Wedge angle	
	Spread Sheet Analysis	Model Test Results	Spread Sheet Analysis	Model Test Results
A	0.399	0.350	45	47
B	0.465	0.450	53	58
C	0.460	0.430	55	60
D	0.465	0.442	53	57

Table 1- Comparison of model test results with Spreadsheet program analysis

4. ANALYSIS OF BURGOYNE'S WALLS

Harkness R.M et al (2000) numerically modeled the behavior of four dry masonry retaining walls done by Burgoyne (1834) in Ireland, using the universal discrete element code UDEC, and the results were compared with those of the field trials obtained by Burgoyne (1834). By using appropriate soil and wall mass densities strength and stiffnesses it was possible to simulate numerically the field experiment done by Burgoyne (1834).

Each of these walls was 6.096m (20ft) long and 6.096m high. Walls were founded on a hard rock surface. Although the mean thicknesses of all four walls were the same (1.016m), the cross section of each wall was different. Wall A (Type A- Fig 2) had a uniform thickness of 1.016m and was battered back at a slope of 1 in 5. Thickness of wall B (Type B-Fig 2) varied from 0.406m at the top to 1.626m at the base with a vertical back. Wall C (Type C- Fig 2) was of the same thickness as wall B but with a vertical front and wall D was a plane vertical wall of uniform thickness 1.016m. In all cases the masonry consist of roughly squared granite blocks laid without mortar. Each wall was backfilled with uncompacted loose earth, described by Burgoyne as "loose mould". Its bulk density on placement was 15.21kN/m^3. The angle of friction of the soil in the range of 25-28 degrees and the angle of sliding friction between blocks of granite was taken as 45^0.

4.1 Back analysis using spreadsheets

Back analysis of the Burgoyne walls was done using the developed spreadsheet program. Material properties were taken as; friction angle of soil as 25^0, the interface friction angle between soil and granite 22^0 and the density of the soil 15.21 kN/m^3 for the back analysis. The results of the Spread Sheet analysis are presented in the Table 2. The wall type A had the greatest factor safety and the lowest and most uniform bearing pressures.

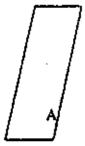
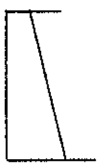
	Results obtained using Spread sheet				Burgoynes Results	
Burgs Wall type	F.O.S. on overturning	F.O.S. on Sliding	σ_{vmax} (kN/m^2)	σ_{vmin} (kN/m^2)	Soil filling height(m)	Result
 A	1.38	0.99	349.6	-26.1	6.1 5.49 5.18 4.88 4.27	NF NF NF NF NF
 B	1.22	0.87	388.1	-147	6.1 5.49 5.18 4.88 4.27	NF NF NF NF NF
 C	0.86	0.9	634.6	-352	6.1 5.49 5.18 4.88 4.27	 F NF NF
 D	0.65	0.87	1132	-749	6.1 5.49 5.18 4.88 4.27	 F NF NF

Table 2- comparision of Burgoyne observations with results of the program
NF - Not Failed

5 ANALYSIS OF DEFORMATIONS USING SIGMA/W

SIGMA/W is a general finite element software product for stress and deformation analyses of geotechnical engineering structures. Variety of different stress-strain constitutive relationships which range from simple linear-elastic, to non-linear elastic-plastic models can be chosen.

Construction and filling behind the four geometries of retaining walls selected in Burgoyne's experiment was simulated using the SIGMA/W program (Figure-4). Wall was modeled as a linear elastic material of modulus E= 2100000 kN/m^2 and the foundation soil was modeled as a elastic ideally plastic material E= 300000kN/m^2, C=0kN/m^2, ϕ =25^0 and poison ratio =

0.35. The back filling material was modeled as elastic ideally plastic material of E = 75000 kN/m^{2,} C=0 kN/m^{2,} ϕ =25^0 and poison ratio = 0.4

The model studies as well as Burgoyne studies were done with sandy soil and the water table was much below. As such, the development and the subsequent dissipation of pore water pressures was not simulated in the study.

The construction of the wall and incremental filling up behind the wall was simulated for all the wall geometries and the deformation patterns could be plotted along with the mesh to an exaggerated scale.

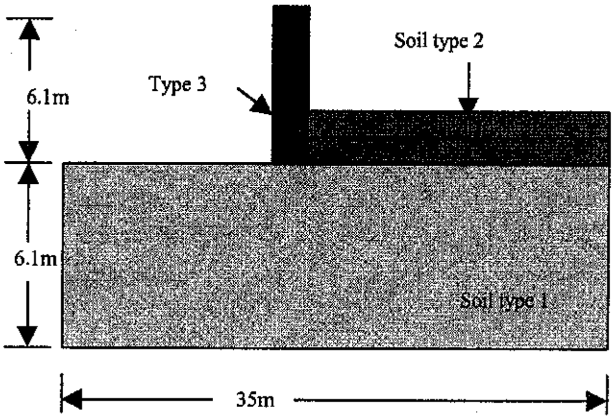


Figure 4 – Numerically modeling of Burgoyne walls

The relative stability of the four geometrical shapes could be assessed by comparing the increase of deformations at a node at a critical location such as the top of the wall.
The increase of the deformations of the top of the wall with incremental filling up is presented in Figure 5. Wall A has the smallest deformation of 51mm and wall D has the highest of 91mm. The rate of increase of the displacement with incremental filling was also the smallest for the wall A.

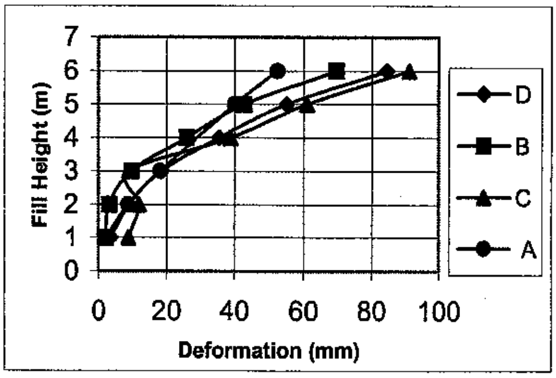


Figure 5- Comparison of wall deformations

6 CONCLUSIONS

The analysis done in this project with the help of limit equilibrium approach revealed that the wall with a back batter has the most optimum shape. This is because that the horizontal component of the earth pressure is a minimum in this case, and that the masonry is placed where its mass has the greatest stabilizing effect. A wall with a sloping front face is more stable on overturning than a vertical wall with parallel sides and of same cross sectional area. While the lateral force that has to be resisted is the same in each case, the lever arm of the center of the mass of the masonry is greater in the case of the wall with a sloping front. However, on sliding instability, factor of safety is the same for both these types. A wall with sloping back is marginally less stable than a vertical wall, because the effect of the batter in this case is to increase the thrust of the soil and therefore the overturning moment more significantly than the restoring moment due to the weight of the wall.

The model test results, spreadsheet analysis and SIGMA/W based finite element analysis, all confirmed the above findings. SIGMA/W showed that the deformations associated with the wall A are the lowest.

Wall sections by Burgoyne and subsequent numerical analysis of that by Harkens et al (2000) also found that the most optimum shape for a gravity wall was that with a back batter.

The optimum shape is the wall with back batter and they can be constructed with inter locking modular block systems, Cribs and Gabion systems. But it is not possible to construct this back battered wall with masonry. Since the bearing pressures exerted are low this wall can be effectively used in areas underlain with soft soil.

The back batter will give an increased secure feeling to the users as compared to a vertical wall. The results of the study indicated that it would be necessary to have a back batter of only about 5^0 –10^0 to bring out a considerable improvement of safety.

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Introducing GeoPro Ver 1.0a for 3G Modelling.

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Abstract : The research primarily was based on application of principles of GIS (Geographical information system) technology in the field of Geotechnical engineering. Objectives being laid on that particular idea, the Gis-Team made plans step by step without heading towards a definite destination, until it was known at the end of a one-year period. With the project, the objectives of a large-scale professional implementation, the feasibility, and the foundation was concluded with an introduction to a new concept – *3G Modelling.*

Objectives Achieved

The Development of a GIS Tool – ***GeoPro*** Ver 1.0a for **3G Modelling** – With tools/Functions being,

- Geographical Processing
- Geological Processing and Layer modelling
- Geographical/Geological/Geotechnical information retrieval
- Contouring
- Profile Creation – Single/Multiple
- Outcropping – Geological Map
- Geotechnical Modelling
- Shallow Foundation Design - In point of Geotechnical Engineering

With features,

- Independent of the Area
- Independent of the amount of details – contour survey
- Could be used for instant decision making
- Instant Profiling

The *GeoPro* as an implemented project was found to be very tedious in arriving on decisions regarding modelling behind the 3G (Geographical/Geological/Geotechnical) entities. This approach to the final objective has been an experimented study rather than a commercial product, which could be marketed at national level.

The *GeoPro* with the current development is meant for providing variety of solutions for a vast range of implementations in the field of Geotechnical and Geological engineering, which has never been implemented before. The development is expected to be continued with being reviewed, and shall be kept hand in hand with Geographical information systems. The system is classified as a special tailor-made system for the implementation of 3G Modelling.

Background

Geographical information

The information locating the ground surface considered in relation to the investigation of Geology and Geotechnical Engineering of a certain area shall be the data of Geography. In the project Geographical information will be preliminary used to locate the data spatially with connecting the information source where the information of Geographical nature should be imported. Investigated data from a Contour surveys, specifically carried out by an EDM (Electronic Distance Measurements) Survey or digitised-data from *Admiralty charts* will be used here. The geographic information could be imported from a "3D file" created from such an automated contour survey with the help of EDM equipment software. The geographical data should be available in a simple formation such as " X-Coord / Y-Coord / Elevation" which is transferred for the purpose of ***Geographical Modelling.***

Geological information

The exploration of sub surface elements (Layers) will be the Geotechnical investigation, expressed in the context of the project. The key value towards the subsurface processing shall be the *"**Depth**"* to *bedding planes* of geological elements (Layers). From boreholes the depth values are obtained for different types of layers appearing underground. With geographical information of the boreholes taken, the elevation of each borehole is refined from the results of the "***Geographical modelling***" and depth values are altered and modified to the elevation information for each *Bedding Plane,* which are later used for the purpose of ***Geological Modelling.***

Geotechnical information

The Parameters for *soils* found in the geological elements (Layers), to uniquely identify each *soil type* are the Geotechnical information. (eg: Density, SPT N Value, Cohesion etc.) For each location where the borehole logging is carried out, these parameters are obtained and different databases are used for Geotechnical information maintenance. This information is separately transferred and correlated with the results of ***Geographical Modelling*** and ***Geological Modelling*** and used for the purpose of ***Geotechnical Modelling.***

Geographical information systems

The GIS shall be an evolving technology in the time beyond 21st Century, the dawning Era after *Information Technology.* The concept was derived and introduced with the development of IT (Information technology) and added a new dimension in the presentation of conventional 2-dimensional data - **Geography**. GIS adds the spatial dimension to the conventional representation of information, which once was 1-D or 2-D and illustrates the distribution of data spatially. (Applied only for Geographical related Data) From which the step to a great analysis was introduced with the data available spatially in the form of shapes/images combined/Linked with numerical data.

The GIS therefore is the technology of the Civil Engineer who shall be the front-end implementer of the system. Further GIS allows the analysis and realistic modelling of natural resources, from which the results are taken for decision-making.

- GIS is the tool to resource management
- Is the tool to resource allocation
- Is the tool for decision making
- Further will be used with parameters such as time and cost, for the optimal analysis

3G Modelling (The concept introduced)

- Geographical Modelling
- Geological Modelling
- Geotechnical Modelling

With interrelation back to back to form a complete integration

The processes are not found isolated in the background execution of the *GeoPro*, rather they are dependant each other to form a complete integration. This particular integration of the above types of modelling is introduced here as **3G Modelling.**

Information of Geographical nature, Geological nature, and Geotechnical nature is obtained from the DBMS system via statements of Standard Query Language.

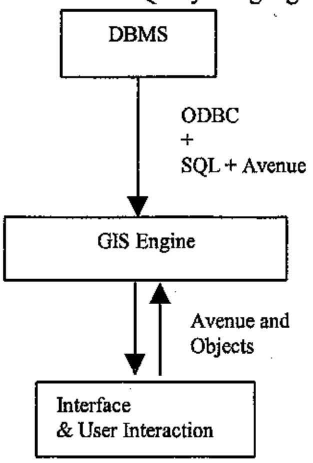

Figure: System model for GeoPro for 3G Modelling

In order to connect the Database to the GIS Engine the Open Database Connectivity is used.

Geographical Modelling

The information, used to locate the data geographically and for the creation of surfaces from point-formed data, is the data of geographic nature. The information is imported from the database source as point themes, which will be used in the process of surface creation in form of graphic algorithms producing the newly created formations of data representing Surfaces. This processing as a whole is called the **Geographical Modelling** in the context of *GeoPro*.

Geological Modelling

For the creation of geological elements (Geological layers), the Geological information (depth values) is obtained from the Database and are refined and processed with the available information from geographical modelling. There onwards based on the *GeoPro* assumption that if "Bedding Planes" of layers crosses each other it is an outcrop of below layer on top layer, layers are processed and stored. The creation of geological elements with the *depth* information from the database source and with the information retrieved from results of geographical modelling is called the **Geological Modelling**.

Geotechnical Modelling

Geotechnical data is coupled with available Geographical and Geological data and three-dimensional interpolation is carried out to model and arrange the data for the entire area concerned. When information is required, these raw-data of three types are referred back and forth, and are combined to obtain new information. eg: Q_{ult}. The process carried out to ensure the availability of information of Geotechnical nature at each geographical and geological (at each depth) location is called the **Geotechnical modelling.**

***GeoPro* – Technical Overview**

Object Models used for the development

Within the environment where the *GeoPro* was programmed, several object models were used for the programming of functions. Object models had been designed in order to support the object-oriented instructions and to apply the principles of object orientation in the programming. A sample object model is shown in the following figure.

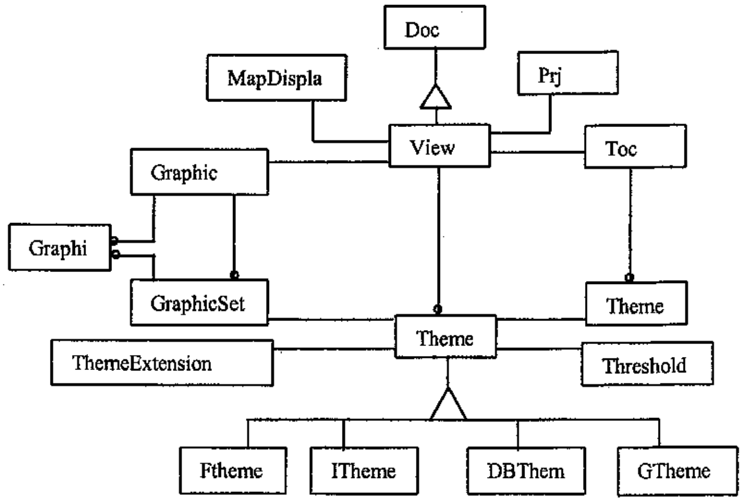

Figure: Object Model for View

Programming resources used

The ArcView desktop GIS Engine was used as the Development Environment, in which the *GeoPro* was programmed. This special platform was used with the addition of several libraries, which were integrated for the development, such as the spatial analyst – for operations related to surface modelling, and 3D Analyst – for operation related to 3D Modelling of surfaces.

Language used was *avenue*, which is an object oriented scripting language. Avenue is the programming language and development environment that's part of ArcView. Avenue is fully integrated with ArcView and the work will be run on any of the platforms for which ArcView is available. There are many uses for Avenue. ArcView provides the necessary customisation and language environment tools in an easy-to-use framework to work with Avenue. Among other resources, Database Management system functions and assistance, Standard query language, and open database connectivity were predominant.

Management system functions and assistance, Standard query language, and open database connectivity were predominant.

Theories behind the operation.

Applied theoretical background of Geotechnical Engineering.

- Application of Energy Method – Refinement of SPT N Values.
- Mayerhof Method of Foundation design (Shallow) – To obtain the Q_{ult}
- Energy method tables – Conversion from N'_{70} to ϕ Value.
- Terzagi and Peck Method – Settlement Calculation

Applied theory behind modelling.

- TIN – Triangular irregular network – Formation of triangles to form a 3d surface
- 3D Spline – Formation of a surface from a set of points with values set for parameters such as Regularized, Tension, Weight, No. of Points
- Object Orientation – Program paradigm applied for programming to integrate objects.
- DBMS – Database Management System – Platform for managing data with reliability
- ODBC – Open database connectivity – Connection via which the data is imported from the DBMS to GIS Platform.

Operation illustrated – (In a Nutshell)

Information gathering with DBMS – Microsoft Access 2000

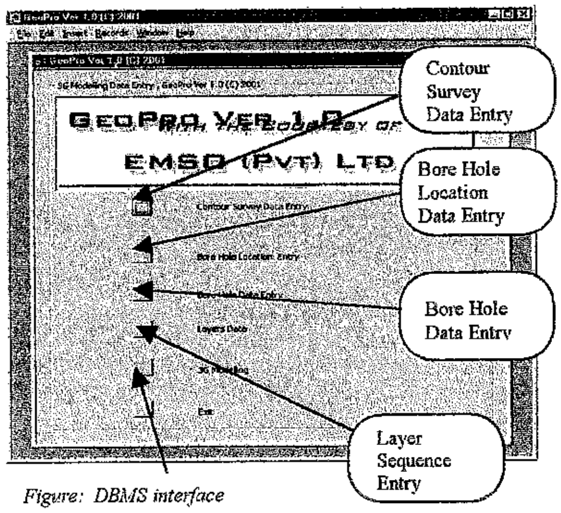

Figure: DBMS interface

Figure : Launching of 3G modelling through GeoPro Ver 1.0

GeoPro Modelling and Tools and Buttons

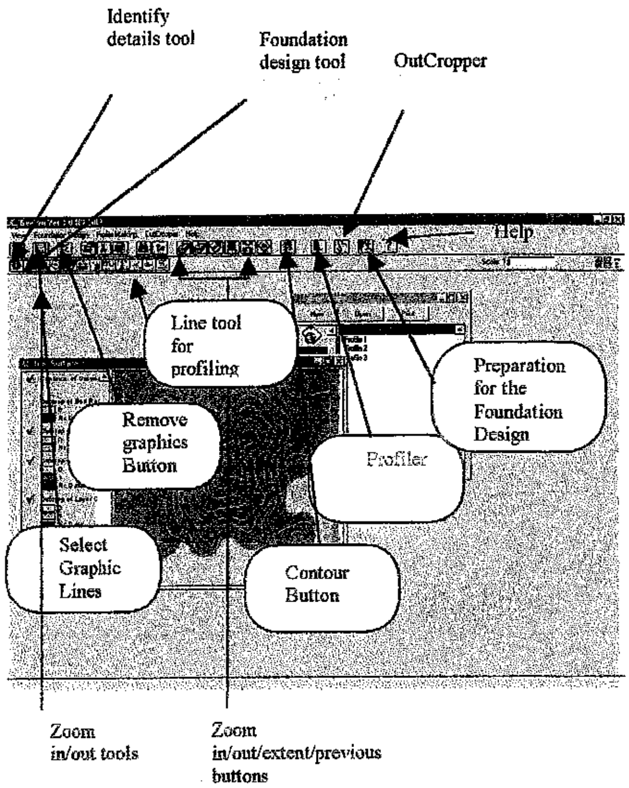

Figure: GIS Platform and interface

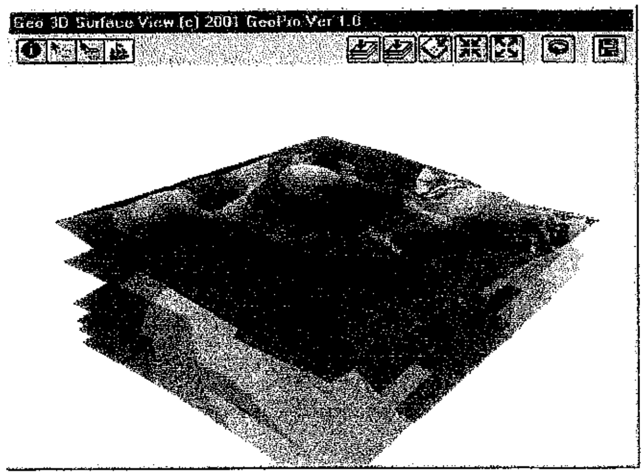

Figure: 3D View : 3-Dimensional View on Modelled Geological Elements with the manipulation capability.

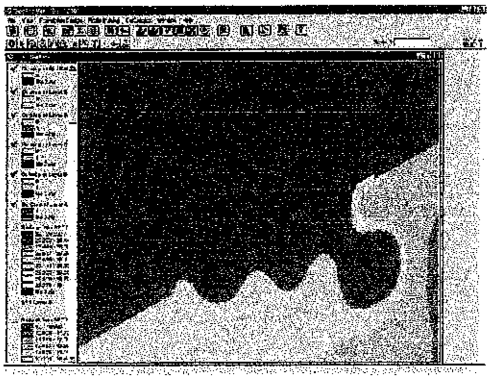

Figure: OutCropping – Geological Map - Activated.

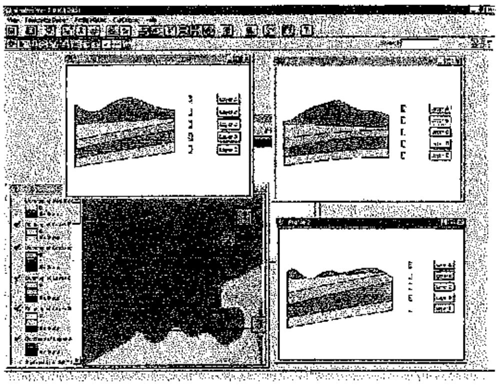

Figure: Profiler - Activated

Shortcomings

- The user has to be aware of the sequence of layering. (Planning to improve is undergone)
- Due to the assumption that if bedding planes cross each other, they have an outcrop on each other, the following situation could not be obtained.

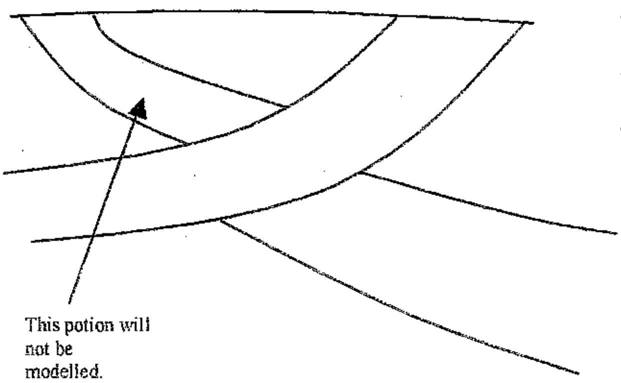

Figure: Impossible to Model

- If the sequence is not in order outcropping shall be an unpredictable situation.
- Require expensive extensions – Spatial Analysis, 3D Analysis
- Takes extensive time for processing.

Remedies and New Modifications as Next Step

- Plans are undertaken for extensive altering in order to facilitate the program for multiple project handling.
- A Conversion Program shall be under development in order to convert the ordinary borehole database to the specific format specified for the database used for the project.
- The GIS tools and libraries for the developer - *Map objects* should be purchased under funds from ESRI and undergo an independent development for ***GeoPro.***
- Chose Oracle as the DBMS system with the user interface for the borehole logging.
- Adapt the design of deep foundation, and analysis of failure surfaces for the *GeoPro.*
- Call the new development as ***GeoStatPro Ver 1.1***

Acknowledgment

We would like to offer our deepest gratitude to Dr. Saman Thilakasiri (Supervisor- GIS Team) for being very cooperative from the time we first requested for the project.

Secondly we offer our warm appreciation to our sponsors, EMSO (Pvt) Ltd, authorised dealers of world-renowned software company for GIS products, ESRI.

We shall be very much thankful to Mr. A. J. Wijesekara – Deputy Chief Hydrographic Surveyor of SLPA for introducing us to personnel at EMSO with establishing steady relations.

We would warmly thank Mr Sarathchandra – Superintendent Engineer of SLPA for the support and the participation in achieving our goals.

Also we should be very grateful to all the Laboratory officers and personnel for providing the assistance in organising the resources for the final presentation to make things wonderful and letting our project have a future.

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A Geotechnical Analysis of a Shallow Foundation System using a Spreadsheet

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Abstract

This paper describes a spreadsheet-based tool to perform the geotechnical design for a pad and a strip footing. The analysis considers a compressible layer identified as the M-I sub series (3), which consists of a peat layer overlain by a fill. The analysis considers a fill of 1-3m thickness placed two years prior to construction. The peat is considered to vary between 1-7m. The selected structural loads correspond to dwellings of one to three stories. The stress increase due to the structural load on the pad footing is estimated using the Dispersion method. The Strip Load method is used for strip footings. The dimensions of the footings are obtained for limiting settlements of 25, 50, 75 and 100 mm. The analysis determines the required fill thickness.

The analysis shows that a minimum fill thickness of 1.0, 1.25, 1.45 m for pad footings and 1.15, 2.05, 2.75 m for strip footings, for the respective loads, ensures safety against bearing failure of peat. The estimated total settlement is presented graphically.

Introduction

Rapid development that has taken place in and around Colombo during the past few decades has resulted in a shortage of space for development. This has resulted in developers making use of less suitable land for construction. These lands lie within the coastal peneplane, some being below mean sea level. These marshy areas where organic decomposition has taken place over time contains mainly peat. They are highly permeable and also have a high content of water, which makes them highly compressible and easily drainable. Peats undergo high settlements when loaded and also possess very low bearing capacities, making them a poor bearing material. Structures designed for shallow foundation systems in such soils require improvements to their engineering properties.

When housing units of relatively low structural loads are constructed over compressible layers, the foundations of such structures are designed to transfer the structural loads via an engineered fill. These fills carry a high bearing stress near the founding level, and are well compacted to have adequate allowable bearing capacities and to limit settlement. This settlement is insignificant compared to the settlement of the compressible layer. Raising the ground level also minimises possible drainage and flooding problems.

Primary and secondary consolidation takes place when the compressible layer is subject to a structural load. Peats show a high degree of consolidation due to high permeability. This makes primary consolidation to take place faster than in other compressible soil types. However, this may continue even after construction is completed. It may be insignificant compared to the settlement that has taken place during the first few years.

The total design settlement of a foundation can be expressed as:

$$S = (S_i + S_c + S_s)_{struc.load} + (S_c + S_s)_{fill\ load} \qquad (1)$$

where S_i is the initial settlement, S_c the primary consolidation settlement and S_s the secondary settlement.

Distribution of Peat in and around Colombo

The low-lying areas in and around Colombo consist predominantly of highly compressible peat. Senanayake (1986) describes a study that classifies peat based on structural characteristics according to McFarlane (1969) and are classified as coarse fibrous, fine fibrous and amorphous granular.

Peat deposits North of Colombo, characterised as the Muthurajawela Series (M-Series) is based on the stratigraphy of the deposits (Reference 4). Senanayake (1986) subdivides this 'M-Series' to five sub series (I through V). This classification is based on the existence of peat horizons and how these layers occur within deposited inorganic layers.

This study models sub-surface conditions for M-I sub-series. Senanayake (1986) describes this sub series as having two distinct horizons, the upper (Horizon 1) referred to as peat and the lower (Horizon 2) referred to as inorganic soil. The depth of peat horizon is found to vary from a minimum thickness of 1.0m to a maximum of 10.7m as observed in the Yakbedda and Battaramulla Zones repsectively. The lower horizon exists up to rock which extends to a depth of 10m to 15m.

The physical and engineering properties of Colombo peats as given in Senanayake (1986) and Senanayake et al (1986) are summarised as follows:

1. The specific gravity of peat = 1.69-2.29.
2. Properties of fine fibrous peat: moisture content = 92.8%; the unconfined compressive strength when not surcharged = 4-10 kN/m^2, when subject to loads due to fill material = 10-32 kN/m^2; the unit weight: =1.05-1.50 Mg/m^3; the secondary compression index C_α = 0.0178.
3. Properties of amorphous-granular peat: moisture content = 70-150% and 145-382%; the unconfined compressive strength = 14-47kN/m^2; the unit weight = 1.36-1.61 Mg/m^3; the secondary compression index C_α = 0.0056.

Table 1 summarises the engineering properties of Colombo peats.

Stress Distribution

The most commonly used methods to determine the stress distribution with depth for Pad footings are based on Dispersion Method (M1) and Newmark's Method (M2), and for Strip footings Strip Loading Method (M3).

In this analysis the imposed load is due to the weight of fill and the structure. The effective thickness of the peat layer corresponds to a depth where the stress equals 5% of the stress due to imposed load, as highly compressible soils such as peat undergo settlements due to much smaller stresses. Figure 1 shows how the stress due to an imposed load varies with depth for the Dispersion method and Newmark's method considered for the pad footing and the Strip method for strip footings in this study. The incremental stress with depth z is denoted as q_i, the imposed stress q, and the width of the footing as B. It indicates where the stresses corresponding to 5% of the imposed load take place. This shows that the effect due to the imposed load ceases to affect the thickness of the compressible layer below this stress value.

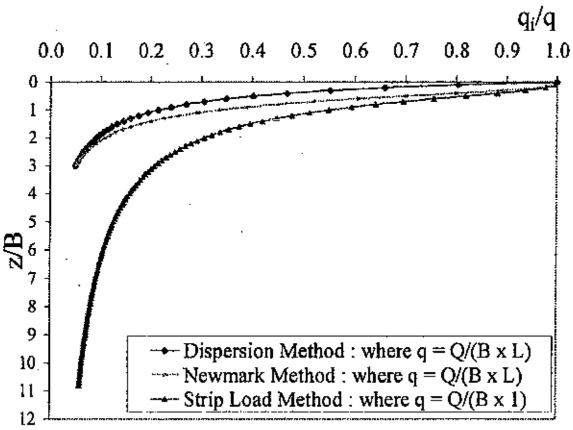

Figure 1: Variation of Normalised Depth versus Normalised Incremental Stress.

Settlement

One Dimensional Consolidation Theory is used to estimate the primary consolidation settlement of peat. It is assumed that drainage takes place in both directions and the length of drainage path, d is half the thickness of peat layer.

This analysis computes the secondary compression assuming that it takes place after primary consolidation is completed. The remaining settlement S_f, after 2 years is computed as

$$S_f = \left(\frac{100-U}{100}\right)S_C + S_S \qquad (2)$$

where U is the degree of consolidation.

The secondary compression due to the fill is computed for the duration for which the fill is placed. This is taken as twenty-seven years (2 plus 25 years).

The primary consolidation was computed by dividing the peat layer in to sub-layers of thickness 100mm. The analysis considers the thickness of the peat layer to vary between 1-7m at 1m increments. The water table is taken to be at the surface of the compressible layer. The settlement is computed using a compression ratio ($C_c / 1+e_o$) of 0.1. The analysis considers fill heights varying between 1-3m at 0.5m increments. The effective thickness of the compressible layer was taken as the lesser of the actual thickness or the thickness corresponding to a stress level of 5% of imposed stress. The primary consolidation is computed based on the Dispersion method considering a dispersion angle of 30°. The primary consolidation due to the fill is considered to have taken place two years before construction has commenced. The degree of consolidation during this period is computed using a coefficient of consolidation C_v = 1.58 m^2/year. This settlement is deducted from the primary settlement due to the fill that occurs during the design life span of the structure.

The secondary consolidation S_s due to the fill is computed. The analysis uses a secondary compression index C_α = 0.0178 which corresponds to fine fibrous peat (Senanayake, Ray and Ganeshamoorthy, 1986). The settlement is computed using equation 2 for a period of 27 years assuming a design life of 25 years and construction commenced after 2 years since filling.

The total settlement of the structure is computed using equation 1.

Differential Settlement

Differential settlement is a function of the uniformity of the soil, stiffness of the structure, stiffness of the soil, and distribution of loads within the structure. Limitations to differential settlement depend on the application. The maximum allowable average settlement of some structures is given in Table 2 (Reference 6). The value estimated from the analysis has to be compared with this permissible value.

Bearing Capacities of the different Soil Layers

The Ultimate Bearing Capacity for a rectangular footing is computed using Terzhagi's Bearing Capacity equation subsequently modified by Schultz.

a) These were based on $\phi' = 30°$ and $c'=10$ *kPa*. For a strip footing the same equation was used considering a unit length, $L=1m$. The Ultimate Bearing Capacity at the founding level was compared with the stress due to structural loads considering a factor of safety of *3*.

b) The actual stress at the top of the peat layer was checked against bearing capacity failure. These were based on $\phi' = 0°$ and $c'=7.5$ *kPa*. For a rectangular footing the equivalent dimensions of $B' = B+2z\tan\theta$, $L' = L+2z\tan\theta$ and for a strip footing $B' = B+2z\tan\theta$, $L = 1m$ was used. A factor of safety of two is used in this analysis.

Results

Figures 2 to 7 give the variation of settlement with fill thickness for different thickness of peat corresponding to bearing pressures of 25, 50 and 75 kPa. These are computed for effective thickness of peat layer based on 5% foundation stress.

Dispersion Method of Analysis

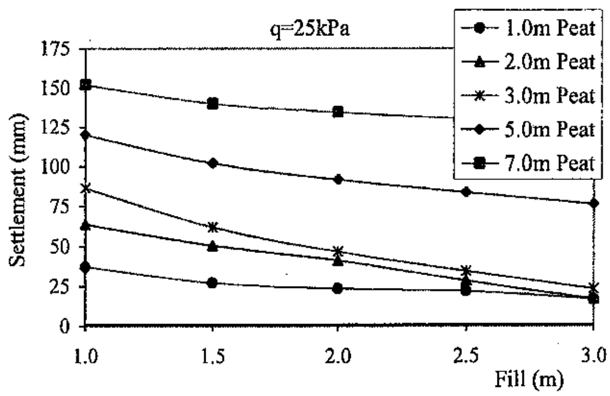

Figure 2: M1 Variation of Settlement versus Fill Thickness.

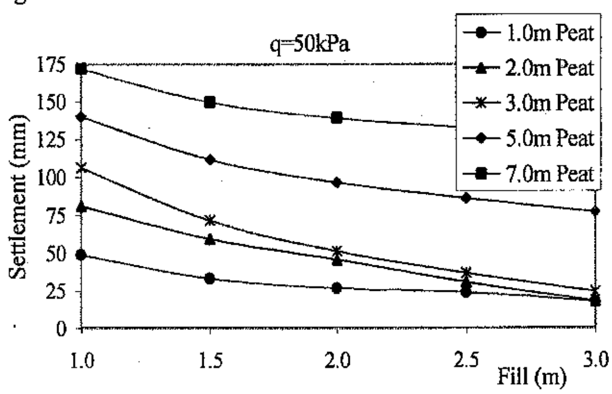

Figure 3: M1 Variation of Settlement versus Fill Thickness.

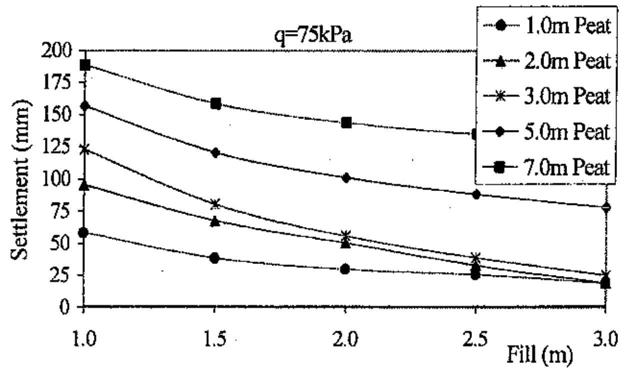

Figure 4: M1 Variation of Settlement versus Fill Thickness.

Strip Load Method of Analysis

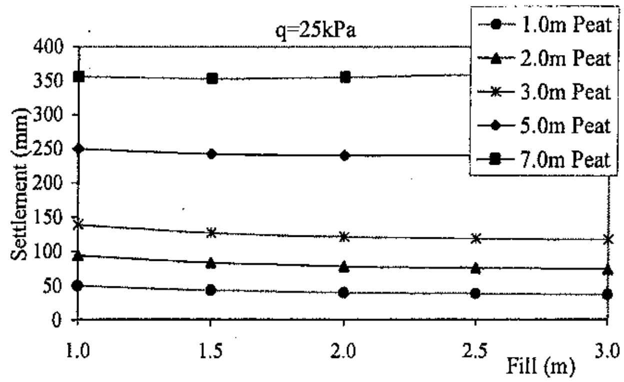

Figure 5: M3 Variation of Settlement versus Fill Thickness.

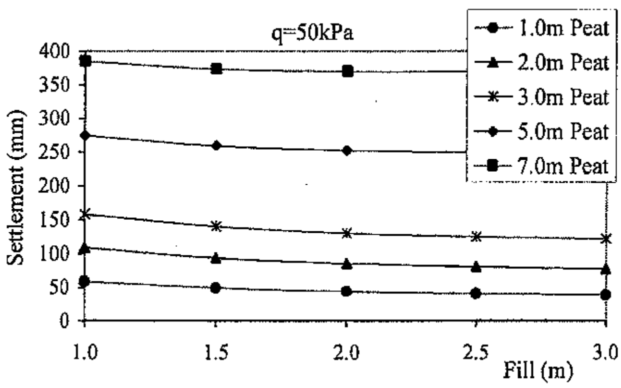

Figure 6: M3 Variation of Settlement versus Fill Thickness.

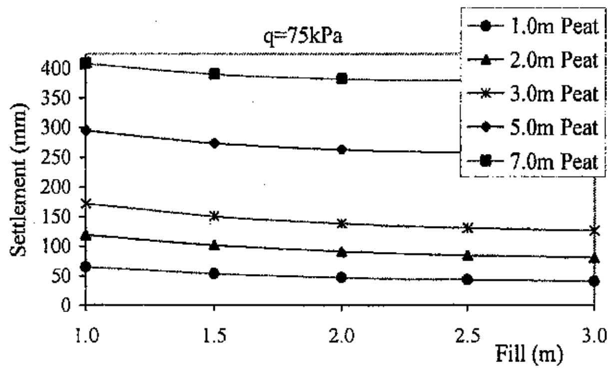

Figure 7: M3 Variation of Settlement versus Fill Thickness.

Allowable Bearing Capacity based on Ultimate Shear Failure

The top layer of peat is checked for safety against bearing failure and a factor of safety of 2.0 is used. Table 3 indicates the limiting fill thickness that is required to satisfy this criterion.

Discussion of Results

Dispersion Method

The analysis checks the allowable bearing capacity at the top of the peat layer to prevent bearing failure taking place. This is ensured by having a minimum fill thickness of 1.0, 1.25 and 1.45 m for the respective loads of 25, 50 and 75 kPa.

Tables 4 – 6 indicate the limiting fill thicknesses that are required to limit settlements to ranges of 0-25, 25-50, 50-75, 75-100 mm.

Strip Load Method

The analysis checks the allowable bearing capacity at the top of the peat layer to prevent bearing failure taking place. This is ensured by having a minimum fill thickness of 1.15, 2.05 and 2.75 m for the respective loads of 25, 50 and 75 kPa.

Tables 7 – 9 indicate the limiting fill thickness (m) that is required to satisfy the settlement ranges of 0-25, 25-50, 50-75, 75-100 mm.

Comparison of Settlements

Senanayake (1986) has identified the compression index, $C_c/(1+e_0)$ to be 0.1 and the results obtained are compared. Table 10 shows a comparison of values obtained for the two methods with typical values given in Senanayake (1986). This is for the range of minimum to maximum settlements arising out of the conditions for the analysis where the combination used is:

For minimum settlements: Maximum thickness of fill (3m) and minimum thickness of peat (2m).

For maximum settlements: Minimum thickness of fill (2m) and maximum thickness of peat (3m).

Conclusion

The spreadsheet programme is designed to accommodate design soil parameters and foundation details in the design of a shallow foundation. It can also be used to determine the required fill thickness for a single layered soil, for a particular loading arrangement.

It is recommended that for highly compressible layers occurring near ground surface the settlements be computed for an effective thickness based on the 5% stress level, as this is a safer option.

It is recommended that a lower value of dispersion angle θ be used for weaker soils, between 30-45° depending on the frictional characteristics of soils (Bowles, 1984). However, a uniform dispersion can be used when vertical stress of several layers is computed.

The total settlements obtained from the above methods are presented in charts (Refer: Figures 2 through 7). The total settlement based on bearing capacity is presented in Tables 4 through 9. These can be used to determine the required fill thickness for the specific load and soil profile encountered.

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Investigation.
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4 ___ (1982), Project report on *Geotechnical Engineering Problems Associated with Development of Low-Lying Areas* by the Central Soil Testing Laboratory - Urban Development Authority
5 Bowles, J.E. 1985. Foundation Analysis and Design (5th Edition)
6 Settlement Analysis (1994) – Technical Engineering and Design Guides as adapted from the US Army Corps of Engineers, No.9 – ASCE.

Table 1: Engineering Properties of Colombo Peats

Water content, w_n (%)	125 - 405
Initial void ratio, e_0	3.4 - 7.7
Unconfined Compressive strength, q_u (kg/cm^2)	0.05-0.19
Liquid Limit, LL (%)	63 - 166
Plasticity Index, PI (%)	24 - 105
Dry density, ρ_d (g/cm^3)	0.23- 0.58
Bulk density, ρ_b (g/cm^3)	1.12 -1.34

Table 2: Maximum Allowable Average Settlement (Settlement Analysis – USACE Design Guide)

Type of Structure	Settlement (mm)
Plain brick walls (length/height ≥ 2.5)	75
Plain brick walls (length/height <= 1.5)	100
Framed Structures	100
Reinforced brick walls and brick walls with reinforced concrete	150

Table 3: Limiting Fill thickness for the different methods

Load Cases	Dispersion	Strip Load
25kPa	≥1.0m	≥1.15m
50kPa	≥1.25m	≥2.05m
75kPa	≥1.45m	≥2.75m

Table 4: M1 Limiting fill thickness for q =25 kPa

Peat thickness (m)	Limiting Settlement (mm)			
	0–25	25–50	50–75	75–100
1	≥ 2.0	≥ 1.0	⊗	⊗
2	≥ 3.0	≥ 2.0	≥ 1.0	⊗
3	≥ 3.0	≥ 2.0	1-2	≥ 1.0
4	⊕	≥ 3.0	≥ 2.0	≥ 1.5
5	⊕	⊕	⊕	≥ 2.0

Note: a minimum fill thickness of *1.0* m is required to prevent bearing capacity failure in peat.

Table 5: M1 Limiting fill thickness for q =50 kPa

Peat thickness (m)	Limiting Settlement (mm)			
	0–25	25–50	50–75	75–100
1	≥ 2.0	≥ 1.25	⊗	⊗
2	≥ 3.0	≥ 2.0	≥ 1.5	≥ 1.25
3	≥ 3.0	≥ 2.5	≥ 2.0	≥ 1.5
4	⊕	≥ 3.0	≥ 2.5	≥ 1.5
5	⊕	⊕	⊕	≥ 2.0

Note: a minimum fill thickness of *1.25* m is required to prevent bearing capacity failure in peat.

Table 6: M1 Limiting fill thickness for q =75 kPa

Peat thickness (m)	Limiting Settlement (mm)			
	0–25	25–50	50–75	75–100
1	≥ 3.0	≥ 1.5	≥ 1.45	⊗
2	≥ 3.0	≥ 2.5	≥ 1.5	≥ 1.45
3	⊕	≥ 2.5	≥ 2.0	≥ 1.5
4	⊕	≥ 3.0	≥ 2.5	≥ 1.5
5	⊕	⊕	⊕	≥ 2.5

Note: a minimum fill thickness of *1.45* m is required to prevent bearing capacity failure in peat.

Table 7: M3 Limiting fill thickness for q =25 kPa

Peat thickness (m)	Limiting Settlement (mm)			
	0–25	25–50	50–75	75–100
1	⊕	≥ 1.5	≥ 1.15	⊗
2	⊕	⊕	≥ 3.0	≥ 1.15

Note: a minimum fill thickness of *1.15* m is required to prevent bearing capacity failure in peat.

Table 8: M3 Limiting fill thickness for q =50 kPa

Peat thickness (m)	Limiting Settlement (mm)			
	0–25	25–50	50–75	75–100
1	⊕	≥ 2.05	≥ 2.05	⊗
2	⊕	⊕	⊕	≥ 2.05

Note: a minimum fill thickness of *2.05* m is required to prevent bearing capacity failure in peat.

Table 9: M3 Limiting fill thickness for q =75 kPa

Peat thickness (m)	Limiting Settlement (mm)			
	0–25	25–50	50–75	75–100
1	⊕	≥ 2.75	≥ 2.75	⊗
2	⊕	⊕	⊕	≥ 2.75

Note: a minimum fill thickness of *2.75* m is required to prevent bearing capacity failure in peat.

⊗ Considers the depth of footing D_f= 0.8m and the minimum thickness of fill required to satisfy the allowable bearing capacity of peat.

⊕ The analysis considers a maximum fill thickness of 3m, the value exceeds this limit.

Table 10: Comparison of Settlements (mm)

Type of foundation	Senanayake (1986)	Dispersion Method	Strip Load Method
Strip:B=0.75m, Q=33kPa	260 - 301		370 - 441
Strip:B=1m, Q=25kPa	259 - 300		369 - 438
Strip:B=1.5m, Q=43kPa	267 - 320		372 - 444
Strip:B=2m, Q=58kPa	277 - 340		374 - 450
Strip:B=1.5m, Q=77kPa	279 - 346		377 - 456
Pad: 3x3m; Q=33kPa	331 - 425	265 - 317	
Pad: 3.5x3.5m, Q=31kPa	328 - 430	265 - 317	

*Note: This shows the range for the minimum to maximum settlement.

Subsurface Investigation using Electrical Resistivity Method at Engineering Faculty Premises

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ABSTRACT

The geophysical methods are quick and inexpensive technique to identify the subsurface layers and to determine the foundation conditions at different depths. The objective of this study is to plot the vertical profile of different subsurface layers and the contour of bedrock for the given area using one geophysical method. The vertical subsurface layer depths were determined by considering the property of electrical resistivity using electrical resistivity meter. First of all the site plan was drawn by doing traverse survey. Then by doing the trial tests, the grid point locations of the instrument stations were determined. Thereafter the resistivity survey was done for each grid point. Finally the sub surface layer thickness was determined by manual interpretation technique (curve matching method) and the contour of sound bedrock was plotted on the site map. This result can be verified using borehole data. Due to the time restriction of the project, no further investigation were done to verify the results of the electrical resitivity survey.

INTRODUCTION

Geophysical exploration is an important activity in most of the engineering projects like mining, oil industry, geo-technical and transportation Engineering. Particularly, in Civil Engineering, it plays a vital role. Generally the information that can be obtained from geophysical methods are as follows: porosity, degree of rock fracture, boundaries between different litho logical type, locations of faults, depth to bed rock, saturated layers etc. This method is very useful for preliminary study of very large area with reasonable time and money consumption.

The principle of geophysical exploration is to measure the variation of physical property of the subsurface material. There are different geophysical methods corresponding to physical properties. Seismic method and Electrical Resistivity Method are commonly used in civil engineering works. This report is limited to the application of electrical resistivity method only.

Electrical resistivity soundings are mainly employed for ground water exploration and foundation engineering works. The accuracy of the result depends on the geological site knowledge and experience. Since the electrical resistivity of a soil or rock is controlled primarily by the pore water conditions, there are wide ranges in resistivity for any particular soil or rock type, and resistivity values cannot be directly interpreted in terms of soil type or lithology. The measurement of resitivity involves injecting a certain amount of electricity into the ground through a pair of electrodes and measuring the voltage produced across another two electrodes. Electrical resitivity survey can be carried out using equipment consisting of:

1. Electrical Resistivity meter
2. Dry lead acid battery (12V).
3. Four copper electrodes and 4 numbers of well-insulated cable rolls.

The theory and field methods used for resistivity survey are based on the use of direct current because it allows greater depth of investigation than A.C current. So in this survey, direct current was used to do the survey. (Griffith.D.H & King R.F, 1969 and Bell F.G, 1980)

Objective

The objective of this study is to draw the vertical profile of different subsurface layers and the contour of bedrock for the selected area. Due to the time restriction, the survey was limited only at the north end of the faculty premises.

METHODOLOGY

Field procedure

First Traverse and chain survey was done to plot the proposed area. Then Grid dimension was decided by doing the trial testing and Grid points were located with the aid of compass, poles, tape etc. Finally resistivity Survey was done.

Resistivity survey

Resistivity survey (Vertical Electrical Sounding, The VES) was done for every grid points. For the resistivity survey, the schlumberger array, which consists of 4 electrodes in a line, was used. In this method of field operation, the potential electrode spacing (inner spacing) remains fixed while current electrodes (outer) were adjusted to vary the distance AB/2 (Fig 1). The inner spacing was adjusted, when decreasing sensitivity of measurement necessitates it. The reading for a grid point was taken for different MN/2 and AB/2 as shown in the (table1). (TORINO Italy, Resistivity Meter Manual, E3DIGIT)

Table 1. Electrode spacing used for the survey

READING	MN/2(m)	AB/2/(m)
1	.5	2
2	.5	4
3	.5	8
4	1.0	8
5	1.0	12
6	1.0	16
7	2.0	16
8	2.0	24
9	2.0	32
10	5.0	32
11	5.0	45

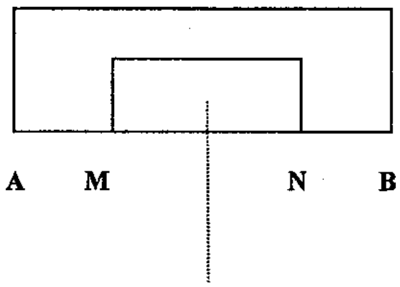

Fig 1. Schlumberger array

METHOD OF ANALYSIS AND RESULTS

Vertical Electrical Sounding (VES)

The interpretation of VES data is to use the curve of apparent resistivity Vs electrode spacing plotted from field measurements, to obtain the parameter of geological section, such as the layer resistivity and thickness. In order to get better results, it is necessary to have a good geological background of the area. (Geological maps and plans).

There are three popular methods to interpret the VES data. They are:

- Moore cumulative method
- Curve matching method
- Using computer technique

For the analysis Curve matching method was used.

Curve matching method

The Curve matching method is widely used to interpret field data of this survey. In this technique, the field-sounding curve is compared with theoretical sounding curves computed for various layer parameters. When the field curve matches with a particular theoretical curve, the layer parameters of the theoretical curve are taken as the solution for that particular field-sounding curve. From this result, the subsurface layer profiles and bedrock contour were drawn. (Bell, F.G, 1980)

RESULTS

Table 2 shows the thickness and resistivity values of subsurface layers at the grid points.

Table 2. Variation of Resistivity with Depth

Grid Point	h1 /m	h2 /m	ρ_1 /Ωm	ρ_2 /Ωm	ρ_3 /Ωm
A4	1.4	21	120	240	4400
B4	2.8	9.8	100	200	2525
A5	2.0	12	40	280	1330
B5	1.3	7.1	60	150	2000
A6	1.7	12.7	75	375	1120
B6	1.3	15.0	90	113	550
A7	1.8	20.6	70	350	6300
B7	1.3	15.6	120	150	6000
A8	1.9	13.3	120	240	1056
B8	1.5	7.5	65	162	1400
A9	1.5	12.0	290	363	6800
B9	1.6	11.2	70	175	1500

hi-Thickness of i^{th} layer
ρi- Resistivity of i^{th} layer

General

Using resistivity method, different subsurface layer depths were determined. The sub-surface layer profile (Fig 3 & Fig 4) and bedrock contour (Fig 2) were plotted using this result. It is seen from the results that the first layer thickness is in the range of 1.3 to 2.8 m and the resistivity is in the range of 40 to 120 Ωm. Therefore it may be clayey sand, sandy clay, gravely clay, and saturated sand. The second layer thickness is in the range of 10 to 20 m excluding some points and the resistivity is in the range of 100 to 400 Ωm. So it may be harder materials probably highly weathered rock and the third layer resistivity is above 1000 Ωm. Therefore we can conclude it as hard rock. But at some places there may be fractures in the rock. Furthermore, finding the water table level by considering the results was very difficult. Normally, the water table interpretation can't be done accurately in fine-grained soils.

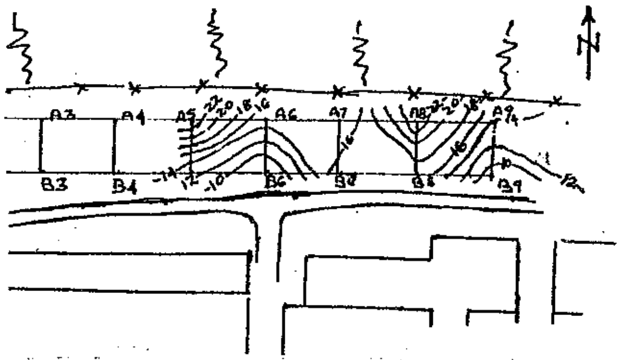

Fig 2. Site map and Bedrock contour

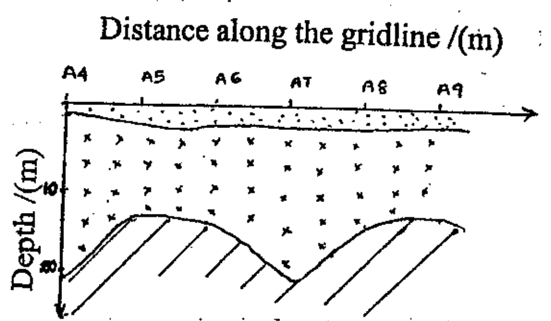

Fig 3. Vertical profile of different subsurface layers along gridline A

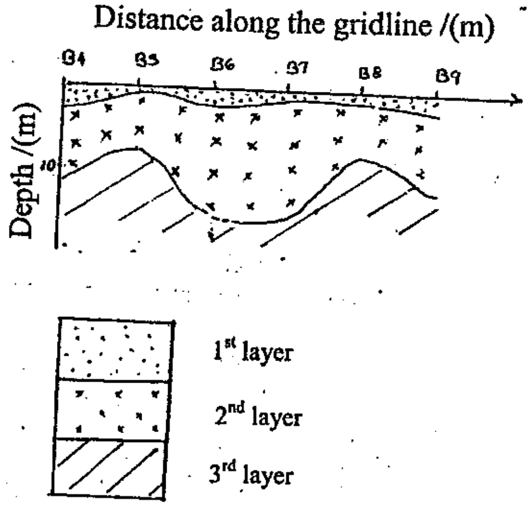

Fig 4. Vertical profile of different subsurface layers along gridline B

It is more important to correlate the resistivity survey results with borehole data to get more accurate information. But the borehole data for the area was not available and thus the accuracy of the interpretation is limited .The preliminary interpretation can be done without borehole data and this will help a more judicious sitting of boreholes and reduce the number of boreholes required.

There are several limitations and possibilities of errors in this resistivity method. Although some practical difficulties were encountered during the fieldwork and analysis, every attempt was made to avoid them.

CONCLUSION

From the analysis, three different types of layers were determined. The first layer was considered as sandy clay, clayey sand, gravelly clay and saturated sand and the second layer was considered as a harder with weathered rock materials. Third layer was found to be bedrock. It is seen from the profiles that the bedrock has a dip nearly in the direction of north to south.

To improve the accuracy of the results, the following suggestions can be made for the future works.

* Use the computer package to interpret the data quickly and accurately.
* In order to get better results, use borehole data for interpretation.
* Try to avoid the possibilities of errors, because small deviation in V and I values cause much error in the results.
* It is not advisable to do the fieldwork during the rainy days.

Finally, it can be concluded that if those suggestions are made, results will be more accurate.

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SOIL NAILS AND THEIR PULL OUT RESISTANCE

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ABSTRACT

The bar force distribution along nail length at different nail displacement of grouted and ungrouted nail of 2 m length were investigated by using compacted clayey sand of 2302 Kg/m^3 bulk density, model slope with surcharge load of 5.67 KN/m^2.Also pullout capacity of the nails were tested in the same model.

Bar force distribution pattern of the test have the closest agreement with the test done by Seto et.al (1985). The maximum pullout load was achieved when the nail displacements reached 7.86mm. Failure also has been mobilized when nail displacement was approximately 7mm.

1. INTRODUCTION

Soil nailing is an in-situ reinforcement technique, which has been used for variety of stability problems. Soil nailing is being used at present to stabilize natural slopes, cuts or excavations, walls in stiff clays, granular soils and soft rocks. The purpose of this technique is essentially to limit decompressions and the opening of pre-existing discontinuities by restraining the deformations. Soil nails often consist of steel bars grouted or occasionally driven; sub–horzontally across potential slip plans to form integral tensile members within the soil mass the stability of slopes using soil nail principle differs considerably from the ground anchoring approach. Ground anchorages are founded out side any potential slip plane or slip circle and are prestressed against a major structure to distribute the soil retaining force at the slope face. Soil nails are installed at a much higher density and generally remain passive. They remain unstressed until nominal soil movement mobilizes the tensile capacity of the soil reinforcement. They are usually 20-30 mm in diameter that are inserted in to the soil either by simple driving or by grouting in pre drilled hole which are usually 90-120 mm diameter.

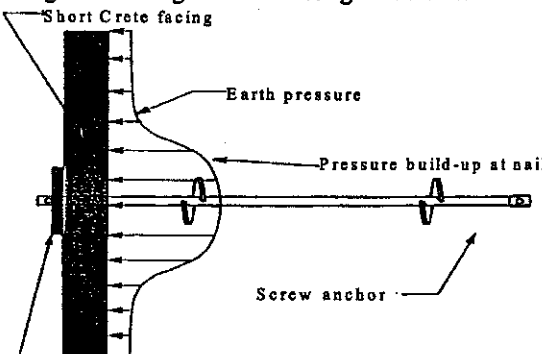
The use of soil nails has a high potential specially in hilly regions like Kandy as a means of stabilizing slopes. Still retaining walls are the basic and popular solution to slope stability problems in this country. This has been due to variety of reasons such as lack of design methods, lack of knowledge & experience on soil nailing techniques under local soil conditions, cost etc. Even though this method is widely used in developed countries with the use of modern equipments. This can be used locally with available equipments and design methods provided. There are prototype test verifications. Literature on soil nailing revise that there are number of methods to analyze the problem under certain assumptions. However as it is the soil that should ultimately give required bonding the system to work, pullout test are essential to confidently predict the anchorage capacity. Therefore this project is aimed to study the anchorage capacity or the pullout resistance of soil nails experimentally using model tests. The experimental findings are expected to give some guidelines in soil nail design.

2. BACKGROUND AND OBJECTIVES

Main objective of the project is to study the behavior of soil nails experimentally in particular the pullout resistance using model tests. The bar force distribution within a nail itself under full-length pullout is to be measured under different confining stresses. Then the experimental results are to be coupled with available design methods to propose suitable design guidelines under local test conditions.

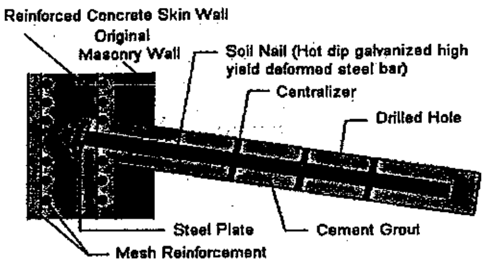
3. LITERATURE SURVEY

There are two different types of soil nails available, screw anchor soil nails and grouted soil nails .Screw anchor soil nails are screw anchors, which consists of 1.5-inch square solid steel shafts on which steel bearing plates or helices are welded at regular intervals. These screw into the soil and obtain their bond with soil through the bearing of the helices against the soil.



Fig(a)

Grouted soil nails are typically consists of 0.25 inch to 1.25-inch diameter deformed steel bar that is placed in a drill hole and grouted in place. The grouted soil nails typically has a minimum diameter of 4 inch. Centralizes are placed around the soil nail to maintain an even thickness of grout around the bar. For permanent applications, nails may be epoxy-coated or provided with a protective sheath for corrosion protection.



Typical Soil Nail Detail

. Fig(b)

4. METHODOLOGY

Apparatus Arrangement

Calibration of Load cell

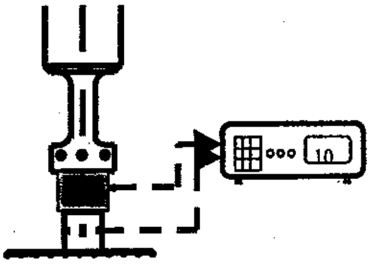


Fig (c) calibration of load cell arrangement

As shown in the figure applying force through the hydraulic jack (range 0-50 kN) was measured using the calibrated load cell (range) with the use of Data logger and strain in load cell also measured by using Data logger. While load was applying, Visual Data logger Software was automatically plotting calibration curve (Load Vs Strain).

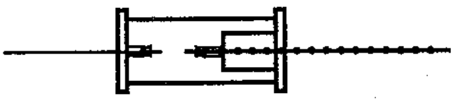


Fig (d) Arrangement for changing tension force to compression force.

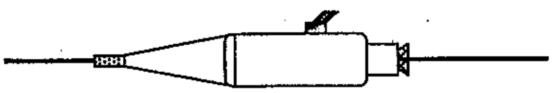


Fig (e) Tension jack arrangement

Axial Force Measuring Arrangement

Four strain gauges were instrumented in each tendon with the equal interval along the tendon. Each strain gauge was separately connected to the Data logger.

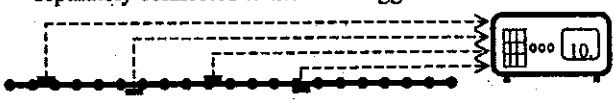


Fig (e) strain gauges along the tendons

Nail displacement measuring arrangement

Nail displacement was measured by displacement gauge, which was connected to electrical analogger. The displacement was limited to 8 mm.

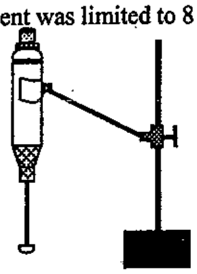


Fig (g)

Tank arrangement

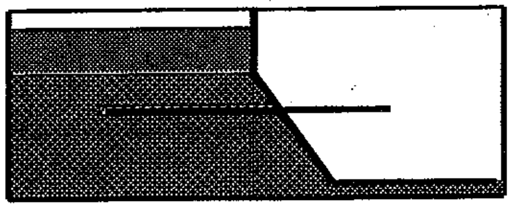


Fig (h) Longitudinal Section of the Model

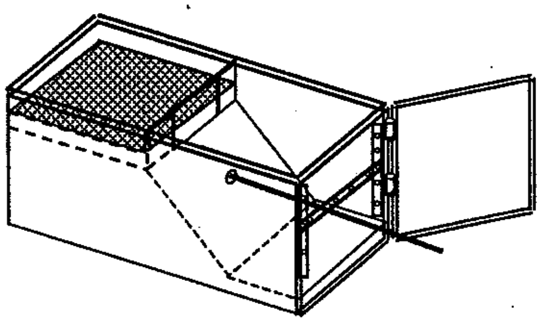


Fig (i) Experimental Model

Outline of model tests
Loading system

Load was applied by a layer of fine sand Bulk density 1.446 g/cm^3) as shown in Fig.(g). Volume of the loading layer is 2mx1.8mx0.5m.

Instrumentation

The model was instrumented to measure the following items during the pullout testing operation (Fig. (h).).

a) Pullout force was measured by the load cell installed at the end of the tendon, which was connected to an Electrical Analogue.
b) Pull out force was applied through a tensioning jack placed at cross bar
c) Axial strain in tendon and
d) Nail displacement.

Fill material

The fill material used was classified as clayey sand. Some physical and mechanical test results are presented in table. (a).

SOIL PARAMETERS	VALUE
Optimum moisture content /(%)	16
Maximum dry density /(kg/m³)	1.804
Cohesion /(kN/m²)	41.11
Friction angle /(deg)	32
Bulk density /(kg/m³)	2302.72

Table (a)

Test Nail

The test nail was manufactured from 12mm deform bar, 2m long and grouted in 50mm pre-drilled hole. Further more, it was instrumented with four strain gauges so that strain development (by the axial force) along the bar could be measured as shown in Fig.(f)

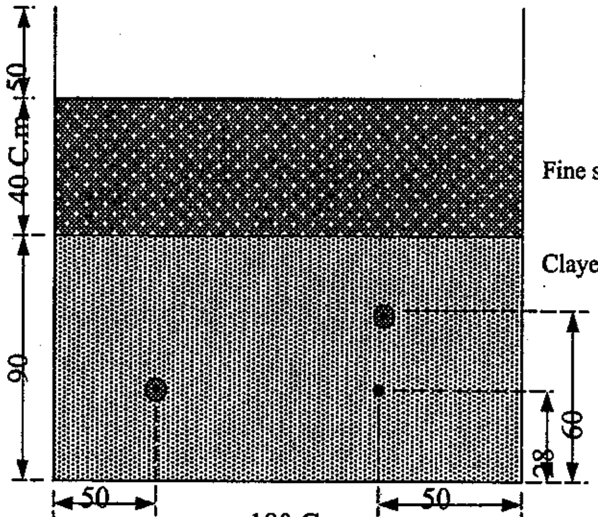
Construction of model slope

The model slope was constructed with 1:0.3 (Vertical: Horizontal) slope gradients and the height of the slope 0.9m.

Construction sequence adopted for model as follows.

a) First moisture content of the soil fill was determined. Additional amount of water was added to the soil in order to maintain 18% water content in the soil fill.

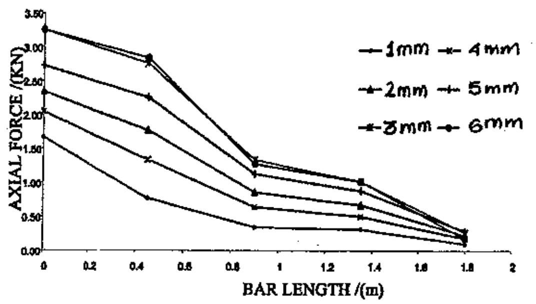
b) The homogenized moist soil was immediately moved to the setup and spreaded. To achieve a uniform unit bulk weight throughout the model, filling and compaction of the soil were carried out in several stages (incremental lifts). The model consists of four incremental lifts. Each lift was compacted by a whacker until unit bulk weight of 2302 kg/m³ (at w.c=20%) was obtained.



Fig(j)

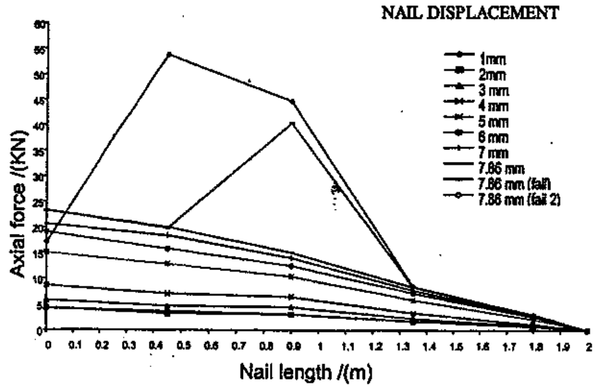
5. RESULTS

Bar force distribution of reinforcement,2min length placed 0.6 m below the top surface



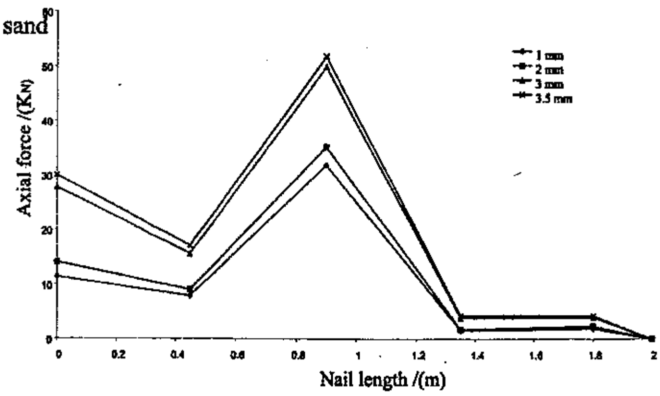
Graph (a)

Bar force distribution of grouted soil nail, 2 m in length Installed 0.3 m below the top surface



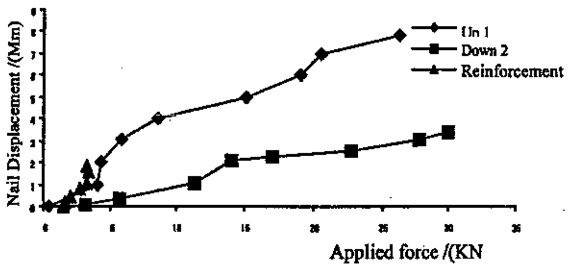
Graph (b)

Bar force distribution of grouted soil nail, 2 m in length Installed 0.3 m below the top surface



Graph (c)

Graph of tension force at head
of nails VS Displacement at nail



Graph (d)

6. DISCUSSION

- Over burden pressure of the reinforcement bar is higher than the grouted nail installed at top. Even though the over burden pressure is high, the reinforcement bar gives low pullout resistance. This may be due to low friction.

- Bar force distribution of top grouted nail that we obtain is similar to the pattern of distribution, which was obtained by Seto et.al (1985). Here for the nail displacement 7.86mm, while tension force at the head of the nail remains constant, there is a sudden reading change in strain gauge3. So it may be due to bending resistance. When the nails have some bending, they are subjected to bending stiffness moment and shear forces in addition to tension force. In this case, the points of maximum tension in the nails divide the mass into an active zone and a passive zone.
  Model and full-scale experiments have shown that bending resistance of the nail is only really mobilized in the vicinity of failure when a surface or zone of shear develops in soil nail.

- In the bar force distribution of bottom-grouted nail, reading of strain gauge3 from the initial is higher than the strain gauge4. So it shows that failure is already mobilized in that zone.

- From the top and bottom bar force distribution at failure, failure surface can be predicted approximately. For the model installed with perfectly flexible nails, if we increase the surcharge load, we can predict failure surface clearly.

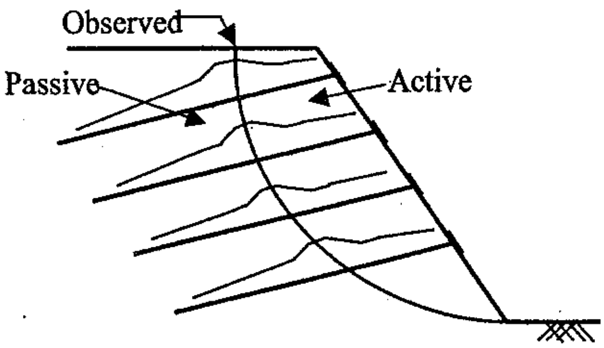


Fig (k)

7. CONCLUSION

According to the aim of the project "Soil nails and their pullout resistance ", we able to get

- Bar force distribution along grouted nail
- Pullout resistance of grouted and ungrouted nail
- Bar force distribution along ungrouted nail

But the other important aim that pullout resistance under different confining stresses, couldn't be reached, because of failure of the model slope. The maximum pull out force was achieved when the nail displacement reached at 7.86 mm. At this nail displacement, failure has been mobilized

Even though we had all the difficulty, this model test was carried out successfully and we able to reach the target of the project. Here we can't predict any design guideline due to lack of test data's that we got from the test. In order to design guideline, we have to do several tests by varying the test conditions and parameters. So it is better to continuity to reach the target

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Computer Program For Cost Analysis of Earth Retaining Walls

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ABSTRACT

This paper describes the development of a software package for the design of an earth retaining wall, which is the most suitable in a particular situation and cost- optimized. The design based on BS8002 is carried out using Visual Basic 5.0 as a programming language. The British Code of practice for earth retaining structures, BS8002, which introduced the concept of limit, state design, emphasizing the serviceability limit state on the effective stress analysis, deals with the externally stabilized type of earth wall using a mobilization factor.

This software, which incorporates two types of retaining walls namely gravity retaining wall, cantilever retaining wall, is applicable under various conditions including inclined backfill, horizontal backfill, surcharge, water table effect and multi soil layers of backfill. The backfill and founding soil may be cohesive or cohesion less (c, ϕ soil). The results obtained from the software are displayed in a graphical form of total cost required for a particular wall construction vs. height, which helps the user in the selection of the most suitable and cost-optimized wall.

Because of the user friendliness of the Visual Basic05 user can easily enter the data required for an analysis in the program, the user will be asked to input data such as cohesion© in angle of friction *(O)* of backfill soil, founding soil, surcharge and water table height etc. Then based on BS8002, design values of C, *O, Coefficient* of lateral earth pressure (K) and forces acting on the wall are calculated by the program. The program runs iteratively until all the stability conditions (overturning moment, sliding resistance and bearing capacity of the soil) are satisfied. Using this iterative analysis, the optimum results achieved. The accuracy of this program is 0.1% comparing with the manual calculation. From our result it can be concluded that the gravity retaining wall with steps is most suitable up to the height of 2.5m.Above this certain height the cantilever retaining wall is compatible.

INTRODUCTION

Among today's competitive business environment, engineers should be aware of the most recent, developments in engineering application and should always use innovative, effective techniques to optimize cost and time. Therefore it is a great challenge to carryout optimum design.

Even though it is possible to construct several types of earth retaining walls in a certain condition, the selection of the most suitable and cost-optimized earth retaining wall for a particular situation is very essential. The software developed here leads engineers to select the most suitable and economical wall according to the prevailing field situation. By giving all required data in their specified places in the program, the software gives corresponding results in a few seconds.

OBJECTIVES

The objects of the study has been to develop a computer program to analyze earth retaining walls which will help the user to select the most suitable, cost-optimized wall and to make a cost comparison of retaining walls under local conditions using the developed software.

SCOPE OF THE STUDY

The program development on geotechnical design of earth retaining walls is based on British Standard 8002 as well as Coulomb theory. The results obtained based on British Standard 8002 are compared in this paper. The software consists of geotechnical design of two types of earth retaining walls, namely gravity earth retaining wall and cantilever earth retaining wall. Various field situations are considered in the program, that is, presence of water table, surcharge, horizontal backfill, inclined backfill and multi soil layer backfill. The main idea of developing this software is to determine the total cost of construction of a retaining wall, which is the optimum for a given retained height in a particular field situation .The results of the analysis are displayed in the form of a set of graph of graphs of total cost versus height of the walls, which is used to compare the cost involved for each choice of retaining wall.

CONCEPT OF MOBILIZATION FACTOR

The basis of this method of design given in BS 8002 is the reduction of soil peak shear strength values on both the active and passive sides of the wall, to values, which would be representative of mobilized shear strengths with the permitted wall movements prevailing under working conditions. Shear strength parameters mobilized are obtained by the use of a mobilization factor as. design C=C/M and design tan ϕ= tan ϕ /M. M is the Mobilization factor as defined in BS 8002.

CALCULATION OF TOTAL COST OF CONSTRUCTION

Whenever a structure is designed, the total cost involved in the construction of that structure should be estimated from an economic point of view. Since there are several choices for a suitable structure, it is first better to analyze all the possible choices, and then select the most suitable and cost-optimized one. In order to estimate the total cost of construction, the developed software needs the current cost of required materials, labour rates & cost, excavation cost and cost for formwork etc. The rates and cost are obtained from Highway Schedule and Rates of Road Construction And Development Company (Pvt.). The cost details used are Materials cost including transport, Concrete Random rubble masonry. Steel items, labour cost with manpower calculated using Number of required days per 1m^3 concreting/random rubble masonry with details of required skilled and unskilled labourers. Excavation cost depends on the existing earth including loading, unloading and transporting within 25 meters. The Cost of earth filling and compaction including loading, unloading and transporting and the cost of Formwork includes dismantling.

COST COMPARISON OF RETAINING WALLS AND VARIFICATION

In a particular location, different types of retaining wall may be constructed. For a given height of a retaining wall, all feasible types of wall are first considered and the minimum value of base width of each type of wall are calculated using the computer program. By giving all possible expenses related to each type of wall, the optimum total cost of a unit length of wall is calculated for each type of retaining wall. These results are plotted in graphs, which are showed in fig1, fig2. So the cost for each type of wall may be compared. Result calculated by the program is verified with manual calculation. All basic steps involved in the development of this software are given in a flowchart (chart 01).

CONCLUSIONS

As today's trend changes towards an era in which computer tasks part in every task of daily life, this is an appreciable endeavor to do design-a tedious task as manual calculation- as computer program. The following graphs have been derived in particular conditions. I.e., the water table height is one third of the total wall height, surcharge of 20kN/m^2 and homogeneous backfill soil and founding soil. In this program, steel amount has been calculated considering maximum moment acting on the wall. If various moment acting along the section were considered the results would be more economical.

Under such conditions, gravity retaining wall with steps is the most economical than others up to wall height of 1.5m. Within 1.5m to 2.5m, gravity wall with horizontal backfill is more economical. Beyond 2.5m, cantilever wall is more economical. The results obtained from computer program and manual calculation have been compared and checked. The accuracy is 0.1%.

RECOMMENDATIONS FOR FUTURE WORKS

This program development on design of earth retaining walls can be extended to the following aspects:

a) Design and analysis of sheet pile walls and Gabion walls.

b) Analysis of walls considering seepage- parallel to the backfill slope or horizontal.

c) Design and analysis introducing some additional parts of retaining walls such as shear key (to increase shear resistance) and buttresses in walls.

d) Precast retaining walls.

SUGGESTION

To get values from various graph, EXCEL package was used to fit polynomial and to get corresponding equations. Since interpolation was done using these equations, program became lengthy. If some other methods were used, it reduces the code length.

The results obtained from the analysis of BS8002 can compared with the available software.

All basic steps involved in the development of this software is given in a flowchart below:

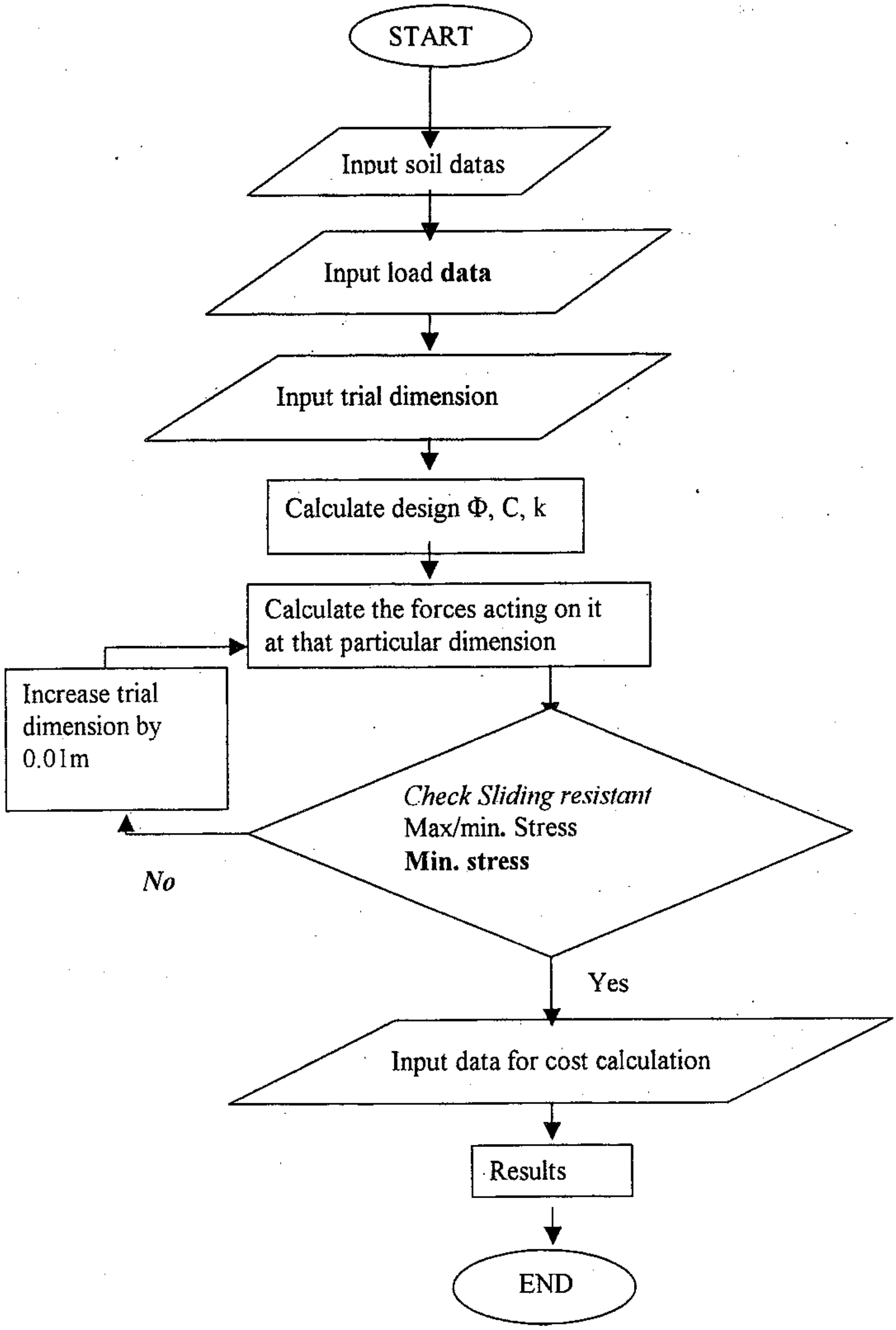

Chart 01

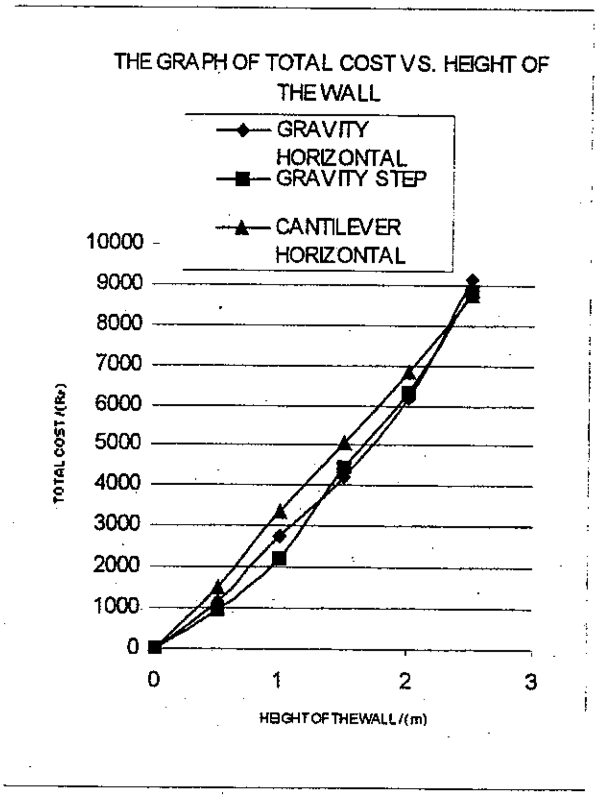

Fig 01

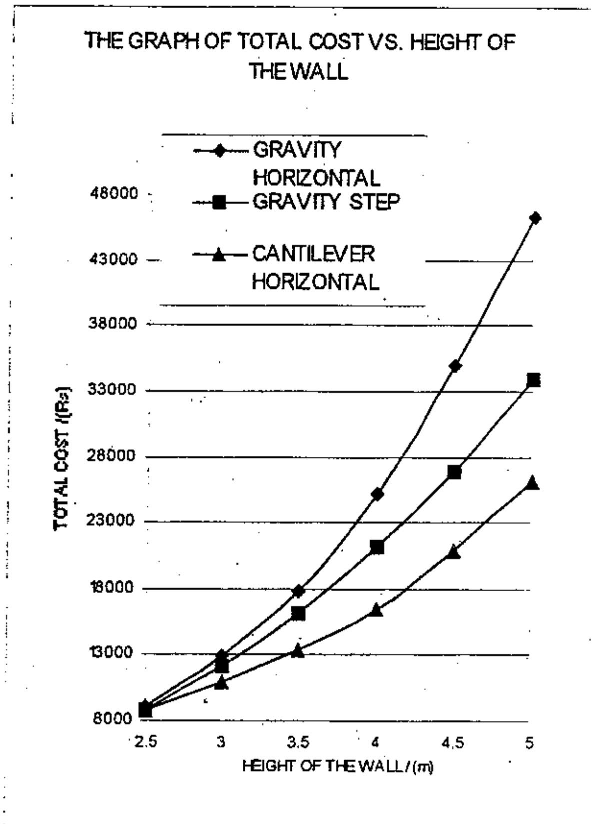

Fig 02

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CLASSIFICATION OF SRI LANKAN RESIDUAL SOILS

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ABSTRACT

In this study an attempted is made possible simple classification system for Srilankan residual soils for engineering purposes. Having selected four sites in the vicinity of Kandy area representing soils from four different parent rocks several laboratory tests such as, Particle size distribution test, Atterberg limit test, Ignition loss test and Specific gravity test and field tests such as, void ratio test, Vane shear test and In situ dry density test were From these laboratory and field tests for completely weathered residual soil samples, it could be observed that there is a relationship between Ignition loss and In situ dry density with void ratio.

1.INTRODUCTION

1.1GENERALBACKGROUND

Geologically about 90 of the Sri Lanka is made up of highly crystalline rocks belonging to one of the most ancient and stable part of the each crust, the Indian shield. The complete weathering of the parent rock has resulted in the formation of residual soils. When compared with the sedimentary soils these residual soils differ significantly in their physical and mechanical behavior. The process of weathering, Leaching and loosening rocks create residual soils.

The soil classification commonly used in sedimentary soil for example unified classification systems are based on the properties of the soil in its remolded state. These methods are often misleading with residual soils. Atterberg limits, Particle size distribution curves and specific gravity depends on how the soil is pretreated before testing.

Even though, there are several classification systems available in the literature for engineering purposes there is no universally accepted method of classification specially for Sri Lankan residual soils it is a great importance as this type of soil cover almost all the island.

Therefore this study is intended to examine a possibility to propose a suitable classification method for local residual soils for engineering purposes.

2. OBJECTIVE OF THE STUDY

The main objective of this study is to propose a suitable simple classification method of local residual soils for engineering purposes.

3. SCOPE OF THE STUDY

Residual soils exist locally to a greater extent originating from verity of parent rocks. In this study, limiting to Kandy area, four sites are selected for the investigation. The residual soils that appear at these sites originate from different parent rocks. A series of possible laboratory tests are performed for these soils to find out possible correlation for classification purposes.

4. METHODOLOGY

• Even though, residual soils are available in the island, four sites were identified for investigation;
1.Kalugamuwa
2.Dangolla
3.Kandy Hospital site
4.Hanthana

•Two categories of samples were taken; disturbed and undisturbed for examination.

•Field tests such as insitu density and vane shear were performed. Insitu density test was carried out by sand replacement method.

•Specific gravity test was carried out using small pyknometer method. And oven dried sample was used.

•To determine particle size distribution of the samples larger than 75μm dry sieving was carried out and percentage of silt and clay was determined by hydrometer sedimentation method. Air dried soil sample was used in this method and samples were broken down using rubber pestles.

•Liquid limit test was carried out using cone penetration method. And mixing time was limited to 15 min.plastic limit teat was carried by using thread rolling method. Each sample was thoroughly mixed to 10 min with distilled water to bring it nearly to its plastic states. In both tests air dried soils samples were used.

•Ignition loss was obtained when samples was heated at 450°C.

•Void ratio was computed by knowing the insitu bulk density, Insitu moisture and specific gravity.

•All the teats were carried out according with BS 1377 with some minor changes to suit local condition.

5. RESULTS AND DISCUSSION
5.1 VISUAL OBSERVATIONS AND PARENT ROCK TYPES

Field investigations were carried to find the features of weathering materials. The general variations of weathered profiles were observed. Weathering grade such as soil, completely weathered, moderately weathered and fresh rock were identified according to field recognition method by Little (1969). Parent rocks could not be seen in sites due to thick residual weathered overburden. The observations were used as guideline to identify the parent rock.
The parent rock types and mineral components of each sits are given in the table.

SITE	PARENT ROCK	MAJOR MINERALS
Kalugamuwa	Garnet silimanite geniss	Garnet, Silimaite,Geniss, Graphite
Dangolla	Garnet silimanite geniss	Garnet,Silimaite, Geniss
Kndy Hospital	Quartzo feldsphatic geniss	Quartz, Feldspar, Geniss
Hanthana	Quartzite	Quartz,Feldspar

Table 01

5.2. USE OF BRITISH SOIL CLASSIFICATION SYSTEM

In the process of proposing a suitable classification system, initially the samples were classified by conventional methods of soil classification such as British Soil Classification system. The results are given below.

SITE	SYMBOL	NAME
Kalugamuwa	CIS	Sandy CLAY of intermediate plasticity
Dangolla	CES	Sandy CLAY of extreme plasticity
Kandy Hospital	SCI	Very clayey SAND of intermediate plasity
Hanthanna	MHS	Sandy SILT of high Plasticity

5.3. ATTERBERG LIMITS

Liquid limit and plastic limit are routinely performed to provide a qualitative indication of soil type and a feel for potential soil behavior.

The Plastic index and liquid limit values are plotted for all fore sites.

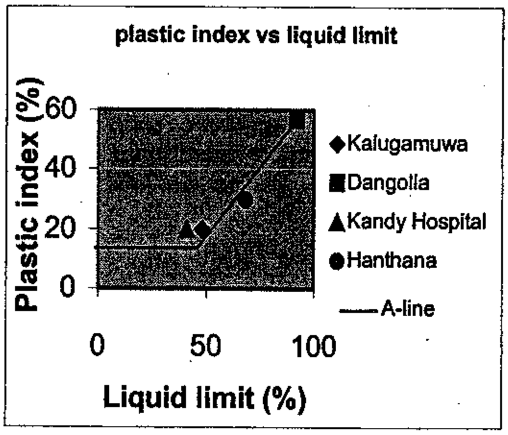

Fig 01

5.4 GRAIN SIZE DISTRIBUTION

Grain size distribution of residual soils depends on degree of weathering and its parent rock. It can be seen that from Dangolla consists of larger fraction of (45%) clay size particles. This is why soil exhibits high plasticity. The soil from Kalugamuwa and Hospital site show lowest clay fraction among the four sites.

5.5 IGNITION LOSS

The Ignition loss is related to the ratio of $Al_2O_3:SiO_2$. This ratio increases with increasing ignition loss this suggests that the ratio $Al_2O_3:SiO_2$ is an effective index for degree of weathering. The Ignition loss results are plotted below for all four sites. It can be noted that the void ratio increases with increasing ignition loss.

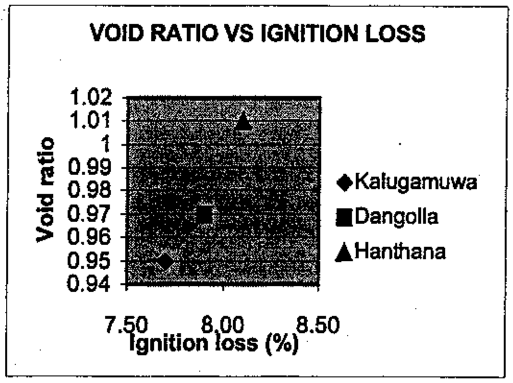

Fig 02

5.6 SPECIFIC GRAVITY AND INSITU DRY DENSITY

Nishda & Aoyama (1990) used void ratio and specific gravity of the soil particle to distinguish both the degree of weathering and the nature of parent rock for the same void ratio different mineralogy should give different specific gravities. The obtained results are shown below. By examining these results it can be seen that there is a relationship between void ratios with in situ dry density i.e void ratio decreases with increasing in situ dry density.

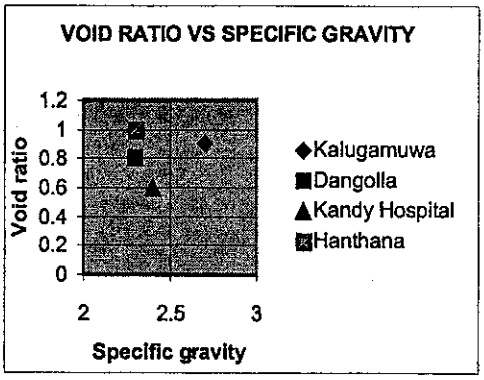

Fig 03

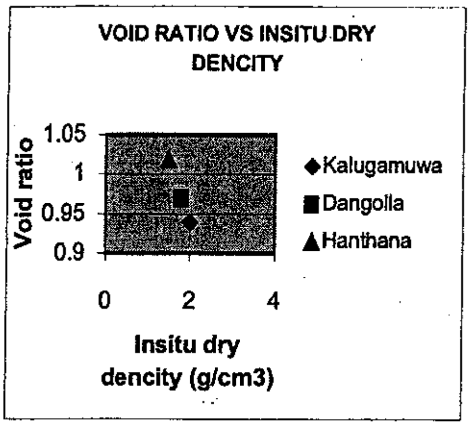

Fig 04

6. CONCLUSIONS

Residual soils, being in a structured state differ from that of sedimentary soils. Therefore, in classifying the residual soils, giving a proper concern to its structured state would be inevitable.

In this study an attempt is made to propose a simple classification method for local residual soils for engineering purposes. By selecting four sites in the vicinity of Kandy undisturbed sampling and testing have been carried out. Theses four sites consist of residual soils that originally from different parent rock types.

By examining the test results it could be seen that

- Void ratio increases with increasing ignition loss.

- Void ratio decreases with increasing in situ dry density.

Therefore, void ratio can be incorporated in the classification on residual soils along with the Particle size distribution and Atteerberg limits.

7. EXTENSION OF THE CURRENT STUDY

For the study like a larger number of test data are required to find clear demarcations for engineering behavior of different soils. Therefore, even from the same source of soil, larger numbers of tests are recommended representing different sections of the deposit. In addition X-ray diffraction tests also required in identifying the mineralogy of the soil.

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ENGINEERING PROPERTIES OF SRI LANKAN RESIDUAL SOILS

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ABSTRACT: This project aims to investigate the engineering properties of residual soil, completely decomposed soil from Kalugamuwa site of known weathering profile and parent rock. In this experimental study, laboratory tests such as compaction test, permeability test, triaxial test & 1-D consolidation test and field tests such as in-situ density test were carried out according to British Standards. We hope that these experimental analyses will provide a great deal of assistance to researchers for summarizing the engineering properties of Sri Lankan residual soil in future.

1. INTRODUCTION

Residual soil is generally defined as a soil formed by weathering of rock in place, but with the original rock texture completely destroyed. The processes of weathering, leaching and loosening of rock, produce residual soil.

There is no universally accepted standard classification system of residual soil available. Classification in terms of weathering ***zones*** and weathering ***grades*** is essential for engineering design. In the case of Sri Lanka, the major factor that influences the formation of soils is the climate while the parent material generally has a minor influence.

In Sri Lanka 90% of the land is made up of highly crystalline rock belonging to the south Indian shield (Cooray 1967). Residual soil formed by weathering of these rocks is found almost anywhere. Laterite, which is locally known as ***cabook*** is found in the southwestern part of the island within a belt of 10 Km from the coast and upto 30 m above mean sea level (Thurairajah & Wijeykulasuria 1985).

Residual soils are not always ***bad soils*** in an engineering point of view. On many occasions they have been successfully used as a foundation stratum for important structures and for constructing dams and highways. Though slope stability problem exists in all types of earth materials throughout the world, they are generally the most important and difficult when residual soils are involved mainly due to partial saturation. Residual soils are generally strong and building of three or four storeys are founded on pad or strip footing whereas high-rise buildings are founded on raft or pile foundation. Along the highways in the hilly region, nearly vertical cuts (about 7 m) in residual soils can be seen to stand unsupported without any problems. Reinforced concrete retaining walls are used in residual soils only for major works. In water retaining structures such as dams, reservoirs and canals are commonly built in areas of residual soil, and there are many geotechnical problems like high permeability associated with this.

1.1 SPECIAL PROBLEMS

Residual soils have received very little attention from researchers, mainly because of the extreme difficulties of working experimentally with these soils. Inhomogenity and anisotropy are the major features of residual soil that adds high permeability and non saturation behaviour and render them so difficult to deal with from an engineering point of view. The inhomogenity is the most serious problem, which makes the small test sample unrepresentative and misleading. In laboratory strength tests, influence of relict joints, boulders, variability of soil matrix may not lead the testing in a meaningful way. Presence of gravel, cobble and boulders in the soil matrix cause serious sampling problems. The encountered difficulties have kept the research in this area at a minimum level.

2. OBJECTIVE

The main objective of this study is to investigate the engineering properties of residual soils in Sri Lanka and to summarize these for engineering use. Particularly, completely decomposed soil from a single site of known weathering profile and parent rock is selected for these experimental studies

3. SCOPE OF THE STUDY

This studies limited to the residual soil taken from Kalugamuwa site of which the soil consist of mainly ***Feldspar & Granite.*** As a new initiative towards this, all the experimental findings in this research are documented so that future research works will be supplemented in achieving this final goal.

4. METHODOLOGY

- Both disturbed sample and undisturbed sample were collected for experimental studies during our project.
- In the case of undisturbed sample, ring sample and block sample were collected. Wooden cubes with 300mm sides were used to collect

block sample. They were waxed and trimmed in-situ. Block sample ensure the minimum disturbance during transportation. In our case the block samples were trimmed vertically.

- The laboratory tests like compaction was carried out using disturbed samples where as permeability, 1-D consolidation and triaxial were carried out three times using undisturbed samples.
- In triaxial test, three samples in the same condition are tested under 50,100,150 kPa cell pressure. The cohesion and internal angle of friction that reflect the shear strength of soil can be obtained in a certain condition. Triaxial test for Kalugamuwa site were carried out in the condition of consolidated undrained.
- 1-D consolidation test is carried out in the odeo meter with double drainage condition. The coefficient of consolidation (C_v) can be computed using square root time fitting method.
- All the laboratory tests and field tests were carried out in accordance with British standards

Compaction test – BS 1377 : Part 2 : 1990
Permeability test – BS 1377 : Part 5 : 1990
Triaxial test – BS 1377 : Part 8 : 1990
Consolidation test – BS 1377 : Part 5 : 1990
In-situ density test – BS 1377 : Part 9 : 1990

5. RESULTS

Kalugamuwa soil (gravel 8%, sand 52%, silt 20%, clay 20%), which consists of mainly Feldspar & Granite, can be tabulated as follows

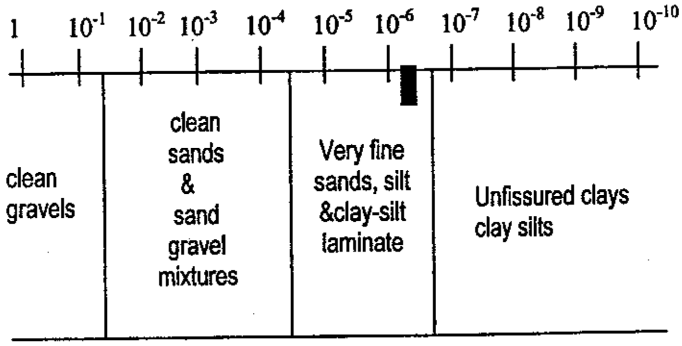
PARAMETER	VALUE
Classification properties	
Liquid limit	46%
Plastic limit	26%
Plasticity index	20%
Specific gravity	2.628
Void ratio	0.943
In-situ density	1.572 Mg/m^3
Moisture content	20.5%
Ignition loss	7.712%
Engineering properties	
Maximum dry density	1.61 – 1.68 Mg/m^3
Optimum moisture content	19.5 – 20.3 %
Coefficient of permeability (k)	2 – 3 * 10^{-6} m/s
Cohesion (c')	0
Internal angle of friction (ø')	32 – 36 °
Coefficient of consolidation (C_v)	15 – 60 m^2 / year
Coefficient of compressibility (m_v)	0.05 – 0.25 m^2 / MN

Table 01

6. DISCUSSION

From compaction test results, it was found that the maximum dry density value is around 16 kN/m^3 when the optimum moisture content 20%. It can be observed that dry density of residual obtained by in-situ density test is increased by 5% by means of compaction. It was also noted that the dry density obtained by compaction test (1.646 Mg/m^3) is greater than the dry density obtained by in-situ density test (1.57 Mg/m^3). However residual soil fill properties can be very variable, and an unacceptably large number of compaction tests are required to ensure meaningful results. In such cases where this material is used in a core of an earth dam, special care has to be taken to achieve lower permeability. It means that prior treatment is needed to achieve higher density so that k is minimum.

The presence of fine particles (40%) in Kalugamuwa site soil (sandy silt soil) results in the values of k significantly lower (the order of 10^{-6}m/s) than the value for the some soil without fines. It was noted from the comparison between consolidation and permeability test results that the coefficient of permeability obtained by falling head permeability test is greater than the coefficient obtained from 1-D consolidation test. This may be because of the lower degree of saturation (0.68) found in consolidation test due to the non-saturation behaviour on residual soil. Permeability test results for reddish brown sandy silt comply with standards in (BS 8004: 1986) as shown below



Coefficient of permeability (m/s) (BS 8004: 1986)
Table 02

Kalugamuwa residual soil (40% fine, 52% sand, 8% gravel) would be classified as a frictional-cohesive type according to the classification criterion shown below. Results (32°-36° for ø' and 0 for c') comply with the range of 25°-35° given in the table 03,

Type	Criterion	Angle of shearing resistant
Frictional	Sand and gravel content greater than 80%	33° - 40°
Frictional-cohesive	Sand and gravel content 50%-80%	25°-35°
Cohesive- frictional	Sand and gravel content less than 50%	30°-35°
Cohesive	Clay content greater than 20% PI greater than 30%	-

Shear strength characteristics of some residual soil in Hongkong
Table 03

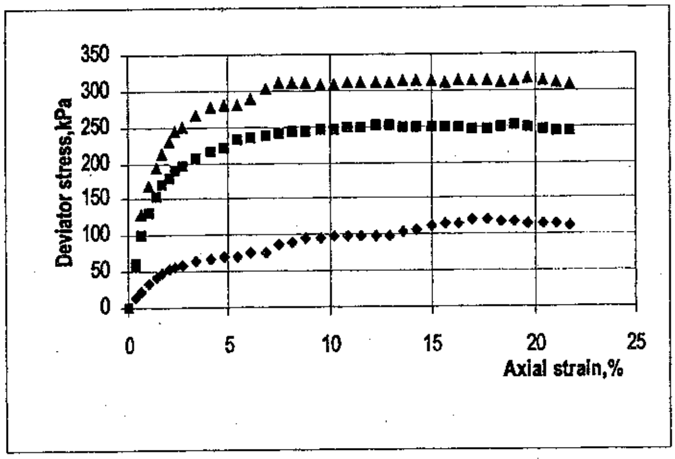


Fig 01

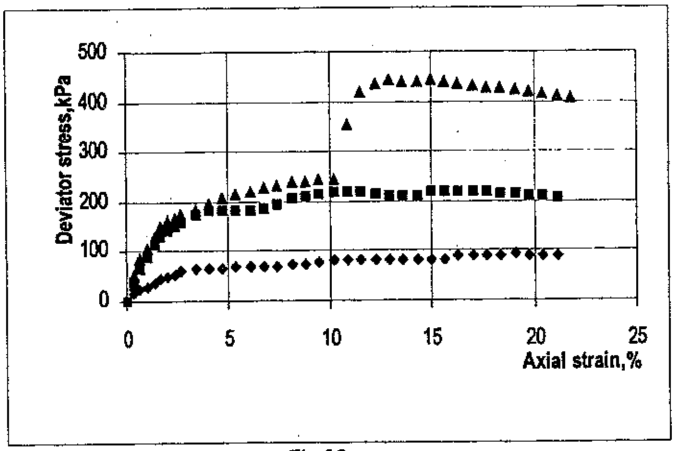
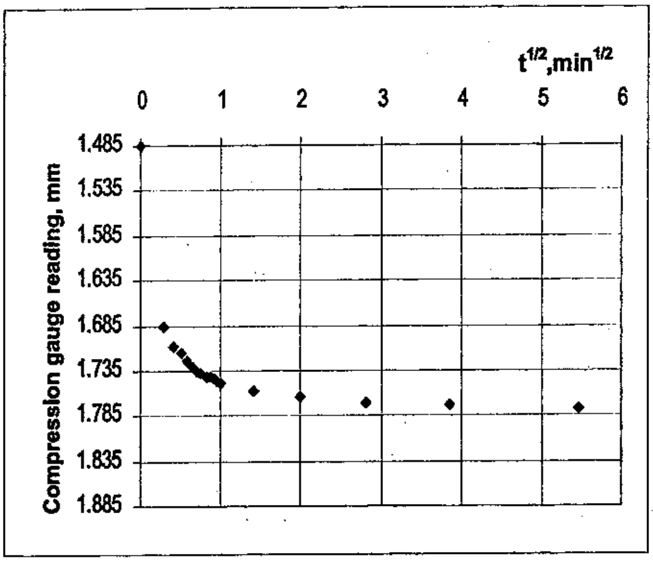


Fig 02

◆ cell pressure 50 kPa ■ cell pressure 100 kPa
▲ cell pressure 150 kPa

THE GRAPH OF DEVIATOR STRESS VS AXIAL STRAIN

Triaxial test graph for 150 kPa cell pressure (trial 2), shows some sudden changes of deviator stress due their heterogeneity, which are difficult for sampling and testing in a meaningful way.



Laboratory consolidation curve: square root of time fitting method
Fig 03

In odeometer tests, there is no way to find out the degree of saturation before applying load in odeometer test. As a result of this shortcoming, it is difficult to get enough points to draw the linear portion of primary settlements.

- To overcome this, time interval has to be reduced. Since it is difficult to get the reading manually, computerization is the better solution
- To have more points in linear portion, single drainage condition can be adapted. By using larger sample, more points can be obtained
- To achieve fully saturation, higher ceating pressure with backpressure would be necessary.

7. CONCLUSION

We hope that the results obtained from this series of laboratory tests will provide great deal of guidance to researchers for summarizing the engineering properties for different types of residual soil (according to the parent rock) in future. The following conclusions may be pointed out from this experimental program,

- Special attention to be given on the method of saturation of residual soil in odeo meter test, because there is no way to find out the degree of saturation before applying load. It is necessary to consider this shortcoming into account in future research.

- In triaxial testing, 37.5 mm diameter sample can be unrepresentative and misleading. It would be better to test larger sample to avoid the problems associated with heterogeneity.

- It was noted that the compaction effort is increased by 5% by means of standard proctor compaction test. This may be due to the nature of residual soil.

8. SUGGESTION FOR FURTHER WORK

In this project, since we concentrated on the experimental analysis of residual soil at Kalugamuwa, we couldn't summarize the engineering properties with different types of parent rocks. It would be better, if this experimental study is carried out for several types of residual soil (according to parent rock) and produce an engineering properties chart.

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BEARING CAPACITY OF STRIP FOUNDATION ON GROUND IMPROVED BY SAND COLUMN AND SAND REPLACEMENT

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Abstract:

A computational method was introduced to determine the bearing capacity of strip foundation on weak soil. The study performed by using software. Which based on constitutive matrix for the Elasto plastic analysis under plain strain condition for isotropic material. The ground improvement methods, Sand column and Sand replacement, were introduced to improve the bearing capacity of strip foundation on weak soil. The analysis of results is observed with different depths of sand column and sand replacement layers. This paper attempts to identify the significant improvement in bearing capacity of strip foundation by using both methods as expected.

1. Introduction:

In Geotechnical Engineering it is often said that "nothing can be determined with out measured data", Geotechnical engineering can be said as the study, which has made progress by concentrating on how to clear the uncertainty in materials, soil test, and design procedure concerned.

The places have peat deposit like Japan; the soil improvement techniques have been made a remarkable progress since peat deposit have small shear strength and large consolidation. A variety of improvement methods like deep mixing, stone column lime column, ground freezing, chemical stabilization, sand drains etc have analyzed in early years.

In our project the bearing capacity of a strip footing on a ground improved by sand column and sand replacement have been dealt numerically using FORTRAN and details are produced in this paper. The Dascar programme, which is developed by HIDEKI OHTA.

The constitutive equation proposed by Ohta and Sekiguchi (1979) take account of the effects of stress state by changing the initial stress state by increasing shear and the stress reorientation. Thus the program coded to solve a boundary valve problem has been termed as DASCAR (Deformation Analysis considering stress Anisotropy and reorientation). This model has been developed as an extension to the original cam-clay model which does not take the model proposed by Sekiguchi(1977) which does not take account of the stress induced anisotropy in clays. However the effect of secondary consolidation formulated in the model by Ohta and Sekiguchi (1979) is not considered for the numerical simulation. The significance of program DASCAR is its capability to consider stress induced anisotropy and stress reorientation.

The determination of ultimate bearing capacity is one of the most important aspects of geotechnical engineering in designing structures and for clayey soils the un-drained shear strength will have a significant effect on the value of ultimate bearing capacity. Here numerical assessment of the effect of sand column depth, sand replacement depth on ultimate bearing capacity has been analyzed.

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2.1. Governing equations:

$$\begin{pmatrix}\Delta\sigma'xx\\ \Delta\sigma'yy\\ \Delta\sigma'xy\\ \Delta\sigma'zz\end{pmatrix} \equiv \left\{ \begin{pmatrix}\lambda+2\mu & \lambda & 0\\ \lambda & \lambda+2\mu & 0\\ 0 & 0 & \mu\\ \lambda & \lambda & 0\end{pmatrix} - \frac{1}{C_1}\begin{pmatrix}A^2{}_{XX} & A_{XX}A_{YY} & A_{XX}A_{XY}\\ A_{XX}A_{YY} & A^2{}_{YY} & A_{YY}A_{XY}\\ A_{XX}A_{XY} & A_{YY}A_{XY} & A^2{}_{XY}\\ A_{XX}A_{ZZ} & A_{YY}A_{ZZ} & A_{XY}A_{ZZ}\end{pmatrix}\right\}\begin{pmatrix}\Delta\varepsilon_{xx}\\ \Delta\varepsilon_{yy}\\ \Delta\varepsilon_{zz}\end{pmatrix}$$

Where,

$$\lambda = \frac{3\nu'}{1+\nu'}\cdot\frac{\Lambda}{MD\,(1-\Lambda)}P'$$

$$\mu = \frac{3(1-2\nu')}{2(1+\nu')}\cdot\frac{\Lambda}{MD\,(1-\Lambda)}P'$$

$$A_{ij} = \lambda f_{kk}\delta_{ij} + 2\mu f_{ij}\,(i,j = x,y,z)$$

$$f_{kk} = f_{xx} + f_{yy} + f_{zz}$$

$$C_1 = \lambda f^2{}_{kk} + 2\mu\left(f^2{}_{xx} + f^2{}_{yy} + f^2{}_{zz}\right) + f_{kk}$$

$$f_{ij} = \frac{D}{3P'}\left[M - \frac{3}{2\eta\cdot}(\mu_{kl} - \mu_{klo})\right]\delta_{ij} + \frac{3D}{2\eta\cdot P}(\mu_{ij} - \mu_{ijo})$$

$$\eta_{ij} = \frac{\sigma'_{ij}}{P'} - \delta_{ij} \quad ; \quad \eta_{ijo} = \frac{\sigma'_{ijo}}{P'_o} - \delta_{ij}$$

$$\eta_{kl}(\eta_{kl}-\eta_{klo}) = \eta_{xx}(\eta_{xx}-\eta_{xxo}) + \eta_{yy}(\eta_{yy}-\eta_{yyo}) + \eta_{zz}(\eta_{zz}-\eta_{zzo}) + 2\eta_{xy}(\eta_{xy}-\eta_{xyo})$$

$$\eta^* = \sqrt{\frac{3}{2}(\eta_{ij} - \eta_{ijo})(\eta_{ij} - \eta_{ijo})}$$

$(i, j = x, y, z)$

The volumetric plastic strain is the tracer of $\varepsilon_{ij}{}^{p}$, which is computed as,

$$\varepsilon_{kk}{}^{p} = N\frac{\partial f}{\partial \sigma'_{kk}}$$

$$\varepsilon_{kk}{}^{p} = N\frac{\partial f}{\partial P'}\frac{\partial p'}{\partial \sigma'_{kk}} \qquad P' = \frac{1}{3}\sigma_{ij}$$

$$\varepsilon_{kk}{}^{p} = \frac{N}{3}\frac{\partial f}{\partial P'}\frac{\partial \sigma'_{ij}}{\partial \sigma'_{kk}}$$

$$\varepsilon_{kk}{}^{p} = \frac{N}{3}\frac{\partial f}{\partial P'}\delta_{ik}\delta_{ik}$$

$$\varepsilon_{kk}{}^{p} = \frac{N}{3}\frac{\partial f}{\partial P'}\delta_{ii}$$

$$\varepsilon_{kk}{}^{p} = \mathrm{N}\frac{\partial f}{\partial P'}$$

Therefore,

$$N = \frac{\varepsilon_{kk}{}^{p}}{\partial f/\partial P'} = \frac{V^{p}}{\partial f/\partial P'}$$

$$\varepsilon_{ij}{}^{p} = \frac{V^{p}}{\partial f/\partial P'}\frac{\partial f}{\partial \sigma'_{ij}}$$

Since,

$$f = \frac{\lambda - \kappa}{1 + e_0}\cdot \ln\frac{p'}{p_0} + D\mu^* - \varepsilon_v{}^{p} = 0$$

$$\frac{\partial f}{\partial p'} = \frac{\lambda - \kappa}{1 + e_0}\cdot\frac{1}{p'} - \frac{3D}{2\mu^*}(\eta_{kl} - \eta_{klo})\eta_{kl}\cdot\frac{1}{p'}$$

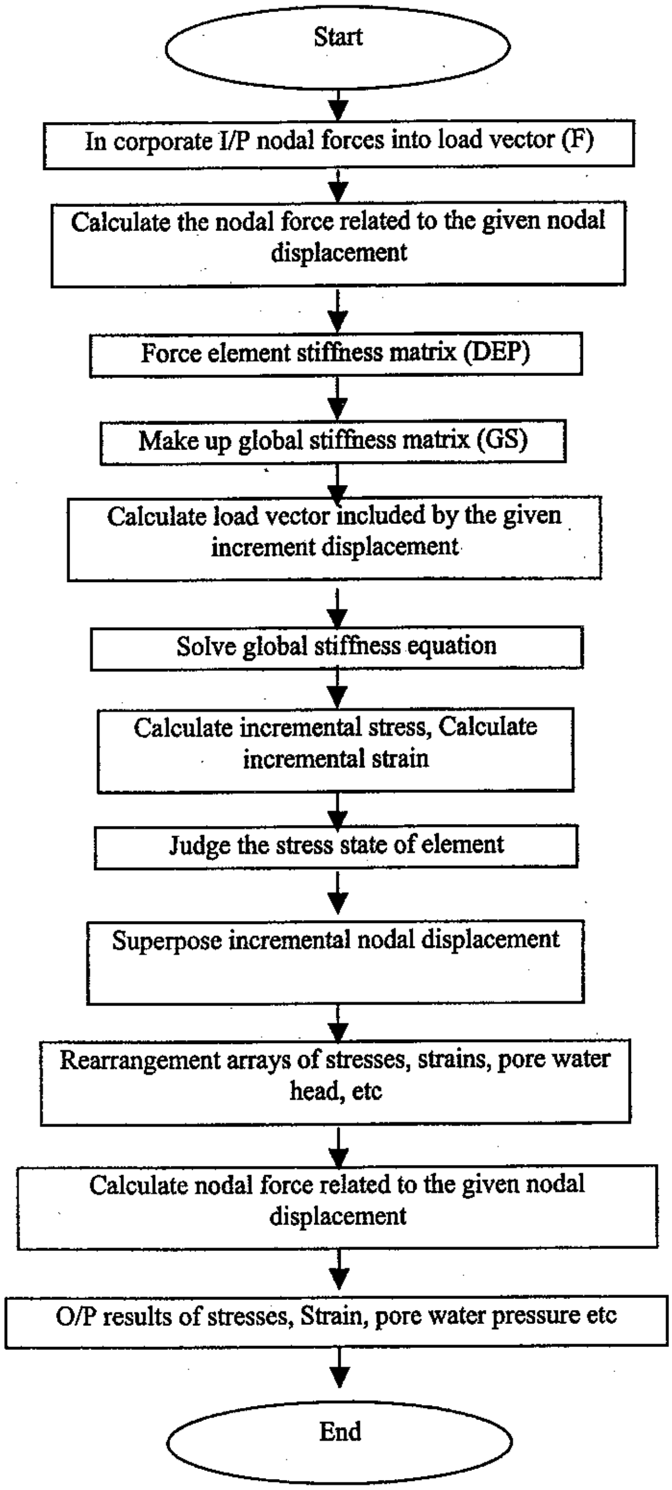
$$\frac{\partial f}{\partial p'} = \frac{D}{p'}\left[\frac{\lambda - \kappa}{D(1 + e_0)} - \frac{3}{2\mu^*}(\eta_{kl} - \eta_{klo})\eta_{kl}\right]$$

$$\varepsilon_{ij}{}^{p} = \frac{V^{p}\left\{\left[\frac{\lambda - \kappa}{D(1 + e_0)} - \frac{3}{2\mu^*}(\eta_{kl} - \eta_{klo})\eta_{kl}\right]\frac{\delta_{ij}}{3} + \frac{3}{2\eta^*}(\eta_{ij} - \eta_{ijo})\right\}}{\left[\frac{\lambda - \kappa}{D(1 + e_0)} - \frac{3}{2\mu^*}(\eta_{kl} - \eta_{klo})\eta_{kl}\right]}$$

Which reduces to,

$$\varepsilon_{ij}{}^{p} = \varepsilon_v{}^{p}\left\{\frac{\delta_{ij}}{3} + \frac{\frac{3}{2\mu^*}(\eta_{ij} - \eta_{ijo})}{\frac{\lambda - \kappa}{D(1 + e_0)} - \frac{3}{2\mu^*}(\eta_{kl} - \eta_{klo})\eta_{kl}}\right\}$$

2.2. Flow chart of program DASCAR:



3. Methodology:

In this study under strain control it was simulated an actual model of a rectangular container of 1570mm length, 280mm width and 750mm depth which was filled with clay sample up to 500mm depth after placing a drainage layer at the base of the container

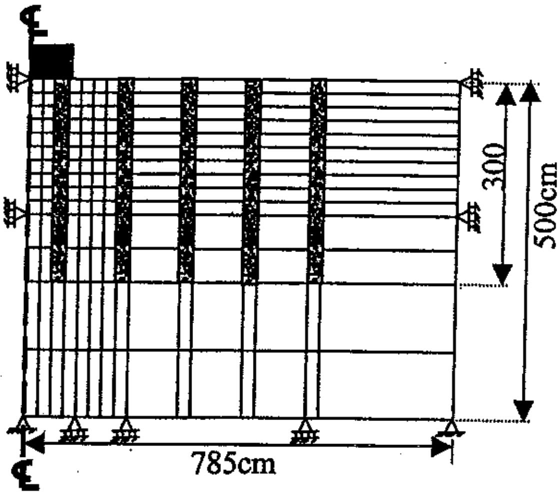


Fig 1: Configuration diagram of model and mesh for finite element simulation of foundation problem

The ultimate bearing stress was decided to take as 15% of settlement/width ratio from the graph of stress vs settlement/width ratio. Bearing stress was obtained from the results of above stated package, which has to be input fowling parameters.
Those parameters were determined following standard laboratory tests.

1. Institu density test.
2. Triaxial test.
3. Permeability test.
4. Consolidation test.
5. Direct shear.

Since the analytical model is symmetric in geometry and loading, half spaced model was taken in to account with 210-element mesh as shown in fig 1. Except top and bottom, all other external boundaries were kept impermeable.
Initially the analysis was conducted with clay alone and then continued with different with sand column depths of 100mm, 200mm, 300mm and 400mm, while column dimension and spacing kept constantly as 20mm and 100mm respectively. The analysis was extended to sand replacement method too, with different layers replaced by sand depths like 20mm(1 layer), 40mm(2 layers), 80mm(4 layers), 100mm(5 layers), 160mm(8-layers), 200mm(10-layers), 250 (11 layers).
From the plot of bearing stress vs settlement /width ratio, the ultimate bearing stress was picked corresponding to 15% of settlement/ width ratio. All other external hydraulic boundary conditions were kept impermeable and geometry boundary conditions are shown in the fig 1.

4. Results and Discussion:

Fig 2 and 3 give variation of bearing capacity with settlement/width of foundation ratio for different depth of sand column and sand replacement respectively. Those graphs horizontal axis were represented in a non-dimensional form for users convenient. Since this study has performed using computerized model the plots obtained don't show any ultimate point.

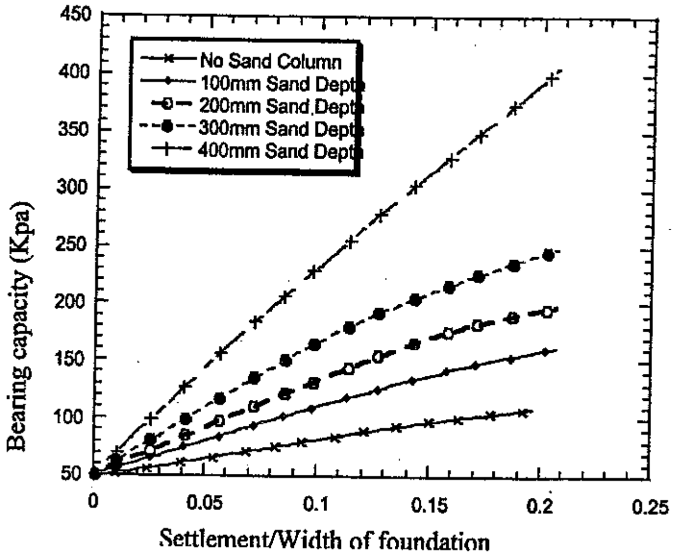


Fig 2 Graph of bearing capacity Vs settlement/width Of footing ratio (with different sand column depths)

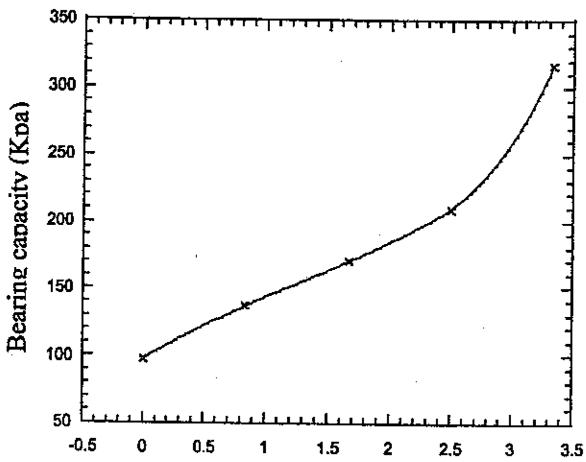


Fig 3 Graph of bearing capacity Vs column depth/width Of footing ratio (with different sand column depths)

Fig 4 and 5 shows the variation of bearing capacity with sand column depth/width of foundation and with replaced layer depth/width of foundation.

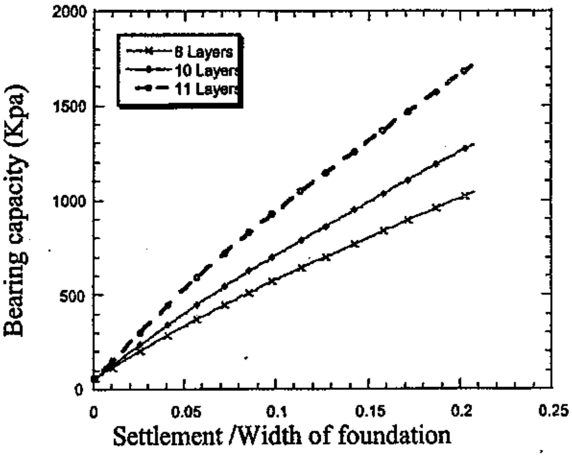


Fig 4 Graph of bearing capacity Vs settlement/width of footing ratio (With large sand replacement depths)

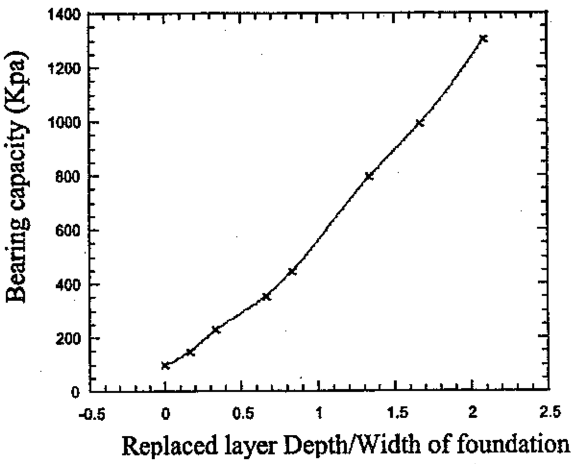


Fig 5 Graph of bearing capacity Vs replaced layer Depth/ width of foundation

Finite element analysis has been made to investigate the bearing capacity variation on a strip foundation with sand column and sand replacement, and the following results were pointed out.

DEPTH OF SAND COLUMN INSTALLED/(mm)	BEARING CAPACITY / (Kpa)
0	97
100	136
200	170
300	209
400	316

Table:1 Results of bearing capacity with different sand column

DEPTH OF SAND LAYERS REPLACED/(mm)	BEARING CAPACITY / (Kpa)
0	97
20	140
40	230
80	352
100	445
160	794
200	989
250	1303

Table: 2 Results of bearing capacity with different sand layer displacement

5. Conclusion:

In this analysis, the effect of sand column and sand replacement methods of improving the bearing capacity of strip foundation on weak ground is clearly observed.

This study can be extended to identify the variation of sand drain diameter and/ or spacing.

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CONTROL OF DIFFERENTIAL SETTLEMENT IN FOUNDATIONS

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Abstract: Numerical analysis has been performed to investigate the occurrence of differential settlement in foundations and for possible control measures.

The soil-water coupled finite element computation based on infinitesimal deformation theory is used. The foundation soil is assumed to follow the linear elastic constitutive law. Limiting the analysis only to two cases: **(a)** a raft foundation with eccentric loading over a homogeneous ground, **(b)** a raft with uniform loading over a homogeneous ground with inclined base. The action of rigid raft and pile is simulated using linear constraint conditions such as *no length change* between two nodes; *no angle change* between three nodes and *no direction change* between three nodes.

Through this investigation, it is found that there is significant settlement reduction with the introduction of piles into the system. In case of a single pile best location is found to be closer to the edge of the raft.

1. Introduction:

The foundation is a structure, which is, defined as the part of a structure in direct contact with ground and which transmits the loads of the super structure to the ground. The design of a foundation must satisfy two fundamental requirements;

1. Factor of safety against shear failure of supporting soil/rock must be adequate
2. Magnitude of settlement should be within tolerable limits. In addition, differential settlement of the foundation should not exceed the acceptable limits. Depending on the variety of factors such as type of soil in the bearing stratum, foundation geometry, magnitude of loads, either the shear failure or the settlement criterion will govern the design criterion of the foundation.

The settlement can be categorized into;

1. Immediate settlement, which occurs immediately after external load application
2. Consolidation settlement, which occurs as a result of generated excess pore water pressure dissipation.
3. Settlement due to creep, which is sometimes attributed to loss of soil structure

The later two are time dependent. The total settlement should include all the three above. Since the settlement due to creep is generally small compared to others and it takes considerable time to develop, in usual designs it is ignored. However for structures like airport runways, large telecommunication antennas, where precise settlement control is required, creep has to be taken into account.

If a structure settles uniformly as a rigid body, there may not be a danger to its performance. When differential settlement occurs between any two points in a foundation that may endanger;

1. Some structural elements connected
2. Intended performance

In case of buildings, bridges etc, occurrence of differential settlement may not only damage the structural elements and finishes but also lead to total collapse of the structure. The possibilities of differential settlements are of major concern such as in airport runways, ground antennas etc, where least margin tolerances are allowed. The most common case of occurrence of differential settlement is observed in approach roads to bridges, where the earth embankment settles at a faster rate than the bridge structure itself.

In order to control differential settlement, first it is needed to investigate the nature and occurrence of such. In this analysis it is expected initially to study the nature and occurrence of differential settlement and then to study possible remedial measures. In fact a numerical investigation is carried out for the purpose. The finite element method is used to analyze several cases that would lead to differential settlement. For simplicity soil medium is assumed be linear elastic.

2. Methodology:

Firstly two cases appear below were analyzed to investigate the occurrence of differential settlement. Then the same cases were re-analyzed with piles installed as a means of controlling the differential settlement.

i. A rigid raft foundation over a homogeneous soil stratum under eccentric loading

ii. A rigid raft foundation over a homogeneous soil stratum with inclined base under uniform loading.

Case (1)

Fig.1 shows the finite element mesh used with necessary boundary conditions. The plane strain condition was assumed.

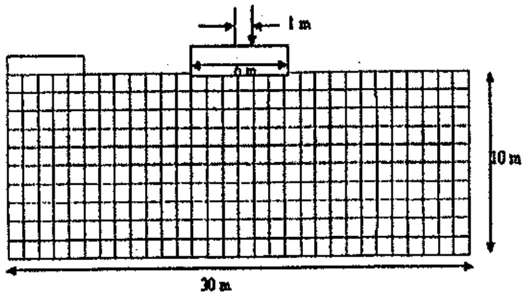

Fig. 1 300 element mesh

This mesh consists of 300 isoparametric elements. Rigid raft foundation of width 6mwas placed over the soil. An eccentric load of 500kN was assumed to act at 1m from the center of the foundation. The load was applied incrementally at a rate of 5kN/day.

The drainage boundary conditions for the domain were taken as follows:

a) Sides PS and QR: drained
b) Sides PQ and RS: undrained

Case (2)

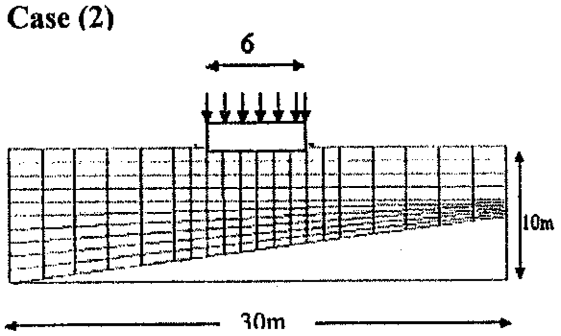

Fig.2 200 element mesh

The Fig 2 shows the finite element mesh used for the soil stratum with inclined base. This mesh consists of 200 isoparametric elements. Rigid raft foundation of width 6m was placed over the soil. The load was applied uniformly over the raft at a rate of $6kN/m^2/day$. The displacement, traction and drainage boundary conditions were as in the previous case.

2.1 Material properties:

The soil was assumed to follow the linear elastic constitutive law. Table 1 shows the material parameter used in this analysis.

Bulk modulus $K/(KN/m^2)$	266.67
Shear modulus $G/(KN/m^2)$	240.00
Poision ratio v	0.25
Permeability /(cm/s)	$3.7*10^{-7}$

Table 1

2.2 Load application:

Load was applied incrementally at a rate of 5kN/day in case (1), $6kN/m^2/day$ in case (2). After reaching the specified total load, the soil was allowed to consolidate. Fig. 3 shows the load application procedure.

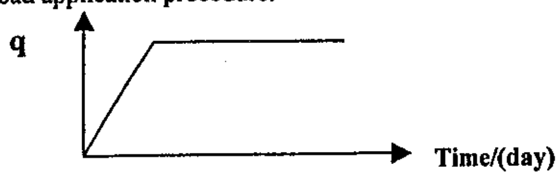

Fig.3 Rate of applied load

2.3 Simulation of the action of a rigid raft:

Assuming infinitely rigid raft foundation, *no length change* and *no angle change* constraint conditions were introduced to the appropriate nodes to simulate the action of rigid raft. Referring Fig.1 and Fig.2 above, for example, nodes 1 and 2 were given the above mentioned constraints during deformation.

2.4 Simulation of the action of a rigid pile:

As in the case of rigid raft, appropriate nodes are subjected to no length change and no angle change constraints to simulate the action of a rigid pile. The pile raft connection can be fixed or pinned. If fixed joint is to be introduced, then the addition constraint condition "*no direction change*" is introduced for the common node of raft and pile.

3. Analysis:

Firstly each case was analyzed without any counter measure for differential settlements to examine the nature of settlement behavior. Next rigid piles were introduced at locations where the settlements were greatest and re-analyzed the problem to check the effectiveness in reducing differential settlement.

3.1 Results and Discussion:

Case1:

Deformed profile at the end of loading and consolidation showed –416.09 mm and –733.85 mm of differential settlement respectively (Fig. 4 and 5) between the two edges of the foundation at the two stages of the deformation process.

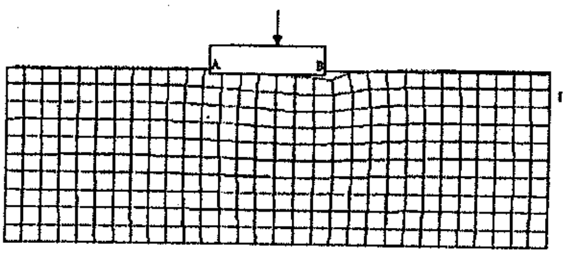

Fig.4 Deformed mesh at the end of loading without pile

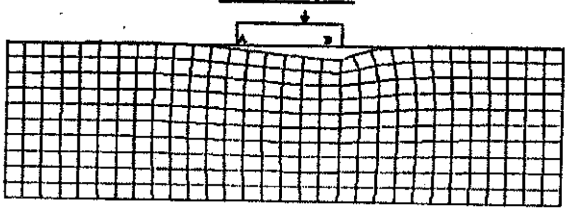

Fig.5 Deformed mesh at the end of consolidation without pile

Since the amount of differential settlement exceeds the allowable limits (for example, 1/300), four rigid piles were introduced in the area where the settlement was greatest (see Fig.6). In order to simulate the fixed pile-raft joint, no angle change condition was introduced for the common nodes. As a result of piling negligible amount of differential settlements (2.26mm) was observed at the end of loading. As expected at the end of consolidation almost equal amount of settlement (differential settlement was 28.26mm) was observed between two edges of the foundation. This indicates the significance of the effect of piles in controlling differential settlement.

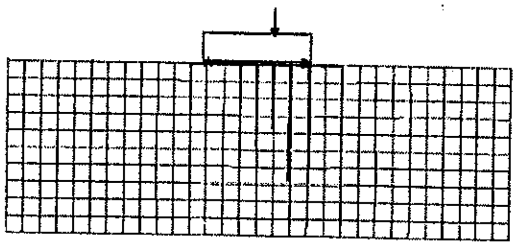

Fig.6 Pile positions

The variation of differential settlement behavior with time is depicted in Fig. 7. In the case of without piles 57% of the total differential settlement has occurred during load application. Thereafter consolidation settlement occurred for a significant time period.

When piles are introduced settlement reduces significantly. Only 8% of total settlement occurred at the end of loading (total differential settlement was 28mm). The rest had occurred within a long time period.

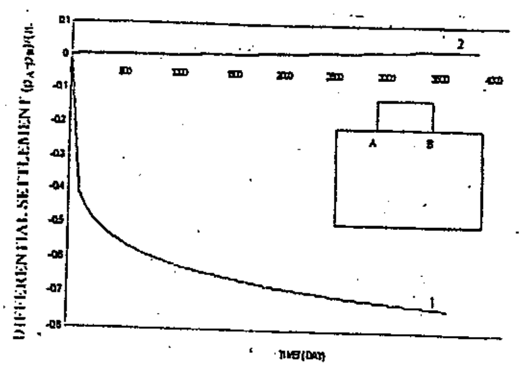

Fig.7.Differential settlement Vs Time

The excess pore water pressure distribution in the case of with and with out piles are shown in Fig.8 and 9. When piles are not installed excess pore water pressure concentration is observed in the area below the point of load application. When piles are introduced the excess pore water pressure concentration is observed approximately towards the pile tips. This suggests that most of load is taken by piles as end bearing.

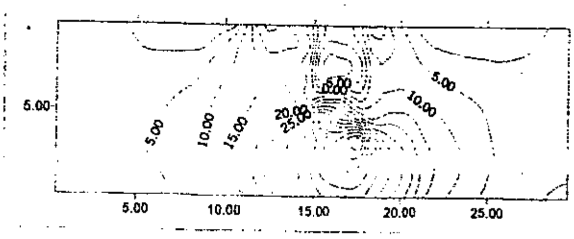

Fig.8 pwp contours at the end of loading without pile

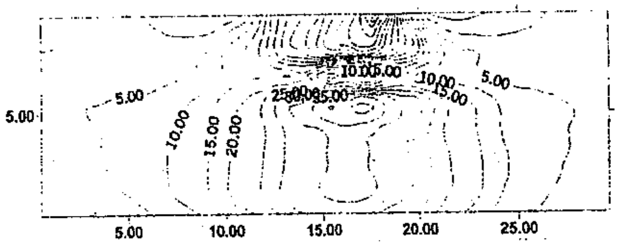

Fig.9 pwp contours at the end of loading with piles

Case2:

Deformed profile at the end of loading and consolidation showed −13 mm and 277 mm of differential settlement (Fig.10 and 11) between the two edges of the foundation at the two stages of the deformation process.

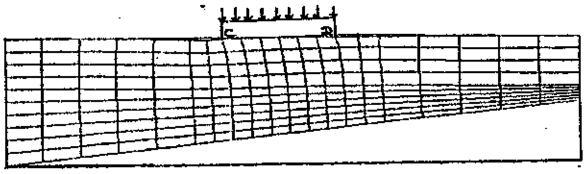

Fig.10 Deformed mesh at the end of loading without pile

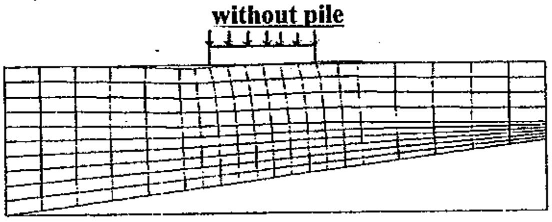

Fig.11 Deformed mesh at the end of consolidation without pile

To minimize the amount of differential settlement within the allowable limits (1/300), three rigid piles were introduced (see Fig. 12) in the area where the settlement was greatest. With the installation of piles there was −19mm and 25mm of differential settlement at the end of loading and end of consolidation respectively. Through this it is clear that there is a significant decrease of differential settlement (202mm) as a result of the installation of piles.

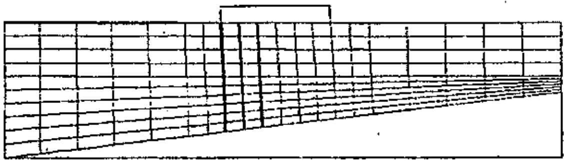

Fig.12 Pile positions

Further analyses were carried out to find out the optimum position of a single pile in a homogeneous soil stratum if the differential settlements are to be reduced. It is found that when the pile is positioned closer to the edge of the raft will reduce the settlement significantly.

4.1 Conclusion:

Through this numerical investigation following conclusions could be drawn.

1. Introduction of piles into the soil-raft reduces the differential settlement significantly.
2. Piles transfer the load deeper into the soil reducing surface settlements.

If a single pile is to be used to control the settlement, positioning it closer to the edge of the raft will be much more effective than any other location.

4.2 Recommendations for further analysis:

1. To investigate the effect of fixity between raft and piles on the settlement behavior.
2. To investigate the settlement behavior under multi layered soils.

5. References:

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