CONFERENCE ON

GROUND IMPROVEMENT TECHNIQUES

Organised by the

SRI LANKAN GEOTECHNICAL SOCIETY

7th May 2002

At Trans Asia Hotel

CONTENTS

Modeling the Consolidation Behavior of Peat and Improvements

W.G.S. Munasinghe, National Building Research Organisation, Sri Lanka
S.A.S. Kulathilake, University of Moratuwa, Sri Lanka

Abstract

Construction on peaty soils is problematic and it is necessary to find cost effective methods for the improvement of peat. Therefore, this research was aimed at studying the response of peats with different levels of humification, on the improvement techniques such as; preloading, and deep mixing with cement and lime. Peats at different levels of humification were used in the study. Peats were mixed in the laboratory at percentages of 5, 10 and 15 with cement and with lime at 15%, on the wet weight, and left to harden for a period of 4 weeks. It showed that preloading caused improvements in all types of peat and deep mixing was effective only for amorphous peat that are of higher degree of humification.

Also, an attempt was made to model the consolidation behavior of peat using Terzaghi's one dimensional consolidation model and Bjerrum's model. A new laboratory set up was developed for the simultaneous observations of settlement and pore water pressure. Observations were compared with the predictions done through the Terzaghi's model and the Bjerrum model. Different methods were used to determine the coefficient of consolidation. Results showed that with the Terzaghi's model level of agreement depends on the method of obtaining the C_v value and the secondary consolidation settlement was not predicted. Bjerrum's model accounted for the secondary consolidation settlement.

1. Background

Very high primary and secondary settlements and very low shear strengths are the major problems associated with peaty soils, which are often found in low-lying areas of Colombo and other suburbs. With the increase of population, demand for land goes up and engineers are compelled to use these lands for future development projects.

Construction on peaty soils leads to stability problems due to its very high compressibility and very low shear strength. Although heavy structures can be built on piled foundations, use of shallow foundations on improved grounds would be more appropriate for light structures such as service lines, light buildings and roads occupying a large plan area.

The most widely used method of improvement for peat is the pre-consolidation by preloading. But, it requires a rather long time period. Deep mixing method mixes the existing soil with a cementiteous material using mixing shafts and nozzles over the full thickness of the layer. Although this method has been successfully used in Japan and Sweden for soft inorganic clays, not much information is available on application in Peaty soils except for few publications in late 1990's from Finland and Sweden. Several research projects have commenced in Sweden and Finland within the last six years. There were laboratory results as well as instrumented field applications (Huttunen et al 1996, Telisic and Leppanen 2000). Considerable improvements in strength were reported for the use of cement / lime and industrial by products such as Blast Furnace Slag. In Sri Lankan peats, organic content is very low at 20% to 30% as compared with values reported in literature. Considering this rather low organic content, the cement and lime mixing has the potential to be a very effective and reliable way of achieving a rapid improvement of engineering properties of Peat. Therefore, study of the possible improvement of consolidation behavior of peat by deep mixing is of vital importance.

This paper presents an attempt to study the consolidation behavior of peat by mixing different percentages of cement or lime with peat at different levels of humification. Peats can be grouped as Amorphous and Fibrous peat. Amorphous peat has a higher degree of humification. The level of humification can vary within a short distance even in the same locality. The improvements of primary and secondary consolidation achieved by cement and lime mixing are compared with the improvements achieved by preloading.

Also, an attempt was made to model the consolidation behavior of both the natural peat and peat improved by cement mixing using the Terzaghi's one dimensional consolidation model (Terzaghi 1925) and Bjerrum model (Bjerrum 1967). The prediction of settlement and pore water pressure behavior done using the Terzaghi's theory and Bjerrum's theory were compared with the experimental observations.

2. Comparison of methods for Improvement of Compressibility of Peat

2.1 Laboratory simulation of pre-loading and Deep Mixing

In order to simulate the preloading process; loading, unloading and reloading increments were applied in the Oedometer. Loading increments represent the untreated peat and the reloading increments represent the Preloaded Peat. Improvements achieved at similar stress levels were compared. Load increments of two-week duration were used in view of the dominant secondary consolidation in Peat.

To simulate the site condition in the deep mixing method, peat was remoulded in the laboratory by mixing with cement and lime using an electrical hand mixer over the same time period, at the same rotating speed and they were left to harden in buckets for a period of four weeks under water. Consolidation tests were conducted on undisturbed samples obtained from the treated peat.

With types of peat shown in Table 01, mixing was done with the addition of 5%, 10%, and 15% cement and 15% lime. Above mix proportions were decided based on the results of some initial tests.

Place	Type	Natural Moisture Content	Organic Content	PH Value	Specific Gravity	Initial Void Ratio
Wattala	Amorphous Granular Peat	387.00	29.00	2.60	2.23	8.63
Madiwela	Fibrous Peat	297.00	34.80	2.99	1.87	5.55

Table 01: Basic properties of selected Peat

2.2 Improvement of Primary consolidation characteristics of Peat

Improvement of primary consolidation characteristics were studied by comparing the parameters; coefficient of volume compressibility - m_v, compression index - C_c, recompression Index - C_r, compression ratio $C_c/(1+e_o)$ and recompression ratio $C_r/(1+e_o)$.

The effect of preloading and deep mixing with cement and lime on the coefficient of volume compressibility m_v is illustrated in Fig. 1 and Fig. 2 for Wattala Amorphous Peat and Madiwela Fibrous peat respectively. Both peats possessed similar organic contents and void ratios but the degree of humification was much higher in Wattala Peat. It is evident from Figure 1, that due to the mixing of 5% cement by weight, Wattala amorphous peat experienced a significant reduction of m_v, that was of the same order as the improvement achieved by preloading. With the fibrous Madiwela peat, although the preloading has caused a reduction in m_v, even the mixing of 15% cement or 15% lime has not caused much improvement in the coefficient of volume compressibility as illustrated by Fig. 2.

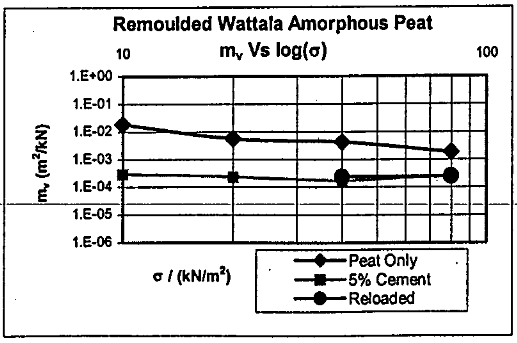


Figure 01:Effect on m_v for Wattala Amorphous Peat

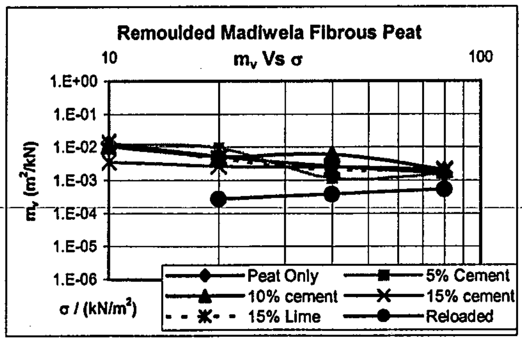


Figure 02: Effect on m_v for Madiwela Fibrous Peat

Alternatively, improvements achieved in primary consolidation characteristics can be illustrated with e vs log σ plot. It is evident from e vs log σ graphs corresponding to natural peat for Wattala Amorphous peat (Fig. 3) and Madiwela Fibrous Peat (Fig. 4), that re-compression index C_r is much smaller than the (less than 8%) compression index C_c. Thus, it is clear that the preloading can cause a significant improvement in primary consolidation characteristics in both types of peat. The C_c value corresponding to 5% cement mixed peat in Fig. 3 is of the same order as the C_r for the preloaded stage for Amorphous Wattala peat thus indicating that similar levels of improvements were achieved. However, as illustrated by figure 04 mixing of different percentage of cement and lime has not caused much of a reduction in the C_c in the Madiwela Fibrous peat. This is further illustrated by the C_c / $(1+e_o)$ and C_r / $(1+e_o)$ values computed for different conditions reported in Table 02.

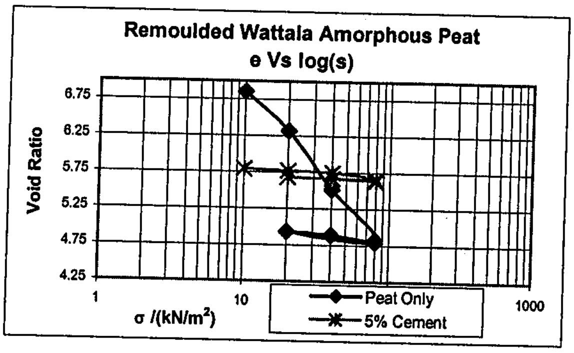


Figure 03:Comparison of e vs log σ plots Wattala Peat

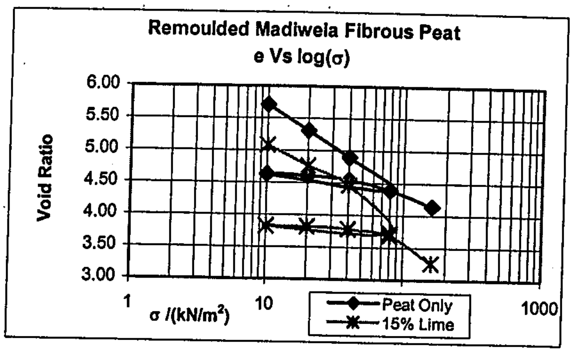


Figure 04:Comparison of e vs log σ plots Madiwela Fibrous Peat

Peat Type	Method of Improvement	Curing Time Period (wks)	$C_c/(1+e_o)$ or $C_r/(1+e_o)$
Amorphous Peat - Wattala	Non Improved	-	0.3644
	Reloaded	-	0.0527
	5% cement mixed	2	0.0365
Fibrous Peat - Madiwela	Non Improved	-	0.2516
	Reloaded	-	0.0962
	5% cement mixed	4	0.2625
	10% cement mixed	4	0.3717
	15% cement mixed	4	0.1831
	15% lime mixed	4	0.3979

Table 02: C_c / $(1+e_o)$ or C_r / $1+e_o)$ values for non improved and improved peat

2.3 Improvement of secondary consolidation characteristics of peat

The effect of the improvement methods on secondary consolidation characteristics was assessed by comparing the coefficient of secondary consolidation-c_α and its variation with time and the stress level. The variation of C_α values with stress level is plotted for Wattala amorphous peat and for Wattala peat mixed with 5% cement in Fig. 5 and it is evident from these plots that the mixing of 5% cement has caused a significant reduction in C_α. This reduction is of the same order as the reduction achieved through preloading.

Test results presented in Fig. 6 show that mixing of cement or lime has not caused much improvement in the C_α values of Madiwela fibrous peat while the C_α values for the reloading increments were much smaller. Thus, it is clear that the preloading will cause an improvement in secondary consolidation characteristics even in a fibrous peat.

3.0 Modeling the Consolidation Behavior of Improved and Natural Peat

3.1 Importance of measuring pore water pressure

Since the primary consolidation settlements are accompanied by the dissipation of excess pore water pressure, any laboratory consolidation tests conducted to model the consolidation behavior of a soil should be ideally done with the measurements of both the settlements and the pore water pressures. In the conventional laboratory consolidation test, only the settlements are measured. As such, a laboratory setup was developed to conduct the consolidation tests with simultaneous measurement of pore water pressure and settlement.

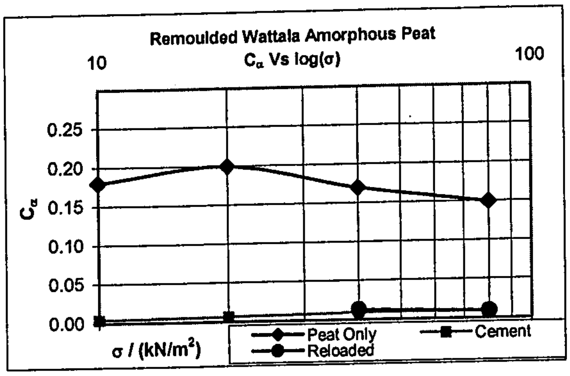


Figure 05: C vs log plot for Wattala Amorphous Peat

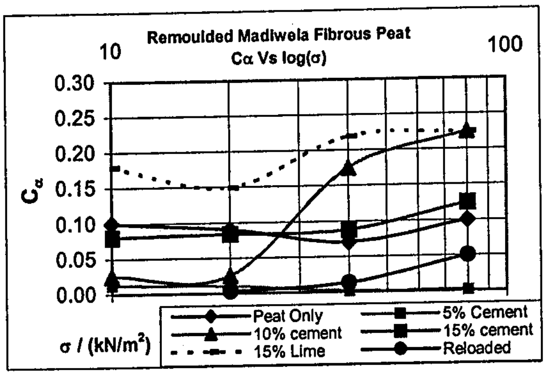


Figure 06: C vs log plot for Madiwela Fibrous Peat

3.2 Development of a Laboratory test setup for simultaneous measurement of pore water pressure and settlement

In the new laboratory setup, a GI pipe of 69.5mm diameter and the height of 85mm was used as the consolidation ring. Drained conditions were provided at the upper boundary of the sample while undrained conditions were established at the bottom boundary. A plate containing four holes to facilitate drainage was used on top of the sample with a porous plate and the bottom plate consist of one hole at the center to be connected to the pore water pressure measuring arrangement. Top plate and the top porous plate were made to a smaller diameter facilitating the free movements inside the cylinder and the bottom plate was made water tight by fixing two rubber "O" rings around. The bottom plate has a threaded end to facilitate fixing to the triaxial base (Fig. 8 (a)). The pore water pressure measurement was done through the diaphragm system that was used to measure the pore water pressure in the triaxial setup. In these tests, vertical loading was applied directly on the sample using a loading frame (Fig. 8 (b)). Complete details of the laboratory setup are discussed in Munasinghe (2001).

3.3 Testing Procedure and Results

The sample was subjected to a vertical stress of 40 kN/m^2 using the loading hanger. Two soil specimen; specimen of Madiwela Peat remoulded with 10% cement and a specimen of Madiwela peat remoulded without addition of any cement were used for the tests.

The major shortcome seen with pore water pressure measuring system was the slow response time. It was minimized as far as possible by reducing the length of the connecting tubes. However, with all that efforts the maximum pore water pressure was developed only after 4 minutes. Fig. 9 presents the graphs for remoulded natural Madiwela peat. The graph for the peat with 10% cement is presented in Fig. 10. Results shows that the excess pore water pressure has dissipated completely within a 200-400 minutes period. Therefore, it can be deduced that primary consolidation has finished by that time. But the settlements continued even after the 100% pore water pressure dissipation. This is due to the secondary consolidation.

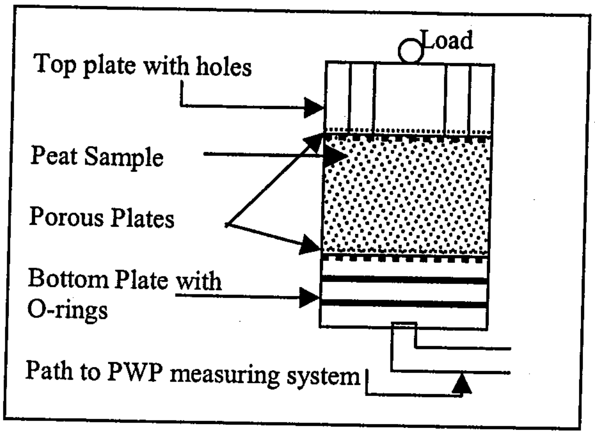


Figure 07: Schematic Diagram of the New Laboratory setup

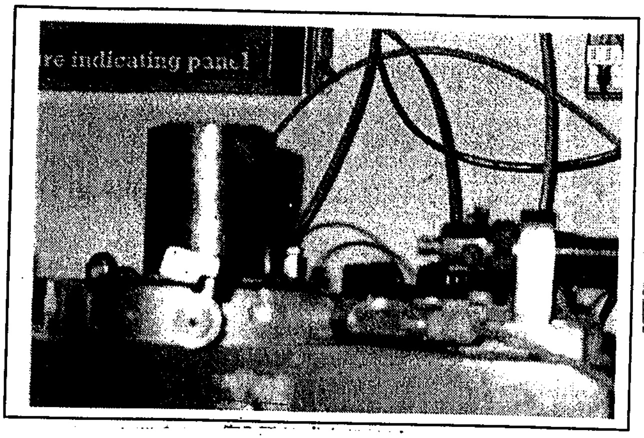


Figure 08 (a): Peat Sample with the triaxial base

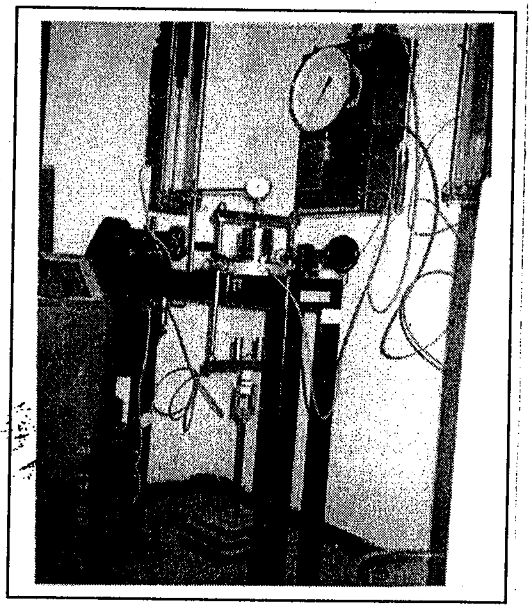


Figure 08 (b): Complete Laboratory Set up

3.4 Back analysis of the test results using Terzaghi model

Although there are some limitations, Terzaghi's one dimensional consolidation theory is still widely used to model the consolidation behavior of clays. As such, initially an attempt was made to model the consolidation behavior of both untreated peat and cement treated peat using the Terzaghi model. Under the boundary conditions prevailing in the test setup, the pore water pressure at the bottom of the sample is given by;

$$u = \sum_{n=0}^{\infty}\left[\frac{2u_0}{M}.Sin\frac{MZ}{H}.e^{\left(-M^2T_v\right)}\right]$$

Where $M = \frac{\pi}{2}(2n+1)$ $\qquad T_v = \frac{C_v t}{H^2}$

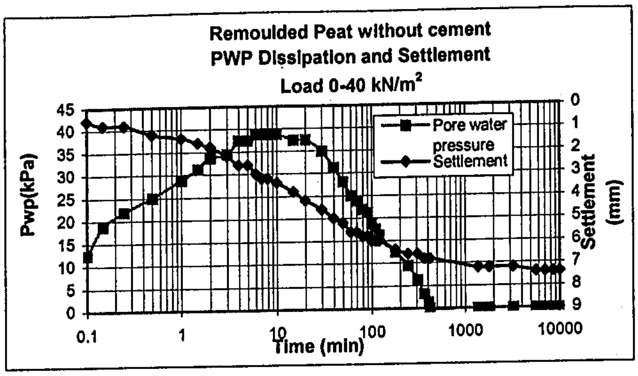


Fig 09:Variation of Settlement and Excess pore water pressure for Madiwela Peat without cement

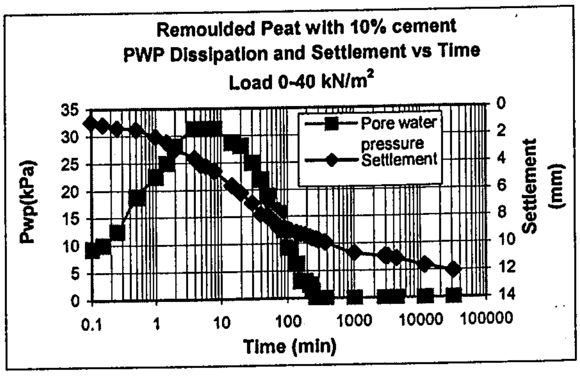


Fig 10: Variation of Settlement and Excess pore water pressure for Madiwela Peat with 10% cement

The above expression is evaluated by adding up the values in the series from n = 0 to 10 (first ten terms) using an EXEL worksheet and the variation of the pore water pressure u with time is computed. (It was shown that the addition of first ten terms is adequate). The average degree of consolidation of the specimen can be expressed in terms of the pore water pressure by;

$$U = \frac{\left(u_0 - \bar{u}\right)}{u_0} = 1 - \frac{\bar{u}}{u_0} = 1 - \sum_{n=0}^{\infty} \frac{2}{M^2} e^{\left(-M^2 T_v\right)}$$

The settlement δ_t at a given time t can be expressed as $\delta_t = \delta * U$, where δ is the ultimate settlement at, 100% pore water dissipation. Another EXEL spreadsheet was developed to obtain δ_t variation with time; by adding first ten terms of the expression. Further details are presented in Munasinghe (2001).

The C_v value required to model the test results was found by three methods. In the first method value of C_v was obtained using the time taken for 50% primary consolidation settlement of the sample. The settlement at 100% excess pore water pressure dissipation was found and time required for 50% of that settlement was taken as t_{50}. Thereafter, C_v was calculated using the time factor of 0.197 at 50% average degree of consolidation. This yielded a C_v value of 15.64 mm^2/min.

The pore water pressure and settlements computed are compared in Fig. 11 and Fig. 12 respectively for Madiwela untreated peat. It shows that the settlement behavior up to end of primary is reasonably well predicted. As expected the secondary consolidation settlements were not predicted by the Terzaghi model computations. The prediction of the excess pore water pressure dissipation is not satisfactory. Madiwela peat treated with 10% cement also exhibited a similar behavior.

In the second method, C_v was obtained using the time taken for 50% excess pore water pressure dissipation at the undrained bottom boundary. The C_v value obtained using isochrones for 50% consolidation at the bottom undrained boundary was 3.02 mm^2/min for Madiwela untreated peat. Fig. 13 and Fig.14 show the comparison of pore water pressure and settlement thus computed and measured values respectively for Madiwela fibrous peat treated with 10% cement in this manner. The pore water pressure behavior is reasonably predicted and the prediction of settlement is not satisfactory. From Fig. 14 also it can be clearly shown that the Terzaghi's one dimensional consolidation theory does not include the secondary consolidation settlement. Also, at the initial stages, it is seen that the measured settlement is higher than that computed. Similar observations were made by Mesri et al (1997). Therefore, it is evident that the Terzaghi model is not ideally suited to model the consolidation behavior of either untreated or treated peat.

In the third method, C_v was obtained through log δ – log t method (Sridharan and Prakash 1997). $t_{88.3}$ was taken as time which corresponds to the point of intersection of the two straight lines identified. Thereafter, C_v was computed using T_v=0.793 (at U = 88.3%) and the value obtained is 9.688 mm^2 / min for Madiwela natural peat. As in the case of two previous methods, Fig. 15 and Fig.16 are the two graphs presenting observed and calculated pore water pressure and settlement using Terzaghi's one-dimensional consolidation theory. As in the earlier cases, Fig. 16 shows that the settlement behavior is reasonably predicted except for the secondary consolidation. However, the pore water pressure distribution is not modeled satisfactorily.

Thus, it can be said that when the coefficient of consolidation is computed based on a settlement-based method, the value of the coefficient of consolidation obtained is much larger. The pore water pressures computed using this C_v value are much lower than what is observed. i.e. the predicted pore water pressure dissipation is more rapid.

When the coefficient of consolidation was computed based on the measured pore water pressure, the values obtained were much smaller. Hence, the computed settlement values are much lower than the observed values. Therefore, it can be said that settlements are taking place more rapidly than the dissipation of pore water pressure. It may be possible that some secondary consolidation type settlements are also taking place prior to the complete dissipation of pore water pressure. However, it is a clear indication that the Terzaghi theory is not ideally suited to model the consolidation behavior of peat.

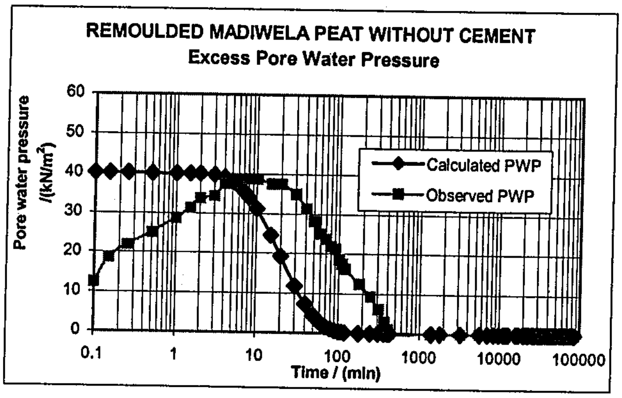


Fig 11:Calculated and Observed PWP using C_v -method 1

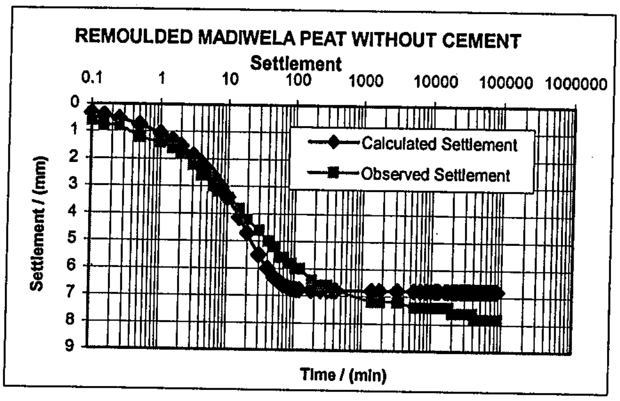


Fig 12:Calculated and Observed settlement using C_v-method 1

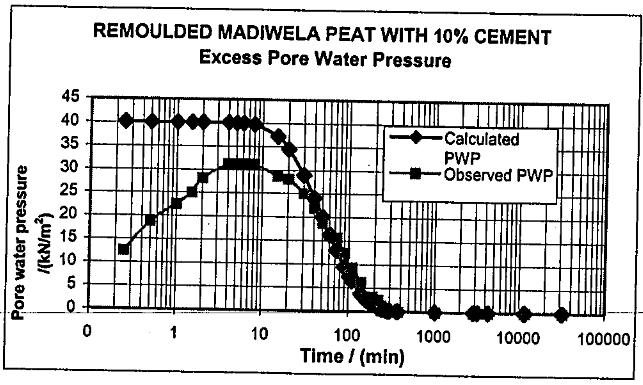


Fig 13:Calculated and Observed PWP using C_v-method 2

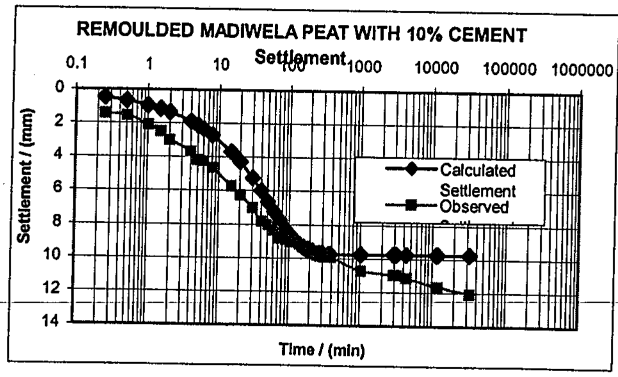


Fig 14:Calculated and Observed settlement using C_v-method 2

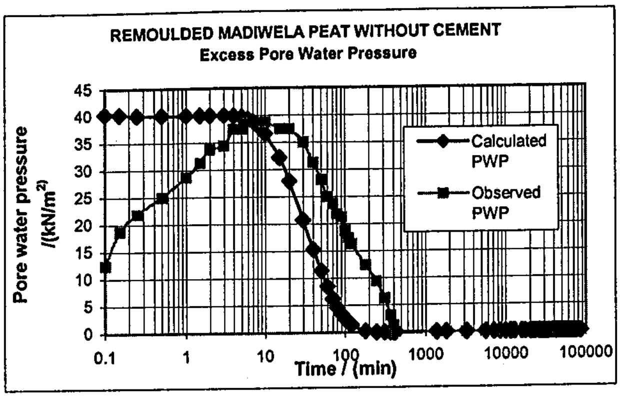


Fig 15:Calculated and Observed PWP using C_v-method 3

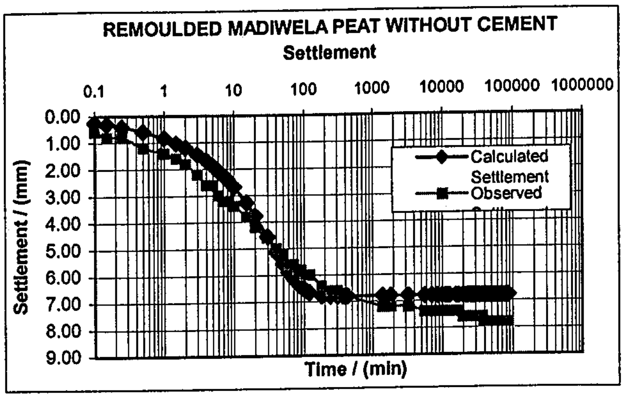


Fig 16:Calculated and Observed settlement using C_v-method 3

3.5 Use of Bjerrum Model

Bjerrum model includes the secondary consolidation settlement and the settlement is given by;

$$\delta = \delta_{primary} + \delta_{secondary}$$

Based on the Bjerrum's model of the consolidation behavior, for loading above the pre-consolidation pressure p_c', the compression at time t can be computed from

$$\Delta e = C_r \log (p_c / \sigma_o) + C_c \log (\sigma_f / p_c) + C_\alpha \log ((t + t_1) / t_1)$$

The first two terms account for primary consolidation settlement and the last term corresponds to the secondary consolidation settlement. The model already developed based on the Terzaghi model accounts for the primary consolidation settlement. Hence, it is necessary to add terms corresponding to secondary consolidation,

$$\delta_{secondary} = (C_\alpha \ / (1+e_p))^* H \ * \log (t_2/t_1)$$

where e_p is the void ratio at 100% primary consolidation and C_α was obtained through the e vs log (t) curve. Fig. 17 presents the computed and observed settlement curves for Madiwela peat treated with 10% cement. The computed settlement includes both primary consolidation settlement obtained through Terzaghi model and the secondary consolidation settlement using the above expression. C_v value was obtained through method 1. The Figure shows that the settlement behavior is reasonably well predicted according to the Bjerrum model when the C_v was computed based on the settlement.

The comparison when C_v computed by method 2 was used in the computation is shown in Fig. 18. The computed pore water pressures for the two cases are same as in Fig. 11 and Fig. 13.

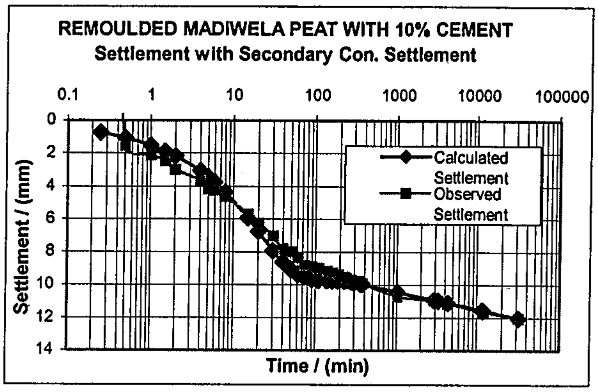


Fig. 17 Observed and Computed δ for Madiwela Peat with 10% cement using Bjerrum Model – Cv method 1

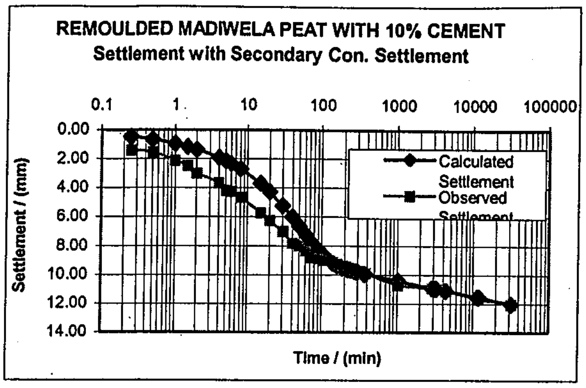


Fig. 17 Observed and Computed δ for Madiwela Peat with 10% cement using Bjerrum Model – Cv method 2

4.0 Conclusions

It can be concluded that preloading causes significant improvements in both primary and secondary consolidation properties on all types of peat that are at different levels of humification. Deep mixing with cement or lime has an ability to improve the stiffness within a period as short as 4 weeks in Amorphous Peat.

However, this method was successful only for amorphous peats that are with a high level of humification. Improvements achieved in primary and secondary consolidation properties were of the same order as that achieved during preloading. Fibrous peats did not show any improvements in either primary or secondary consolidation characteristics even after mixing of 15% of cement.

Consolidation tests conducted with pore water pressure measurements revealed that settlements continue even after the full dissipation of excess pore water pressures. It is clearly shown that the secondary consolidation settlements were not predicted by the Terzaghi Model. Back analysis of Terzaghi's theory showed that the prediction of settlement or pore water pressure depended on the method of computation of C_v value.

Actual settlements appear to be occurring more rapidly than the dissipation of pore water pressure, as predicted by Terzaghi theory. It may be possible that some secondary consolidation take place prior to the complete dissipation of excess pore water pressures. Thus, the Terzaghi model is not ideally suited to model the consolidation behavior of peat. The Bjerrum model accounts for secondary consolidation and models the settlement satisfactory when the computations are done using a C_v computed based on a settlement approach. When the computations are done with a C_v based on pore water pressure dissipation (method 2) agreement of settlement is not satisfactory. Thus, it is clear that even the Bjerrum model is not ideally suited for Peaty Clays.

Acknowledgements:
We greatly appreciate the kind support of the management of NBRO for providing the laboratory facilities and the laboratory staff in University of Moratuwa, NBRO and RDA.

References:

(a) Bjerrrum L. (1967) - " Seventh Rankine Lecture"; Engineering Geology of Norwegian normally consolidated marine clays related to settlements of buildings. Geotechnique vol.22, No.2, pp.82-118.

(b) Huttunen, E., Kujala, K., and Vesa, H., (1996) - Assessment of the quality of stabilized peat and clay. Procedings of the International Conference on grouting and Deep Mixing (in Japan)

(c) Mesri G., Stark T. D., Ajlouni M. A., Chen C.S. (1997) - Secondary Compression of Peat with or without Surcharging, Journal of Geotechnical and Geoenvironmental Engineering. Vol. No.05

(d) Munasinghe W.G.S. (2001) – Methods for Improvement of Engineering Properties of peat – A Comparative Study, M. Eng Theisis, University of Moratuwa

(e) Sridharan A. and Prakash K. (1997) - Proceeding for Institute of Civil Engineers' ; Geotechnical Engineering. Vol. 125, pp. 27-32

(f) Terzaghi. K. (1925) – Erdbaumecchanik auf bodenphysikalischer Grunlage.Leipzig: Deuticke.

Improvement of Engineering Properties of Peat by Preconsolidation -A Comparison of Field and Laboratory Test Results

Asiri Karunawardena, National Building Research Organisation,Sri Lanka.
S.A.S . Kulatilaka, University of Moratuwa, Sri Lanka.

Abstract

Preconsolidation by preloading can be used to improve the Engineering properties of peat. Simulations of the preloading process condition in the laboratory samples of Sri Lankan peaty clays indicate the level of improvements that can be achieved.
With a highly none homogeneous material like peat it is essential to verify these findings with appropriate field measurements. This paper presents an attempt to compare the improvements in primary and secondary consolidation characteristics and shear strength properties obtained through laboratory tests, with those obtained from a monitored fill in the Madiwela Government project. Improvements observed in the field are seen to be comparable with those achieved in the laboratory.
Field consolidation due to the gradual placement of the fill was modeled using the Finite Difference method and comparisons were made with the observed Pore water Pressure and Settlements. The laboratory determined c_v value had to be increased four times to match the field behavior.

1. BACKGROUND AND OBJECTIVE

Presence of soft, highly compressible peat in layers of large thickness is a major problem encountered by Geotechnical Engineers involved in new infrastructure and housing development. Peat layers of thickness ranging from 5m to 13m are present in some of the lands available for development in the city of Colombo and its suburbs. Therefore, it is very important to develop cost effective methods for handling such conditions.

If multistoried buildings are to be constructed in areas underlain by thick layers of peat, the large structural loads will have to be transferred to an underlying dense layer or rock through a system of piled foundations. However, it would not be economical to transfer the moderate loads imposed by services such as water supply lines and sewerage lines also to an underlying hard stratum through piles. Furthermore, it would not be economical to construct new infrastructure facilities with large plan area such as roads, on piled foundations. It is most economical to transfer such moderate loads to the soft peat layers, after improving their strength and stiffness to an appropriate level.

Numbers of different approaches are in use around the world for the improvement of soft peat deposits. Pre-consolidation by preloading of the peat with a surcharge is one such method that could be carried out without the help of any special machinery.
Laboratory tests conducted to assess the improvement of strength and stiffness of peat through pre-consolidation had shown that the above-mentioned properties could be significantly improved through preconsolidation. With a highly non-homogeneous formation like peat, it is extremely important to verify the laboratory findings with field measurement by constructing a trial fill where a large mass of peat is subjected to a significant increase of stress. Such a study was done in the Fill Area 2, of the Madiwela Government project. Figure 1 presents the plan of the instrumented fill area, with locations of auger holes boreholes, settlement plates and piezometers.

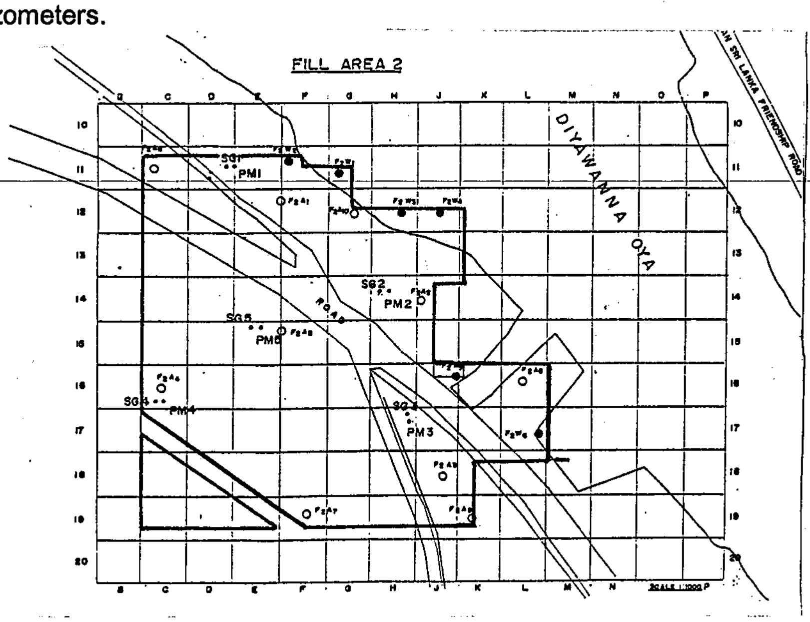


FIGURE. 1. SITE PLAN OF THE INSTRUMENTED FILL AREA.

2. INITIAL SUBSOIL CONDITION AT THE SITE

A comprehensive subsoil investigation program was carried out in order to establish the relevant soil parameters. The subsoil condition at the instrumented fill area was investigated with 3 boreholes and 8 auger holes. The vertical soil profile deduced from the borehole investigation is presented in Figure 2. It revealed that there exists a compressible peat layer underneath and the thickness of this layer was approximately 5m to 6m. The average physical properties of this layer are indicated in the Figure 2.

Standard Penetration Tests were carried out at 1m intervals in order to obtain a continuous strength profile of the subsoil. The data obtained are summarized in figure 3. The zero S.P.T value indicated the existence of soft soil up to about 5m. Field vane shear tests were carried out at number of locations at different depths to establish the initial shear strength. Based on the data of vane shear tests an average undrain cohesion (c_u) in the peat layer is found to be around 10 kN/m^2.

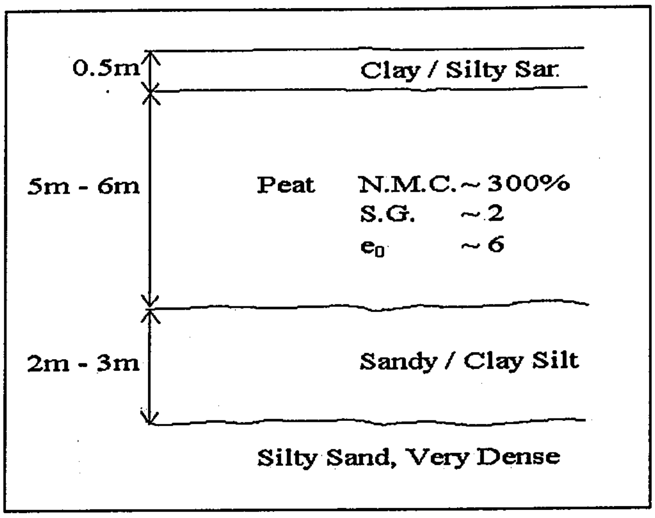


Figure 2. Typical soil profile at the site

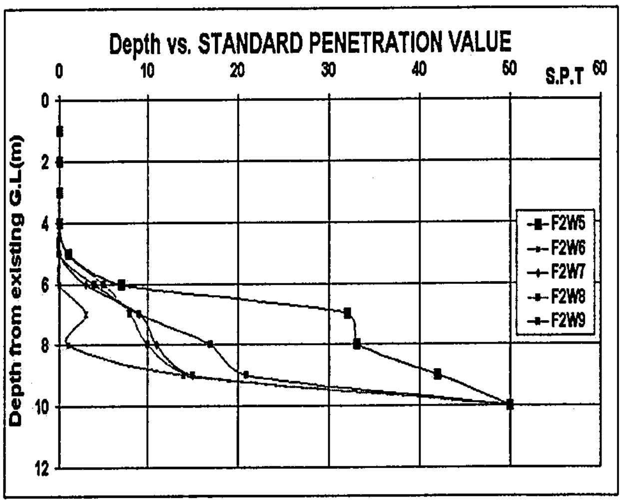


Figure 3. S.P.T. values obtained at different boreholes

3. IMPROVEMENT ACHIVED IN THE ENGINEERING PROPERTIES.

Undisturbed soil sample were taken prior to any filling and specimens were subjected to consolidation tests and Unconsolidated Undrained triaxial tests under different cell pressures in order to establish the initial Engineering properties. Test procedures were developed, for the laboratory simulation of the consolidation of the peat due to the weight of the fill. Subsequently, once the filling was completed and once the peat has experienced some consolidated under the weight of the fill, undisturbed soil samples were taken again and were subjected to a similar series of tests. Improvements achieved under laboratory conditions by simulated testing were compared with the properties established by the samples obtained after the consolidation of the peat in the field.

(a) Improvements in Compression Index

Preloading process was simulated by conducting the consolidation tests with the loading/unloading and reloading Increments in the laboratory. The initial loading increments simulated the natural peat and the reloading increments up to maximum previous consolidated pressure simulated the preloaded peat.
Improvements achieved in primary consolidation were compared using compression index (c_c) and coefficient of volume compressibility (m_v) in each phase. It was evident from the results of the simulated tests that both parameters were reduced up to around 10% due to the preloading. A void ratio vs. log (σ) plot obtained during the simulated testing of a sample obtained before the filling is presented in Figure 4.

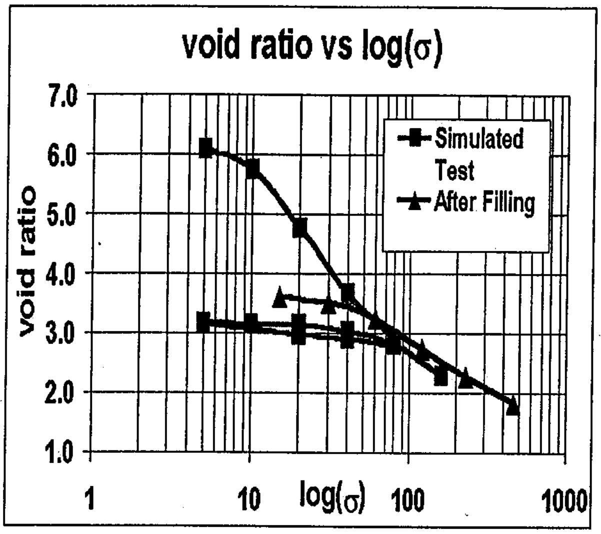


Figure 4.Graph showing reduction of the compressibility due to preloading

	Before Filling	After Filling
Bulk Density(g/cm^3)	1.12	1.31
Moisture Content (%)	307.8	176.2
Specific Gravity	2.00	2.20
Compression Index	3.955	0.397*
Recompression Index	0.265	-
Pre consolidation Pressure (kN/m^2)	15	50

*compression index up to the maximum preconsolidated pressure

Table.1 Details of the soil parameters obtained from the samples obtained before and after filling.

In the field, the peat layer was consolidated under the weight of the fill placed. Therefore, consolidation tests were conducted on the undisturbed samples obtained from the consolidated peat. This sample was taken 260 days after the completion of the fill and is referred to here as the "after filling sample". The gradient of the void (e) vs. log (σ) plot up to the preconsolidated pressure Pc of the "after filling sample" should provide an assessment of the compressibility of the preloaded peat. This should be compared with the gradient of the reloading curve obtained during the simulated testing. Figure 4 compares the void ratio (e) vs. log (σ) plot for the above two cases. It could be seen that the gradient of the reloading curve in the simulated testing and the initial gradient from the test on the "after filling sample" are quite close. The two gradients are 0.265 and 0.397 respectively. This is a significant reduction compared to the c_c value of 3.952 of the untreated peat. However the gradient of the "after filling sample" is slightly greater. As confirmed later, this is an indication that complete consolidation has not taken place under the fill in the field.

(b) Observation on preconsolidation pressure

According to the data from the after filing sample the preconsolidation pressure is around 50 kN/m^2. The preconsolidation pressure of the natural untreated peat is around 15 kN/m^2. If the peat has completely consolidated under the weight of the fill which was around 46 kN/m^2, the preconsolidation should have been shown as 61.0 kN/m^2. The lower preconsolidation pressure also hints that the field consolidation is not complete.

(c) Improvements in Coefficient of Volume Compressibility

Coefficient of volume compressibility (m_v) obtained during the reloading phase of the simulated testing were compared with the values of coefficient of volume compressibility observed in the loading increments up to the preconsolidated pressure of the "after filling sample". (Figure 5)
It shows that the observed coefficients of volume compressibility up to the preconsolidation pressure are of same order but slightly greater than the values obtained in the reloading phase of the simulated testing. (3rd and 4th points in the two graphs)
The values had been reduced to about 10 % of that of the untreated peat.

(d) Improvements in Coefficient of Secondary Consolidation

Results of the simulated tests indicated that the coefficient of secondary consolidation (c_α) obtained during the loading have reduced to about 10% in the reloading stages. Results are presented in Figure 6. Also it can be seen that the observed c_α values; up to the preconsolidated pressure of "after filling sample" and in the reloading phase of the simulated tests are also of same order, but slightly greater. This is again an indication that the field consolidation had not been completed at the time the undisturbed samples were taken.

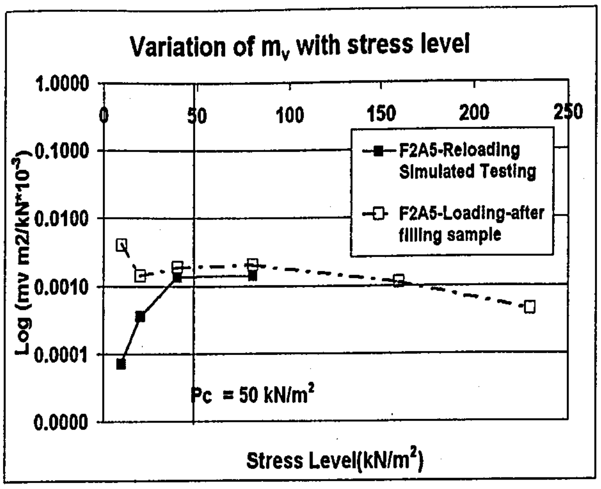


Figure. 5 Comparison of Coefficient of Volume compressibility of "after filling sample" and "simulated testing"

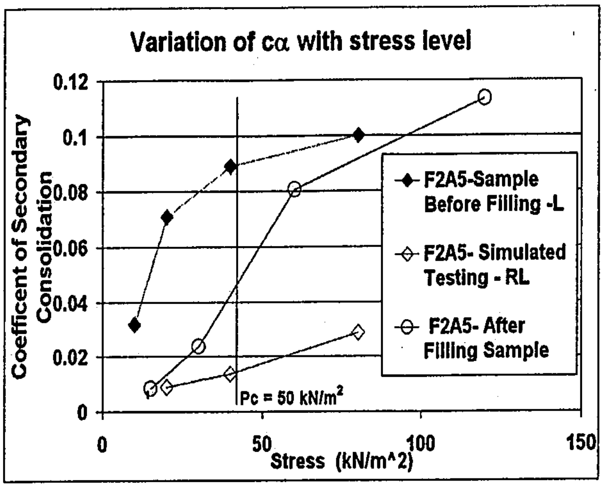


Figure. 6 Comparison of Coefficient of Secondary consolidation of "after filling sample" and "simulated testing"

(d) Improvements in Undrained Shear Strength

Shear strength gain due to consolidation of peat was studied through consolidated undrained tests. Several identical specimens were isotropically consolidated at different cell pressures and subjected to deviator loads under undrained conditions. The change in deviator load at failure due to the change in consolidation pressure was used to obtain the $\Delta c_u/\Delta\sigma$ ratio and it is around 0.21 for this site. Similar results were reported in Kulatilaka (1999).

Undrained shear strength obtained from the samples tested after filling was in the range of 18 kN/m^2 to 21 kN/m^2, an average increase of around 10 kN/m^2 was observed which is equal 0.21 to 0.24 times the weight of the fill (46 kN/m^2). This is approximately equal to the predicted laboratory improvement.

Field vane shear tests were carried out at number of locations at different depths to establish the initial insitu shear strength. Undrained Cohesion c_u values obtained varied from 9 kN/m^2 near the surface to 13.5 kN/m^2 at depths of 4m. The average undrained cohesion for the peat layer was estimated to be 10 kN/m^2. Sometimes after the filling was completed, another vane shear program was carried out to assess the insitu gain in the shear strength. The average undrained cohesion of the peat layer varied from around 16kN/m^2 to just below the fill to around 21 kN/m^2 at depth of 3m below the bottom of the fill. The average undrained cohesion value was estimated to be 20kN/m^2.

The strength improvement noted 10kN/m^2 is approximately equal to 0.2 times the weight of the fill 46 kN/m^2. However, there is evidence to say the consolidation is not complete in the field. Therefore with further consolidation, further improvements in shear strength can be expected. As such α = 0.2 can be taken as a lower bound for the Sri Lankan peaty clays.

4.0 Monitoring of Field Behavior

Traditionally, the rate of consolidation in the field is estimated with the use of coefficient of consolidation c_v determined through laboratory tests. Number of research publications (Wojeieoh et al (1988)) have reported that the field rate of consolidation can be much faster than the rate predicted through the laboratory results and therefore the field rate of consolidation should be obtained by independent field monitoring of the pore pressure and the settlements. Pizometers were installed in the middle of the peat layer and the pore pressure development in the peat layer with the placement of the further fill and the subsequent dissipation with time was monitored daily. Also, settlement gauges were placed just above the peat layer after the placement of a nominal fill of thickness 300mm. Settlement of the peat during further filling and during the consolidation phase was captured through these settlement gauges. The observed pore water pressure and settlement due to filling are presented in Figure 7.

The monitoring of both the settlement and pore pressure independently provided important information about both the primary and the secondary consolidation characteristics of peats.

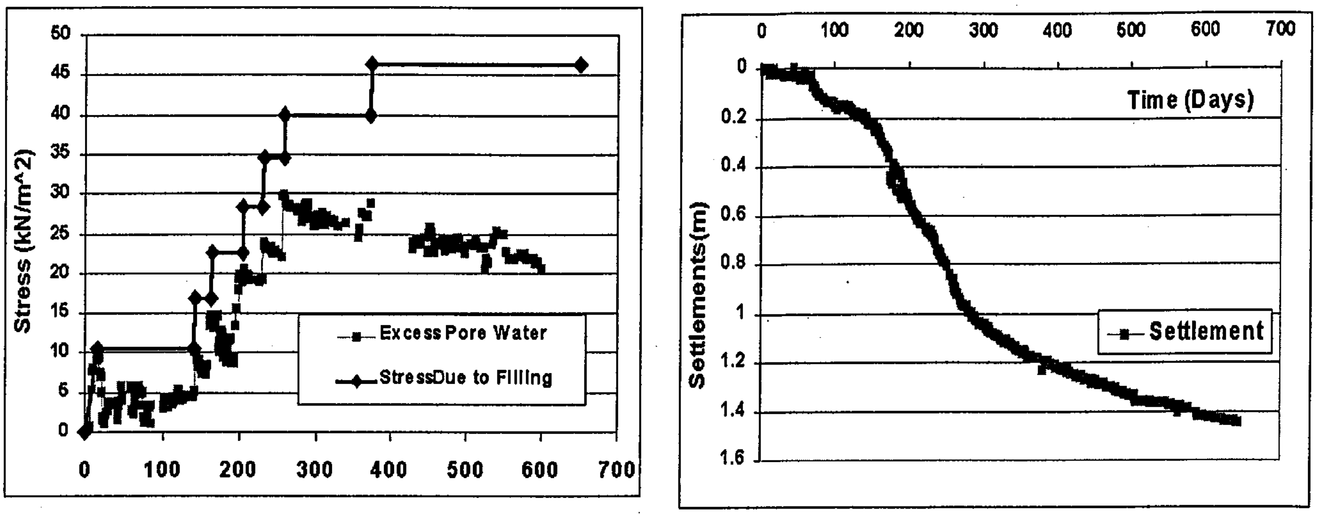


Figure. 7. Field monitoring Data

The fill was placed and compacted in layers under controlled conditions. Fill material was placed in layers not exceeding 300mm thickness in loose state and was compacted at each lift to not less than 95% of the standard proctor density.

5.0 Modeling field behavior and comparison with actual observation

The prediction of the field rate of consolidation is a very important part of a preloading project. The prediction of the field behavior is mainly based on the coefficient of consolidation determined from the laboratory consolidation test. Often the actual observed rate of settlement is much higher than the rate predicted on the basis of the values obtained using undisturbed samples. The possibility of 3D consolidation in the field and the small soil sample being unrepresentative presence of fissures and permeable intrusions in the field peat layer, might be the reasons for this difference. Also, the time required to construct the compacted fill to its final height is fairly long and may be comparable with the time required for the consolidation of the soil layer. Thus, during the gradual building up of the embankment, consolidation occurs simultaneous with the increase of the fill height. Therefore, it is necessary to model this real behavior to obtain a good prediction of the field settlements and the pore pressure response.

In this research project a finite difference model was developed for the prediction of field behavior. Also, the predicted field behavior was compared with the actual rate of consolidation, which was observed through the independent measurement of pore pressure, and the cumulative measurement of settlements in the filed. Finally, when the predicted field behavior using lab values was found to be quite different from the actual observed behavior, fresh prediction was made by changing the lab parameters till the observations are well matched.

5.1 Modeling of Field Behavior using Finite Difference Method.

The idealized section of the sub soil condition deduced from the boreholes investigation is presented in Figure 8.The parameters required for analysis were obtained from the lab tests.

5.1.1 Modeling the pore pressure response

It is done by using the following equation describing practical situation in which the external stresses causing consolidation progressively changes with time;

$$c_v \frac{\partial^2 u}{\partial y^2} = \frac{\partial u}{\partial t} - \frac{\partial \sigma}{\partial t}$$

where

u = excess pore water pressure   y = distance to the point

t = time σ = external pressure

c_v = coefficient of consolidation

A Finite Difference model was developed with an appropriate mesh (figure 9) and excess pore pressures are calculated for different boundary conditions, following the procedures outlined in Lee et al. (1982) and Das

(1985). The top boundary (lateratic gravelly clay layer) was taken to be undrained and the bottom boundary is taken to be drained

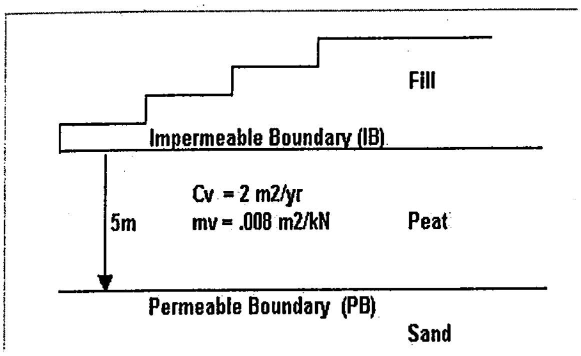


Figure 8. Idealized section used for F.D.M

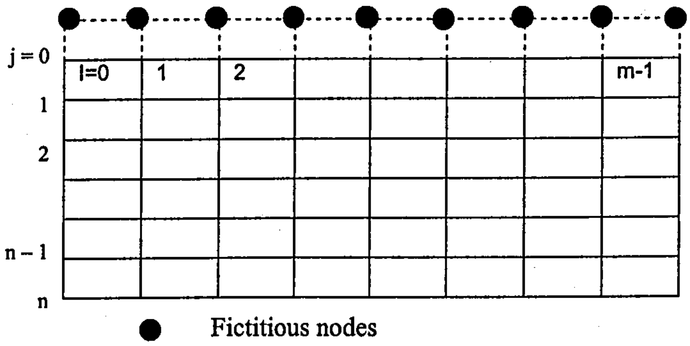


Figure 9. Finite Difference Grid

For permeable boundary
$u_{i+1,j}=u_{i,j}+ c_v\delta t/\delta H^2(u_{i,j+1}+ u_{i,j-1}-2u_{i,j})+(\sigma_{i+1}-\sigma_i)$

For impermeable boundary
$u_{i+1,j}=u_{i,j}+ \beta(2u_{i,j+1}-2u_{i,j})+(\sigma_{i+1}-\sigma_i)$ where, $\beta = c_v\delta t/\delta H^2$
Where $u_{i,j}$ = pore pressure at time $i\delta t$ at depth level j.

5.1.2 Modeling of the Settlement behavior

Then effective stress at time $i\delta t$ at depth level j can be expressed as
$\sigma'_{i,j} = \sigma_{i,j} - u_{i,j}$
where $u_{i,j}$ - pore pressure at time $i\delta t$ at depth level j
$\sigma_{i,j}$ - total stress at time $i\delta t$ at depth level j
Using the Terzaghi consolidation Theory settlement ' δ ' for each layer is calculated as follows.
$\delta = m_v\Delta\sigma\Delta h$
where, m_v - coefficient of volume compressibility
$\Delta\sigma$ - effective stress increase in the soil mass
Δh - thickness of the compressible layer considered
The total settlement due to a time varying load can be evaluated by summing the settlements of each layer. This can be mathematically expressed as follows;
$s_{i,j} = m_v*\delta h*(\frac{\Delta\sigma_{i,0}}{2}+ \Sigma\Delta\sigma_{i,j} +\frac{\Delta\sigma_{i,n}}{2})$
where the for the summation part j= 1 to n-1
(assume soft layer is divided to "n " no. of layers)

5.2 Verification of Finite Difference Method

To compare the accuracy of the finite difference method, the settlements predicted using the finite difference method was compared with the standard Terzaghi solution by considering the instantaneous loading (Figure 10). Model was also verified with the stage loading case for different c_v values and also for different boundary conditions (figure 11 &12). The ultimate primary settlement was seen to be same irrespective of the rate of consolidation, confirming the validity of the finite difference model. Also figure 13 presents excess pore water pressure dissipation for instantaneous loading and stage loading situations for different c_v values.

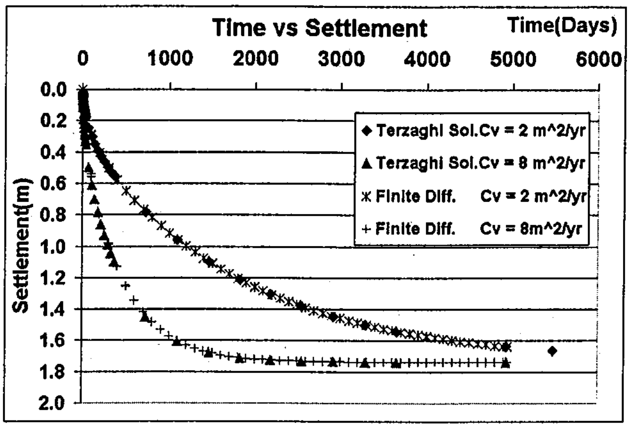


Figure 10. Comparison of F.D.M. with Terzaghi Model

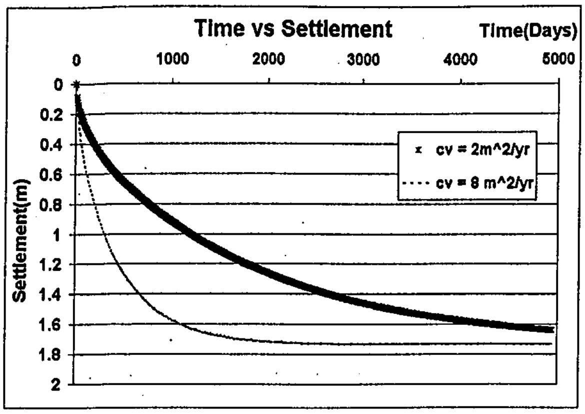


Figure 11. Settlements for stage loading for different c_v values

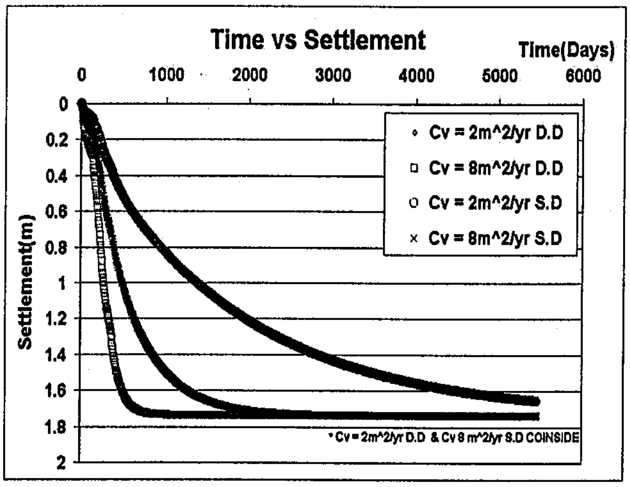


Figure 12. Time settlement graph for different Boundary conditions

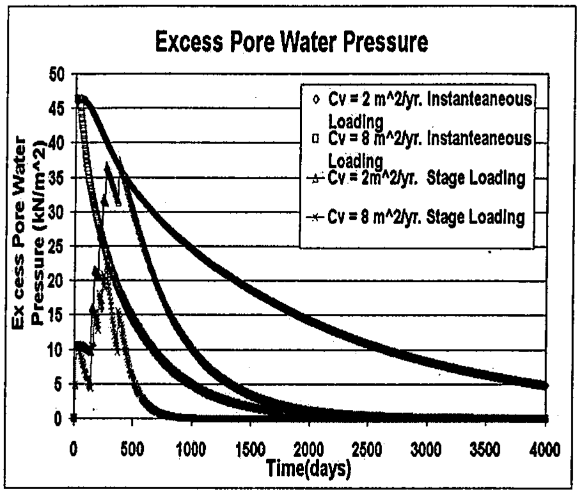


Figure 13. Excess pore water Dissipation using F.D.M

5.3 Comparison of Pore Water Pressure Dissipation.

The predicted excess pore water pressure dissipation was compared with the actual pore water pressure dissipation measured from the pizometer installed in the field, namely; pizometer 5. The results are presented in figure 14.It was clear that the actual pore water pressure dissipation is much faster than that predicted with the laboratory determined c_v value of 2 m^2 /yr. As such, the pore water pressure dissipation was predicted using higher values of c_v.. The prediction was found to be closer to actual observed behavior when c_v equal to 8 m^2/yr was used (Figure 14).

If double drainage conditions were assumed (i.e. lateritc gravelly clay is taken as permeable.) the prediction of pore water pressure dissipation by the finite difference model is presented in Figure 15. This clearly shows that there is no agreement with the observations. Thus the validity of the single drained assumption is confirmed.

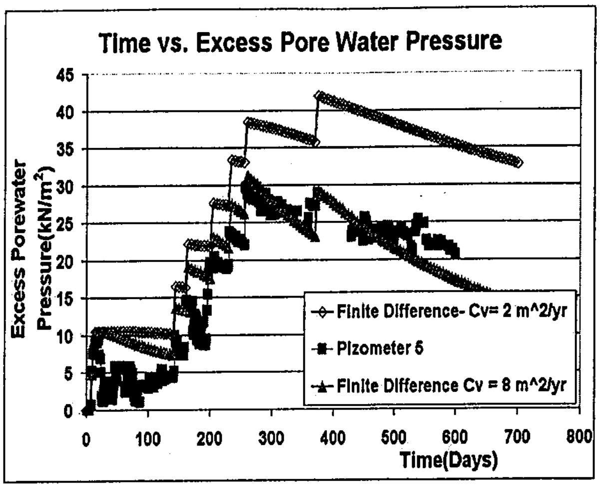


Figure 14. Comparison of p.w.p. For different c_v values

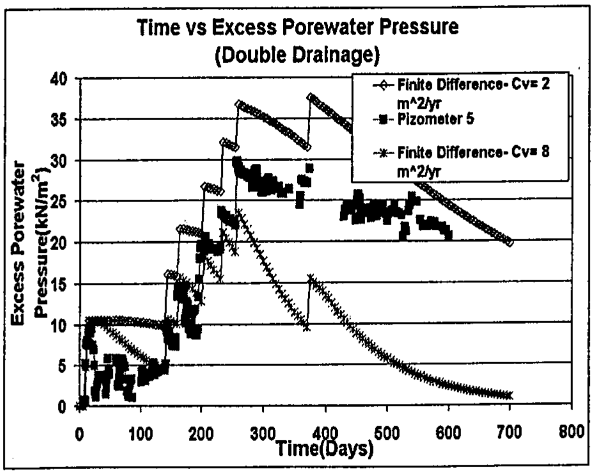


Figure 15.Comparison of pore pressure for double drainage conditionsettlement

5.4 Degree of consolidation in the field

According to the computations the excess pore water pressure of the pizometers at the time when the undrained samples were taken after filling, is around 20kN/m². The pizometer tip was located at the mid level of the peat layer. Hence the degree of consolidation at the mid level can be calculated as follows.

$$U_z = \frac{u_0 - u_z}{u_0}$$

where,

Uz - degree of consolidation $*u_o$ - initial excess pore water pressure u_z - excess pore water pressure

$*u_o = \Delta\sigma = 46$ kN/m² $u_z = 20$ kN/m²

Therefore based on the observed pore water pressure, the degree of consolidation at mid depth is, U_z = 57%.

According to the isochrones corresponding to the one way drainage condition this degree of consolidation (Uz) corresponds to a time factor T of 0.3. (Figure 16)

The average degree of consolidation (U) corresponding to this time factor is 61 % (figure 17). Therefore, based on the pore water pressure observations 61% of ultimate primary consolidation settlement should have taken place around this time.

On the other hand, based on the finite difference computation with c_v = 8m²/yr, the pore water pressure at this time is 17.5 kN/m². This gives a degree of consolidation of 62% at the mid depth and an average degree of consolidation of around 66%. These values are quite close to those obtained from pore water pressure observations

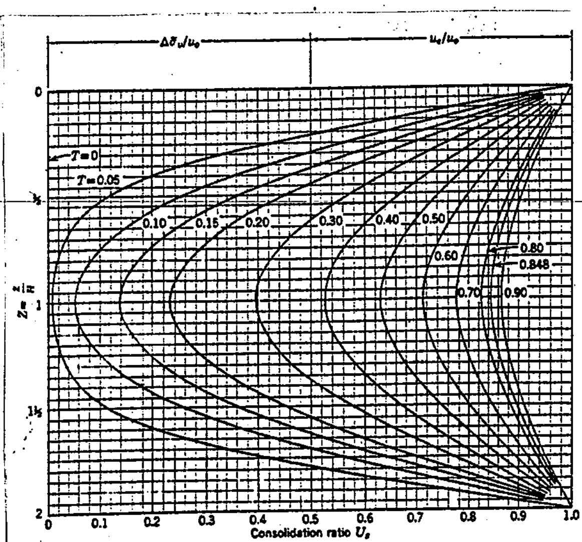


FIGURE. 16. CONSOLIDATION RATIO AS FUNCTION OF DEPTH AND TIME FACTOR.

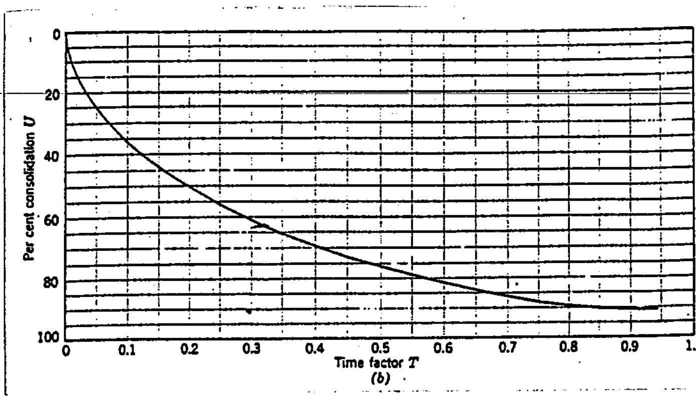


FIGURE . 17. AVERAGE CONSOLIDATION RATIO VS. TIME FACTOR.

5.5 Comparison of predicted and actual settlements

The predicted settlements were compared with the actual cumulative settlements measured from the settlement gauges installed in the field, namely; settlement gauge 3 and 5. It has been observed that actual settlement is much higher than the settlements predicted with the laboratory-determined coefficient of consolidation 2m^2/yr. It is an indication that the field rate of consolidation is much faster than the prediction done using laboratory parameters. Therefore settlements were predicted using a for c_v value of 8 m^2/yr and results are is given in figure 18. It should be noted that a reasonable prediction of excess pore water pressure was also obtained with the use of a c_v value 8 m^2/yr. Assumed single drained boundary conditions appear to be reasonable.

As described in section 5.3.1, the ultimate primary settlement (δ_{sp}) calculated from the Terzaghi's closed form solution and Finite difference method is same. That can be calculated as follows;

$$\delta_{sp} = m_v . \Delta\sigma . H$$

Where ,

m_v = coefficient of volume compressibility determined through laboratory consolidation test

$\Delta\sigma$ = stress induce to filling

H = compressible layer thickness

δ_{sp} = .0075 m^2/kN * 46 kN/m^2 * 5m = 1.725m

The settlement computed by the finite difference model for this time is 1.207m. Thus, the average degree of consolidation is theoretically 1.207/1.725 = 70%. The two values of estimated degree of average consolidation (66% and 70%) are quite close to the value obtained through pore water pressure measurements (61%).

Based on this, estimated primary consolidation settlement should be around 1.725*0.7 = 1.208m. However, the observed settlement is 1.445m, which is greater than the estimated primary consolidation settlement. The difference could be attributed to the secondary consolidation settlement taking place along with the primary consolidation. Similar observations were made by Mesri et al. (1997), Hobbs et al. (1986).

The predicted settlements for double drainage condition did not agree with the observations and it can be concluded that, the assumptions that fill is impermeable is reasonable.

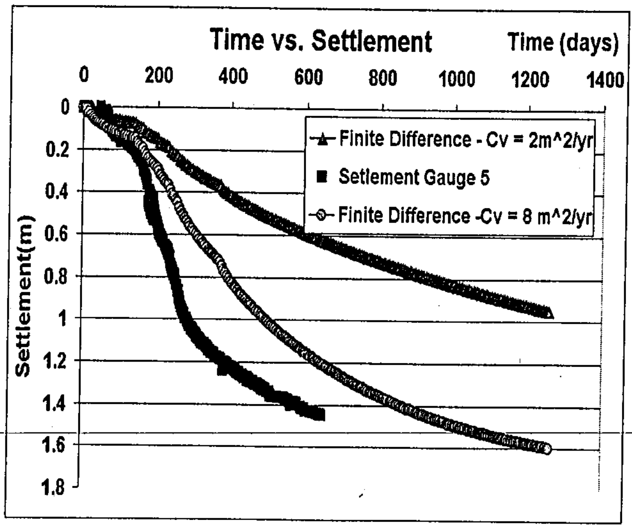


Figure 18. Comparison of p.w.p. For different c_v values

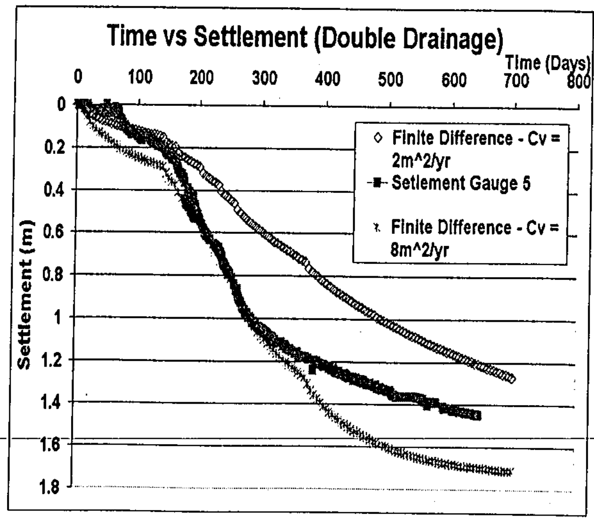


Figure 19.Comparison of settlement with different boundary conditions (Double Drainage)

6. Conclusion

Tests done in the laboratory simulating the field preloading process indicated that both primary and secondary consolidation characteristics of peat can be improved significantly by preloading. When the “after filling sample” which was subjected some consolidation (around 65 – 70% consolidation) was subjected to laboratory testing it showed some improvements from the natural peat.Improvements achieved were of the same order, but slightly lower than in the case of simulated testing.

Results of the simulated testing indicated that shear strength gain due to consolidation was around 0.2 to 0.24 times the applied stress change. Undrained shear strength obtained from the samples tested after the filling showed an average increase of around 10 kN/m^2 which is also equal to 0.2 times the weight of the fill placed(46 kN/m^2) . Comparison of vane shear test results obtained before and after filling also indicated an improvement of the same order. As the field consolidation appear to be incomplete further improvements can be expected. Therefore the relationship $\Delta cu = 0.2\ \Delta\sigma$ can be taken as an lower bound to the improvement of undrained shear strength due to consolidation of the peaty clay.

Field behavior was modeled by Terzaghi Theory with the assistance of the finite difference method. It was shown that the dissipation of pore water pressure in the field was much faster than the rate suggested by the lab c_v value. Therefore, prediction was done using higher c_v values and it was found to be closer to the observed behavior when the ratio between c_v field /c_v lab = 4. Field settlements are still higher than that predicted by the Finite Difference model. This may be due to some secondary consolidation type settlements happening prior to the complete dissipation of the pore water pressures.

Acknowledgements.
The authors wishes to express his gratitude to the Director Geotechnical, National Building Research Organization for giving opportunity to conduct this research and permission to use necessary data.

References

1 Das. B.M.,- Advanced Soil Mechanics ,1985.
McGraw -Hill International Edition

2 .Hobbs N.B., (1986), “ Mire morphology and the properties and behavior of some British and Foreign Peats quarterly”, Journal of Engineering Geology, 19 Vol. 1,7-80.

3. Kulathilaka S.A.S (1999) –“Improvement of peat by preconsolidation”. Proceedings of the Eleventh Asian Regional Conference on Soil Mechanics and Geotechnical Engineering in Seoul, South Korea. Pp97-100

4. Mesri G., Stark T.D., Ajlouni M.A., Chen C.S., “Secondary Compression of Peat with or without Surcharging”, Journal of Geotechnical and Geoenvriomental Engineering Volume No. 5, May 1997

5. S.K Lee, Weels White and Owen G Sngles –Geotechnical Engineering Pitman Publication. 1983

6. Wojcieoh Wolsk, Alijzy Szymansks, “Two Stage- Construced Embankment on Organic Soils”, Swedish Geotachnical Engineering Report- No 32.

7. Yuan Chun Eugene Chang, “Long Term Consolidation beneath the Test Fills at Vasby, Sweden” , Swedish Geotachnical Engineering Report- No 13.

ANALYSIS AND RESPONSE OF GRANULAR PILE REINFORCED GROUND

Madhira R. Madhav
Department of Civil Engineering
Indian Institute of Technology, Kanpur 208 016, India
Email: madhav@iitk.ac.in

ABSTRACT

Installation of granular piles or stone columns consisting of compacted gravel and/or sand, into soft soils, is one of the most common methods of ground treatment. Granular piles reinforce the soft ground by carrying substantially greater proportion of the applied loads but deforming by a significantly smaller amount compared to the in situ soft soil. The consolidation of the soft soil is accelerated because of the higher permeability of the granular material. As a consequence of the installation process, the original ground condition gets densified in addition to being reinforced. Granular piles, if installed in loose sands, minimise the likelihood of liquefaction because of their tendency to dilate while shearing and also, to dissipate the excess pore pressures generated during a seismic event. Granular piles are often used to treat large areas, such as for liquid storage tanks, embankments, abutments, water or waste treatment plants, etc. Where a large group of granular piles are provided, the approach for their analyses is one of reinforced or composite ground. This paper presents some of the results pertaining to the reinforcement effects on strength, settlement and consolidation of the reinforced ground.

INTRODUCTION

Soft soil or loose granular deposits exist all along nearly 5000 km long coast of India and in the alluvial plains of Ganges and Brahmaputra. These areas are the preferred sites for the development of the country with major industrial and other infra-structural facilities located there in. The deposits consist of soft soils with low shear resistance, high compressibility, some times high sensitivity and low permeability. The alluvial belt is subject to moderate to high seismic activity. Consequently, the structures founded on these deposits have low bearing capacity, suffer high settlements, long consolidation times and effected by liquefaction in case of loose granular deposits subjected to earthquake loads.

A variety of ground improvement techniques are available and are being practiced to improve these sites to meet the exacting demands of the projects in terms of safety and serviceability. Amongst the various alternatives, granular piles or stone columns are the most preferred choice in many instances because of the ease of construction, proven applicability and utilisation of low cost natural materials.

GRANULAR PILES

Granular piles or stone columns are cylindrical elements introduced in to soft ground often by displacement method. In some countries, they are known as sand or gravel compaction piles SCP).They are composed of densified gravel or sand or a mixture of both. Granular piles derive their strength and stiffness from the confinement from the surrounding soil. Of late, granular piles are being reinforced by layers of geogrids or are being encased in a geosynthetic to increase their carrying capacity and stiffness.

Because of their strength, stiffness and relatively high permeability, granular piles contribute to the overall strength and stiffness by reinforcement, drainage and consequent reconsolidation leading to densification of the surrounding soil. Granular piles and the surrounding ground act together as a composite medium. Granular piles installed in loose sand deposits reduce the possibility if liquefaction following a seismic event.

METHODS OF CONSTRUCTION

A variety of methods of construction are available for the construction of granular piles. Notable

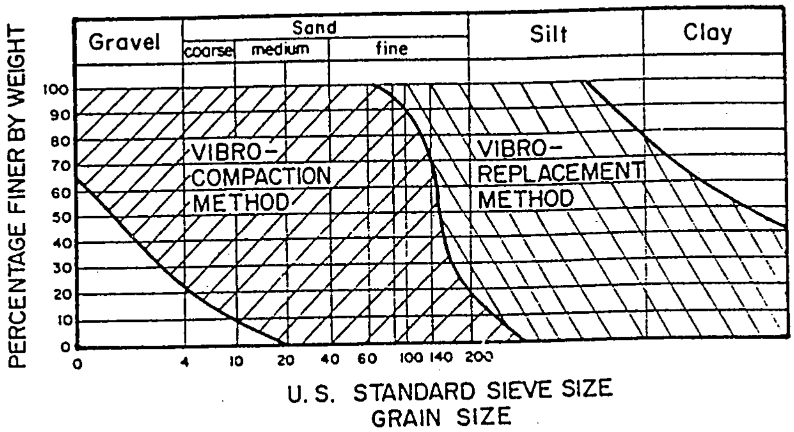


Fig. 1 Range of soils suitable for vibro-compaction and vibro-replacement

among them are 1. The vibro-compaction and vibro-replacement methods (Madhav 1981, Kamon and Bergado 1991 and Bergado et al. 1996), 2. The Rammed Stone Column or the Cased Bore Hole Method (Datye and Nagaraju 1975), 3. The 'Compozer' Method (Aboshi and Suematsu 1985), etc.

The vibro-compaction method utilises a vibroflot to densify cohesionless granular deposits. The vibroflot sinks into the ground under its own self-weight but aided by vibrations and water jets. The vibrator in the vibroflot induces lateral vibrations in to the ground because of which the loose granular deposits initially liquefies and subsequently get densified. The vibroflot is withdrawn gradually after it reaches a predetermined depth. Granular material is added along the side of the vibroflot to compensate for the loss of material that is pushed laterally. The process is repeated gradually by lifting the probe by a meter or two, adding new material and compacting the same with the vibroflot itself. In case of fine-grained soils, the densification effect is negligible and the cavity formed by the vibroflot is filled with granular material and thus the original soil is replaced with granular material. Fig.1 depicts the range of soils that can be treated by vibro-compaction and vibro-replacement methods. Vibro-compaction method is applicable for soils with less than 18 to 20% fines content.

In the Rammed Stone Column or the Cased Bore Hole method, (Datye and Nagaraju 1975) the hole is created as in bored pile construction but usually with a casing as the soils are often soft to very soft. Granular material is dumped in to the hole and rammed by a 15 to 20 kN rammer in lifts of 1.0 to 1.5 m. The drop height of the rammer is 1.5 to 2.0 m. After the formation of the full length of the granular pile, it is further rammed with a heavier weight of 40 kN to precompress the same. Rammed stone column method utilises low cost indigenous equipment and is especially suitable for developing countries.

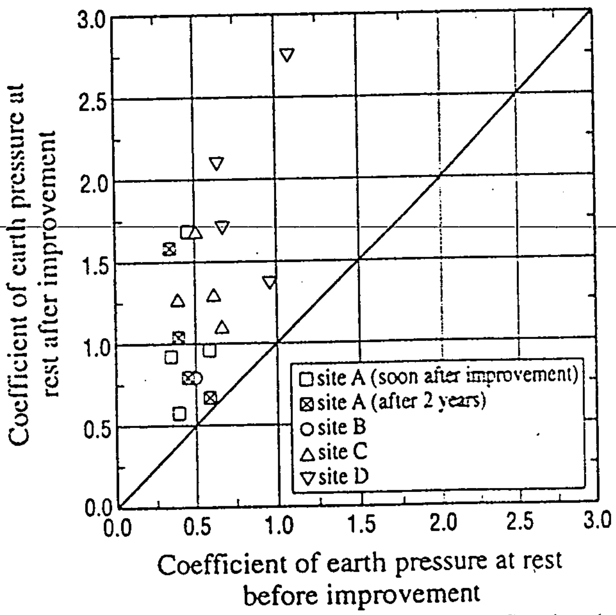


Fig. 2 Coefficient of earth pressure at rest before and after densification (after Ohbayashi et al. 1999)

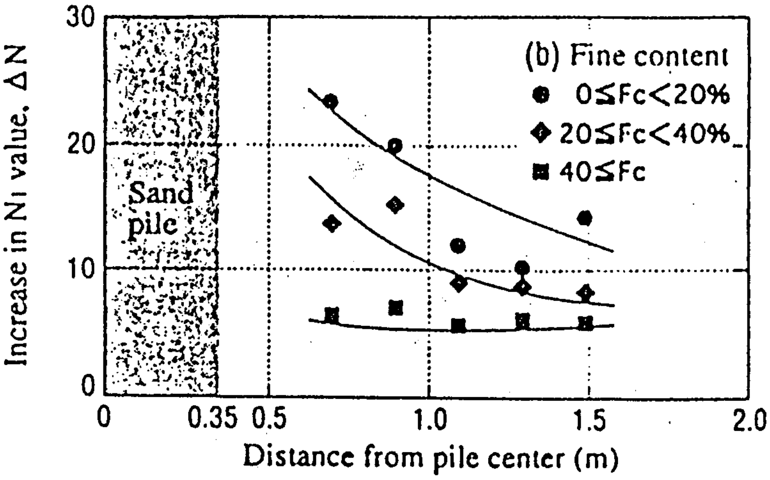


Fig. 3 Increase in SPT N-values (after Ohbayashi et al. 1999)

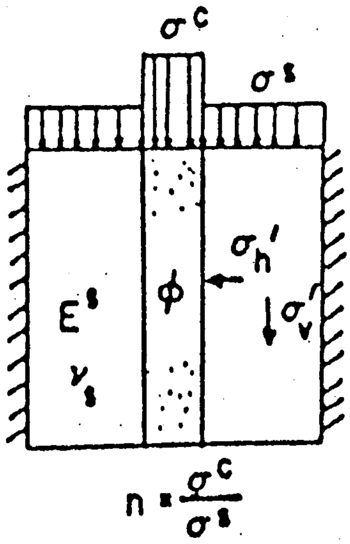


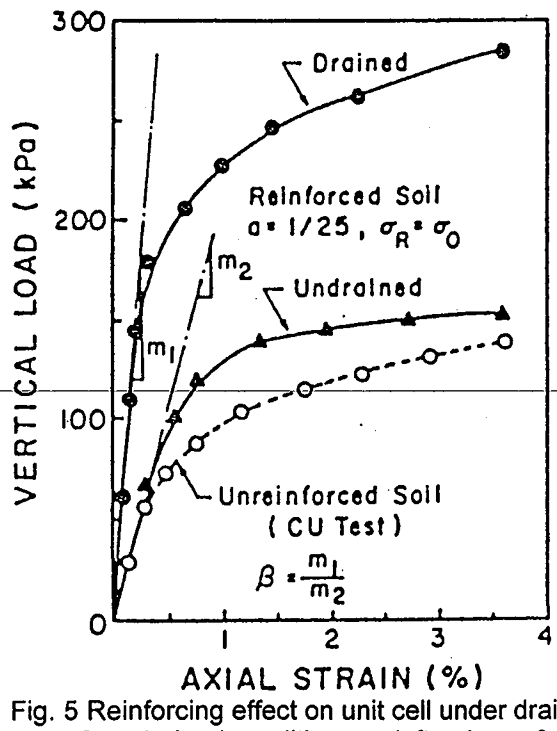
Fig. 4 Unit cell

DENSIFICATION EFFECT

One of the chief benefits of ground treatment with granular piles is the densification of in situ ground. The effect of densification is manifested through an increase in the coefficient of earth pressure at rest and in the values of modulus of deformation of the soil. Fig.2 presents a correlation between the coefficients of earth pressure evaluated by horizontal loading tests in borehole before and after installation of SCP (Ohbayashi et al. 1999). The values of K_0 ranged initially between 0.5 and 0.7 and increased to between 1.0 and 2.0 as a result of densification effect.

The densification effect can easily but indirectly is quantified by in situ tests. The effect is maximum close to the SCPs and least at the farthest point, i.e. at the center of the grid points. The measurements made by Ohbayashi et al. (1999) at different sites by the Swedish Weight Sounding, (N_{sw}), SPT (N) and CPT (q_c) are summarised wherein the increases in the measured parameters are presented as a function of the distance from the center of the compaction point. The densification effect becomes negligible at a distance of about 2.0 m from the center of the SCPs but the increases depend on the fines content (Fig.3). The densification effect is negligible for soils with fines content in excess of 40%.

THE REINFORCEMENT EFFECT



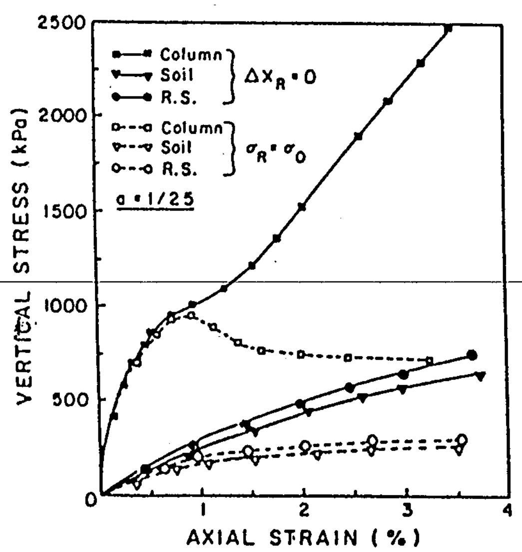


Fig. 5 Reinforcing effect on unit cell under drained & undrained conditions (after Juran & Guermazi 1988)

Fig. 6 Effect of boundary conditions on unit cell response

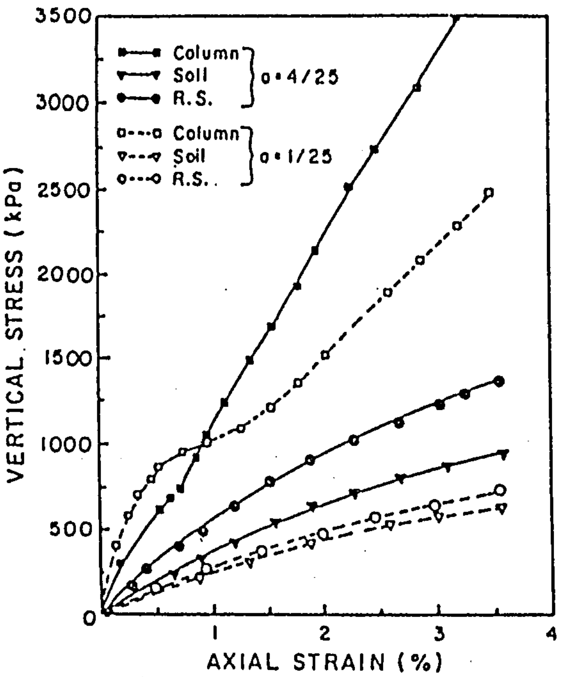


Fig. 7 Effect of area ratio, a_r, on unit cell response (after Juran & Guermazi 1988)

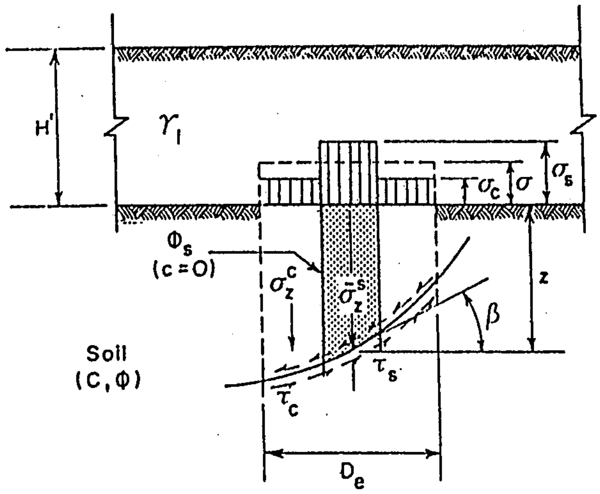


Fig. 8 Equivalent stress method of stability analysis (after Barksdale & Bachus 1983)

STRENGTH

Juran and Guermazi (1988) present an experimental study on a unit cell consisting of a compacted sand column surrounded by soft sol (Fig.4). The test was conducted in a triaxial cell modified to measure simultaneously the stresses and the pore pressures in the sand column and the soft soil separately. The group effect was studied with a replacement factor (ratio of area of granular pile to that of the unit cell) of 1/25 and 4/25. Tests were conducted for two boundary conditions: (i) constant lateral stress and (ii) zero lateral strain (K_0 condition). The latter condition is applicable for reinforced ground. The loading curves (Fig.5) from CU tests on unreinforced and reinforced samples for both drained and undrained conditions, illustrate the reinforcement effect. Even for a relatively small replacement ratio of 4%, the drained specimen exhibits at 100% increase in both the stiffness and the strength compared to the unreinforced one while the reinforced specimen in an undrained condition exhibits only a moderate increase in stiffness and strength.

The behaviour of reinforced specimen under constant confining stress (conventional triaxial test) and K_0 conditions are contrasted in Fig.6. The responses of soil alone and reinforced soil under the latter condition is as expected much better than under conventional constant confining stress. The most interesting aspect of the difference is response of the granular core under K_0 condition. The granular material attains the peak stress at a strain level of about 1%. The peak stress of a dense sample corresponds to maximum dilation rate. While the granular material has a tendency to dilate, and since the sample is under K_0 condition, dilation is suppressed by increased mobilisation of confining stresses. As a result, the granular core does not fail and instead carries increasingly larger stresses. Therefore, the reinforced specimen does not fail under K_0 (one-dimensional strain) condition but deforms with increasing ability to carry axial stresses.

The larger the area replacement ratio, the larger would be the stress carried by the granular core (Fig.7). For an area replacement ratio of 16%, the GP core carries an axial stress of 3,500 kPa at an axial strain of 3% compared to a stress of 2,500 kPa for an area replacement ratio of only 0.04%.

For carrying out stability analysis, the equivalent strength of reinforced ground is often required. Barksdale and Bachus (1983) propose a simple average stress method for estimating the strength of reinforced ground (Fig.8). The granular pile material is characterized by is angle of shearing

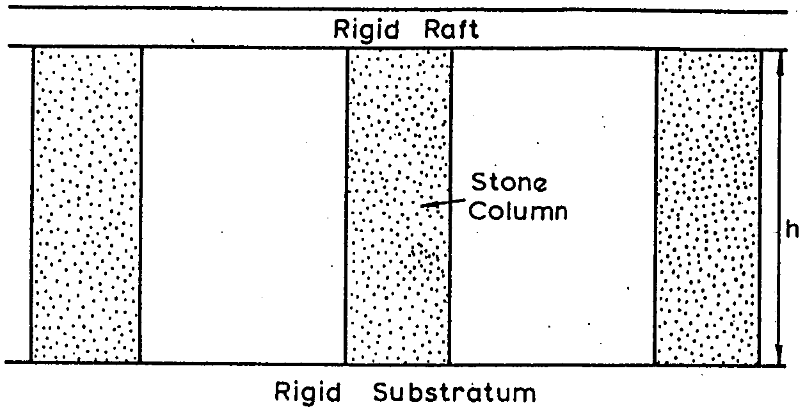


Fig. 9 Definition sketch for analysis of settlement of granular pile reinforced ground

resistance, ϕ', and the soft ground by its undrained cohesion, c_u. The average strength parameters, c_{av} and $\tan \phi_{av}$ are then obtained as

$$c_{av} = c_u \cdot (1 - a_r) \text{ and } \tan \phi_{av} = a_r \tan \phi' \qquad (1)$$

where $a_r = (a/b)^2$ is the area ratio, i.e. area of granular pile in terms of the area of unit cell.

SETTLEMENT

RIGID RAFT ON REINFORCED GROUND

Analysis of settlement of a rigid raft on granular pile reinforced ground (Fig.9) was presented by Balaam and Booker (1981). The granular piles of diameter, d, penetrate fully in to soft stratum of thickness, H, and are arranged either in to a triangular or square pattern (Fig.10). The diameter, d_e, of an equivalent unit cell (Fig. 11) then becomes 1.05.s or 1.13s where is the spacing between the granular piles. If any site has been reinforced by a large number of granular piles, it follows that the response of the unit cell represents the response of the treated ground except near its edges.

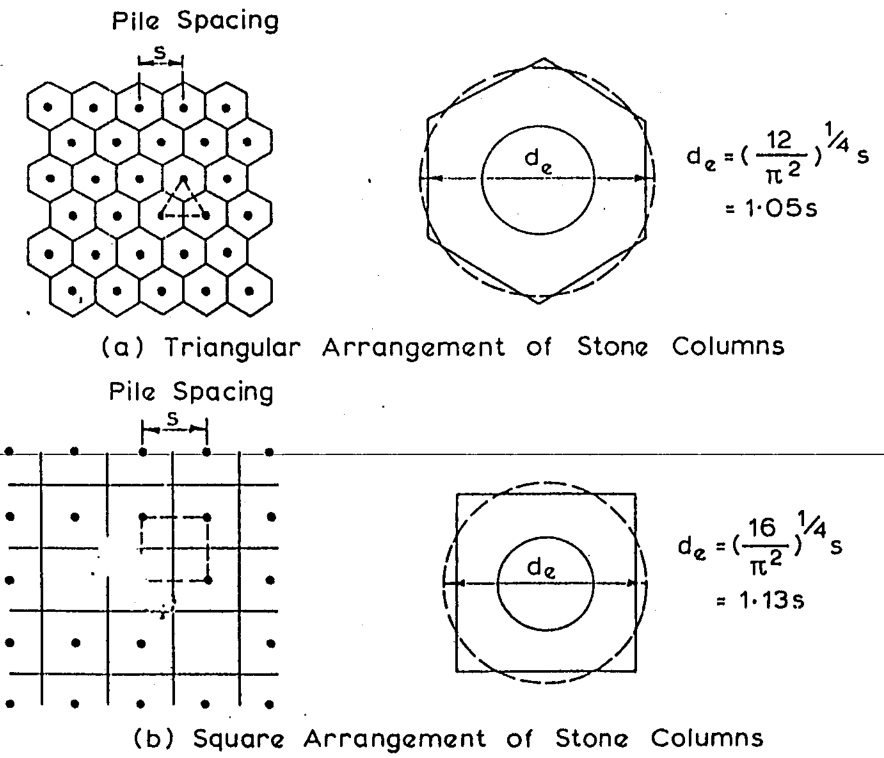


Fig. 10 (a) Triangular and (b) square arrangement of granular piles and the domain of influence

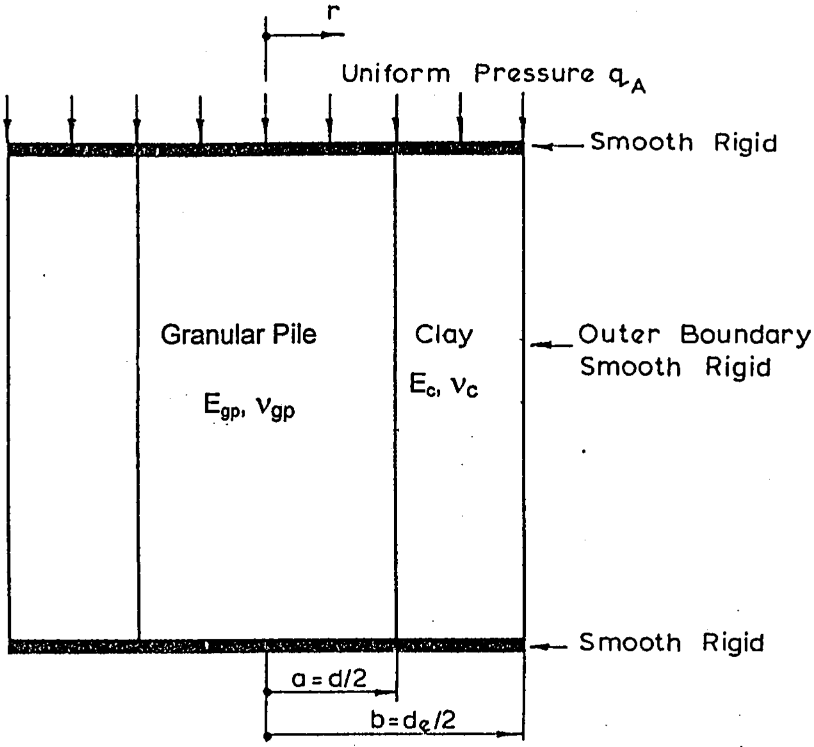


Fig. 11 Definition sketch for settlement analysis of unit cell

By virtue of symmetry, the lateral surface of the unit cell is shear free and the points along it undergo no lateral/radial deformation. The granular pile and the surrounding soft soil are characterized by their elastic moduli, E_{gp} and E_c, and Poisson's ratios, ν_{gp}, and ν_c, respectively. The analysis is carried out based on compatibility of displacements of points just below the rigid raft on the granular pile and the soft soil. The stress state on the granular pile is triaxial while that on soft soil is analysed treating it as a thick cylinder. The normalized settlement ratio of reinforced ground, R_s, or β, is defined as

R_s or $\beta = S_t/S_0$ (2)

where S_t and S_0 are the settlements of reinforced and unreinforced ground respectively.

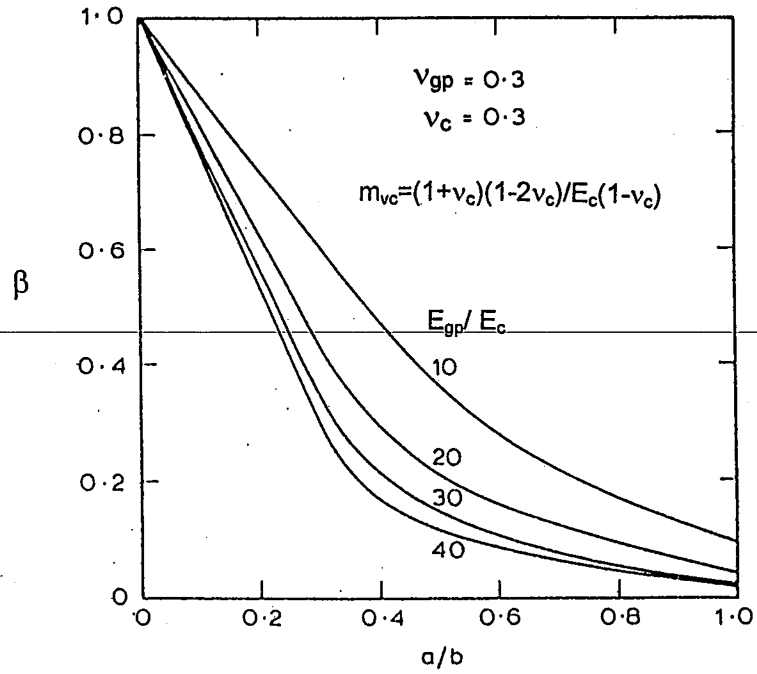


Fig. 12 Reduction in settlement of reinforced ground – effects of a/b & E_{gp}/E_c

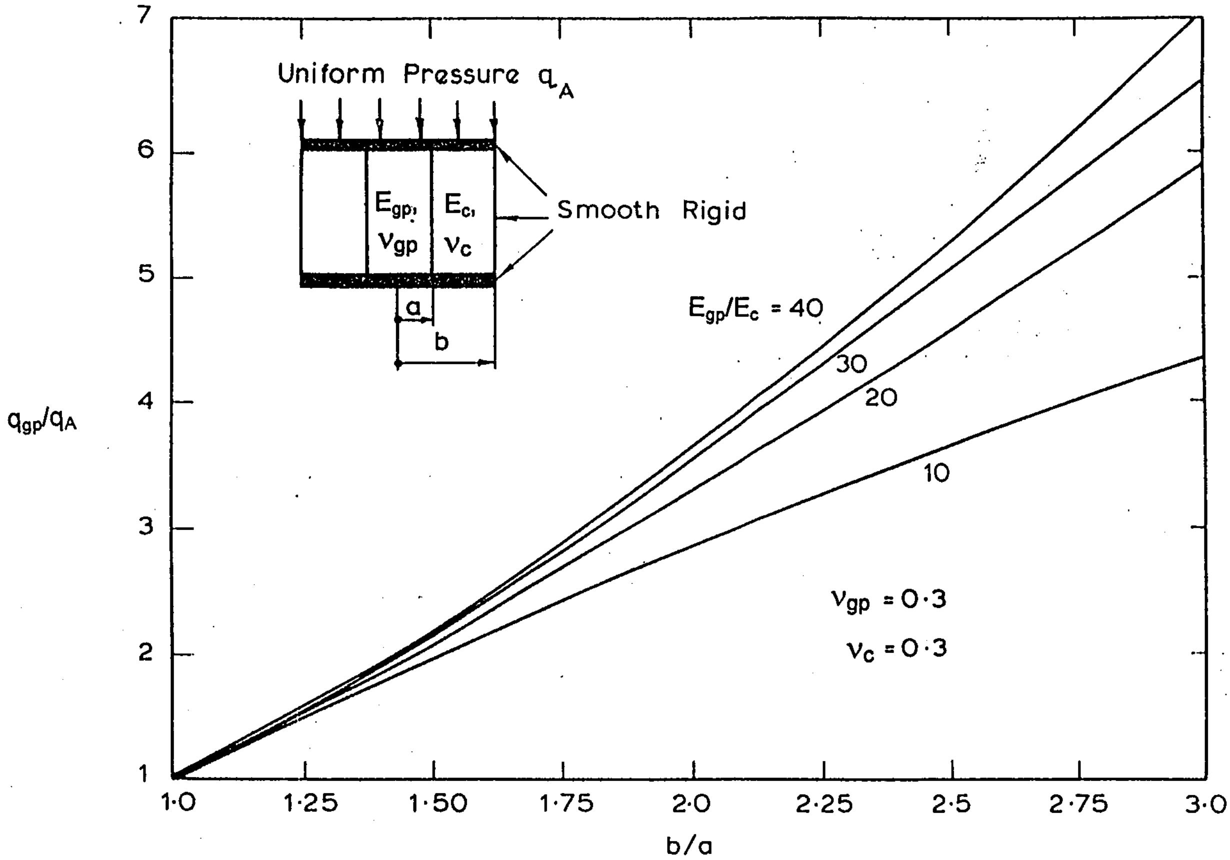


Fig. 13 Variation of normal stress with b/a and E_{gp}/E_c (after Ballam & Booker 1981)

The variation of the ratio of settlements due to reinforcement of the ground with granular piles can be seen in Fig.12. The settlement of the reinforced ground is normalized with that of unreinforced ground and plotted against the a/b ratio (ratio of radii of granular pile and of the unit cell). The settlement reduction ratio reduces sharply with the ratio, a/b, and the modular ratio, E_{gp}/E_c. The decrease is almost linear for a/b ratio of about 0.35. For 50% replacement, i.e. a/b=0.5, the reduced settlements are of the order of 60, 52, 47, and 40% for modular ratios of 10, 20, 30 and 40 respectively. The Poisson's ratios of both the materials are taken as 0.3. The effect of Poisson's ratio on the settlement reduction is negligible.

The stress, q_{gp}, normalized with the applied stress, q_A, with the replacement ratio, b/a, and the modular ratio, E_{gp}/E_c, is depicted in Fig.13. The granular pile tends to carry larger and larger stresses with increasing values of b/a and the modular ratio. The rate of increase is linear for the initial values of b/a (<2.0) and becomes significantly non-linear for further increase in b/a. Once again for b/a=0.5, (25% replacement), the stress ratios are about 2.8, 3.1, 4.3 and 4.4 for modular ratios of 10, 20, 30 and 40 respectively. Stiffer the granular pile larger is the percentage of load carried by it.

DEFORMABLE GRANULAR BED ON REINFORCED GROUND

Whenever granular piles are chosen to reinforce soft ground, a granular bed is laid over it 1. To provide a working platform; 2. To level the site and increase its elevation to make it safe from inundation (tides and floods); 3. To prevent upheaval during granular pile installation; 4. To provide a facility for drainage of water, since granular piles act as drains as well; and to distribute the applied loads on to the reinforced ground. The last function, i.e. the structural effect is very important for the success of the improvement technique. Madhav and Van Impe (1993) analyse the gravel bed – granular pile reinforced ground by considering the former as a shear layer defined by its shear stiffness, G^* (=G.H), where G is the shear modulus and H the thickness. The percentage of load transferred to the granular pile/stone column is dependant on the relative stiffness, K_R (=E_{gp}/D_c) and λ_c (=$K_c a^2/G^*$) with K_c defined ad the modulus of subgrade reaction of the soft ground (=D_c/H). The effect of the relative stiffness of the granular bed in transferring the applied load on to the granular pile is depicted in Fig.14. λ_c equal to zero implies a rigid layer and the results match with those given above.

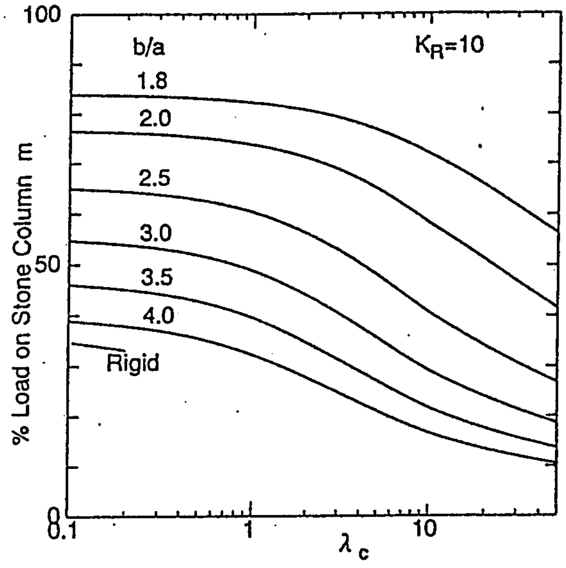


Fig. 14 Percentage load on granular pile with deformable granular pad on top

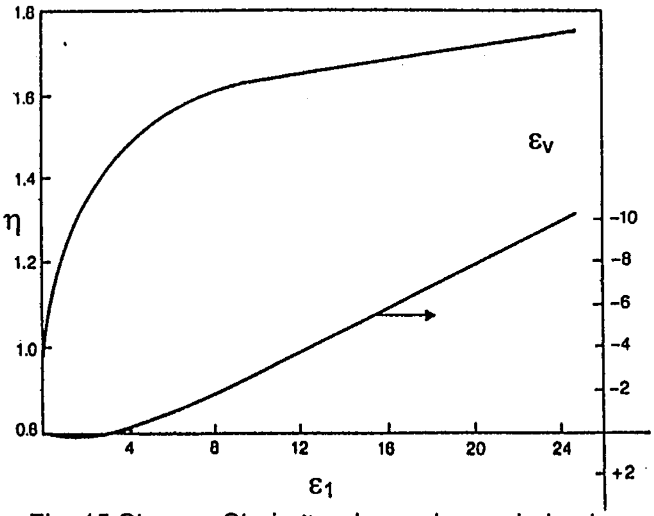


Fig. 15 Stress – Strain & volume change behaviour of granular pile material

With increasing values of λ_c the granular bed becomes more deformable or flexible and decreasing amounts of load are then transferred on to the granular pile. This reduction is dependent on the spacing or the b/a ratio of the unit cell.

RIGID PLASTIC DILATANCY OF GRANULAR PILE MATERIAL

Poorooshasb and Madhav (1985) study the response of granular pile reinforced soil subjected to uniform loading through a relatively rigid raft considering the granular material to follow rigid plastic strain hardening postulates instead of pure linear elastic behaviour. The stress – strain and volumetric - axial strain responses of the granular pile material are shown in Fig.15. The principal stress ratio, η ($=\sigma_1/\sigma_3$) increases with the strain level while the volumetric strain, ε_v, initially shows compression but subsequently indicates large volume increases typical of a dilatant material. The consequence considering dilatancy is dramatically shown in Fig.16 by a concave upward settlement versus applied stress curve which illustrates increasing stiffness of the granular pile material with applied stress. With increasing load, the tendency of the granular pile material to dilate, i.e. increase in volume results in increasing lateral stresses on to it as the unit cell cannot absorb the resulting lateral deformations. Larger the confining stress, stiffer would be the granular pile. It thus carries larger percentage of applied load with smaller increase in settlement leading to a typical one dimensional compression curve.

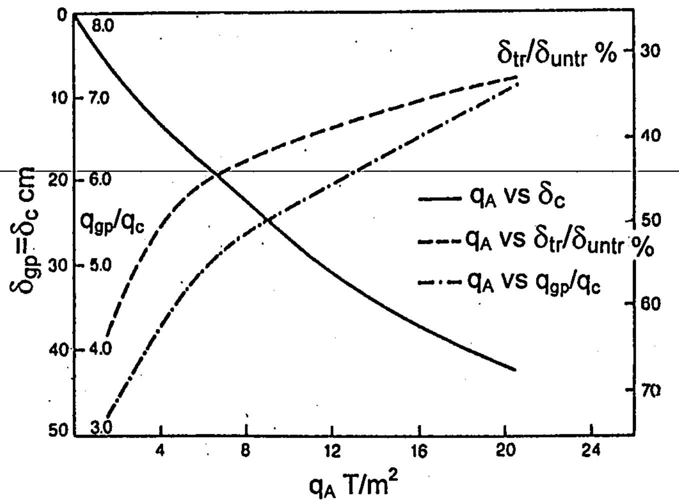
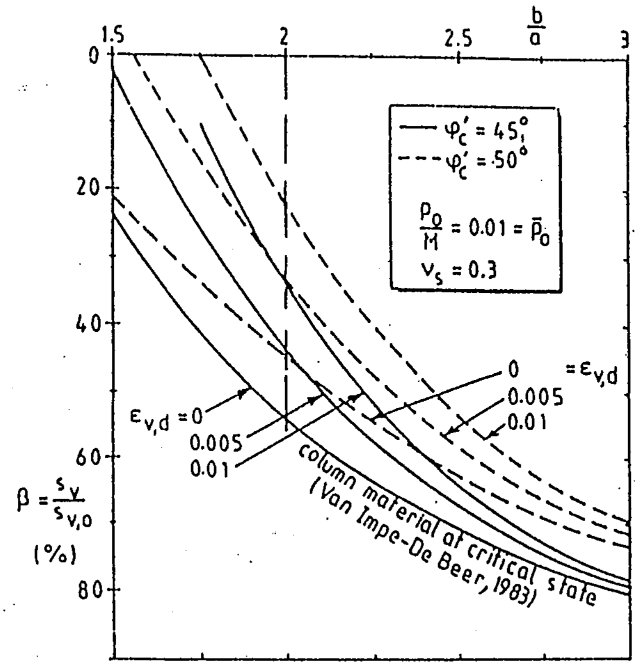


Fig. 16 Results with rigid-plastic dilatancy model of granular pile material



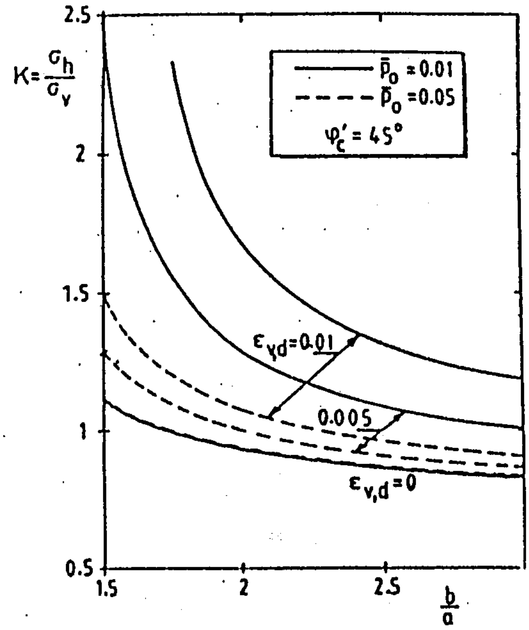


Fig. 17 Settlement reduction of dilatant granular pile at limiting yield condition

Fig. 18 Effect of b/a on lateral stress ratio

GRANULAR PILE AT LIMITING YIELD CONDITION

If maximum benefit from dilation of granular material is to be achieved, the reinforced ground should be designed for the granular material to be at limiting yield condition. Van Impe and Madhav (1992) consider the granular pile material to be at limiting yield condition and to dilate and modify the approach of Van Impe and De Beer's (1983) approach to include axisymmetric conditions of a unit cell. The reduction in settlements with the b/a ratio for different dilations, i.e. ε_v = 0, 0.005 and 0.0, are given in Fig.17.

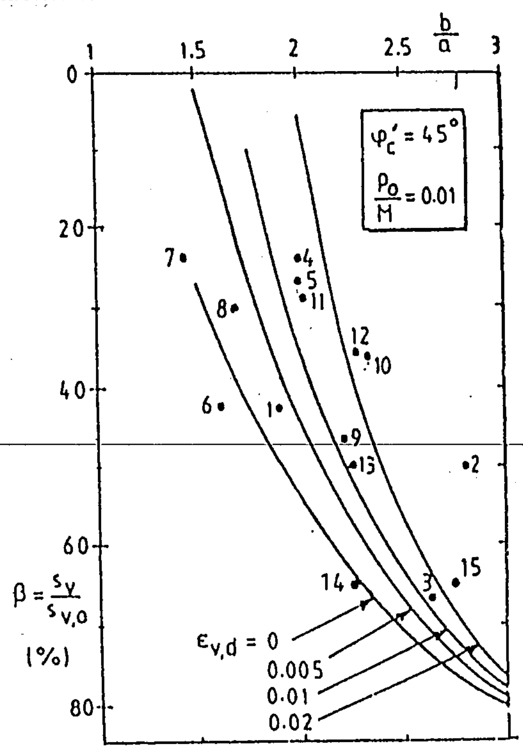


Fig. 19 Comparison of predicted and in situ measured settlement ratios

The settlement ratios reduce significantly if the dilation is considered as mentioned in the previous paragraph. Larger the value of angle of shearing resistance smaller would be the settlements of the reinforced ground as granular material with a higher friction angle can carry higher stresses at limiting yield loads. The lateral stress ratio, K ($=\sigma_h/\sigma_v$) increases (Fig.18) for closer spacing of granular piles (smaller values of b/a) and for higher dilatancy ratios.

VALIDATION

Some of the approaches presented in the above paragraphs could be verified with a limited number case records wherein granular piles were used to reinforce and improve the ground. Fig.19 depicts that the predicted results do fall within the measured range of settlement reduction. The projects were located worldwide from Middle East to Europe and USA. In the absence of complete data, the predictions were made with ' value of 45^0 and a Poisson's ratio of 0.33. The settlement ratios range between a low 25% to about 65% thus validating the applicability of granular piles as reasonably good reinforcing elements.

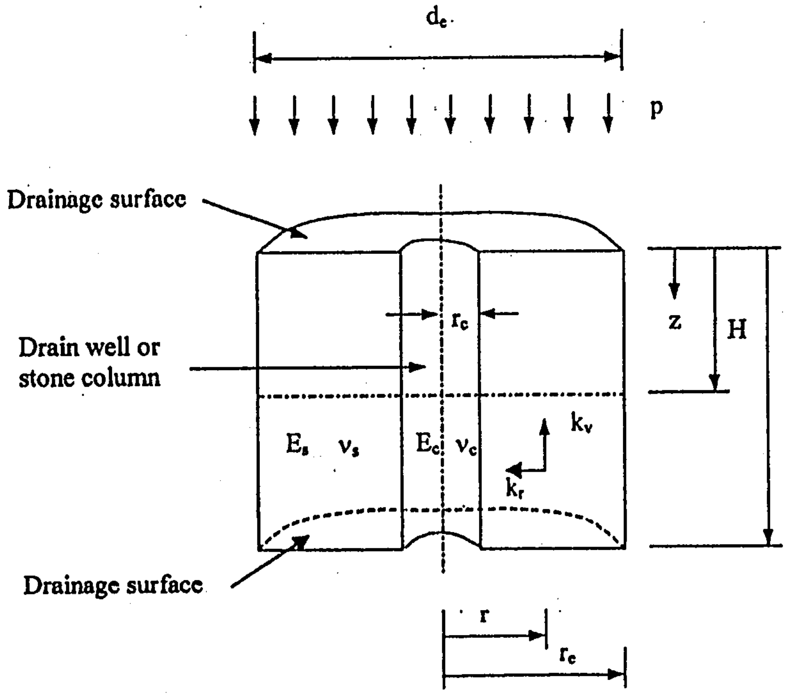


Fig. 20 Definition sketch for modelling consolidation of unit cell

CONSOLIDATION

Consolidation characteristics of granular pile reinforced ground are very different from those of drain treated ground even though they have many similarities such as freely draining central core and an annulus of soft and compressible soil. The flow is predominantly radial (Fig.20). The major difference arises due to stress transfer between the soft soil and the granular pile. Vertical drains, particularly, the Geosynthetic (PB drains) ones have no axial stiffness and cannot carry any load unlike the granular piles. Results shown Fig.21 for a spacing ratio, N, of 3, and relative modular ratio, n_s, of 5, the increase of stress concentration ratio with time. Initially, the entire applied load is taken up by the soft soil alone as it is fully saturated and is incompressible in the undrained state. With time the soft soil consolidates and tends to compress. However, the granular pile being stiffer than the soft soil takes up more load, as the compatibility of deformations has to be satisfied. Thus the stress concentration ratio increases from 0 at time factor, T, equal to 0, to 3.5 for T equal to 0.1. Consequently, the soft soil gets a relief of the applied load and consolidates under continuously decreasing total stress. The pore pressures in the soft soil get reduced with the reduction in the load carried by the soft soil. The dissipation of excess pore pressures depends on the two factors, drainage as in the case of vertical drains and reduction in vertical stress due to partial transfer of applied stress from soft soil to granular pile. Fig.22 depicts this phenomenon very clearly.

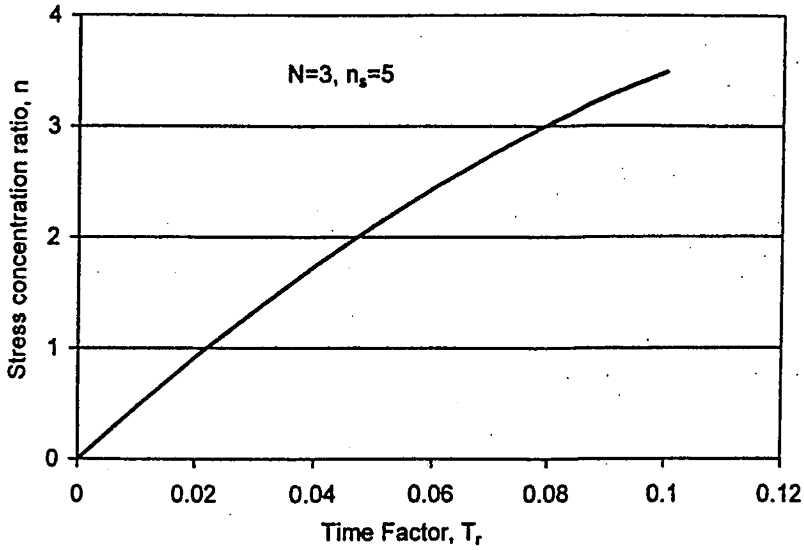


Fig. 21 Variation of stress concentration ratio, n, with time (after Han and Ye 2001)

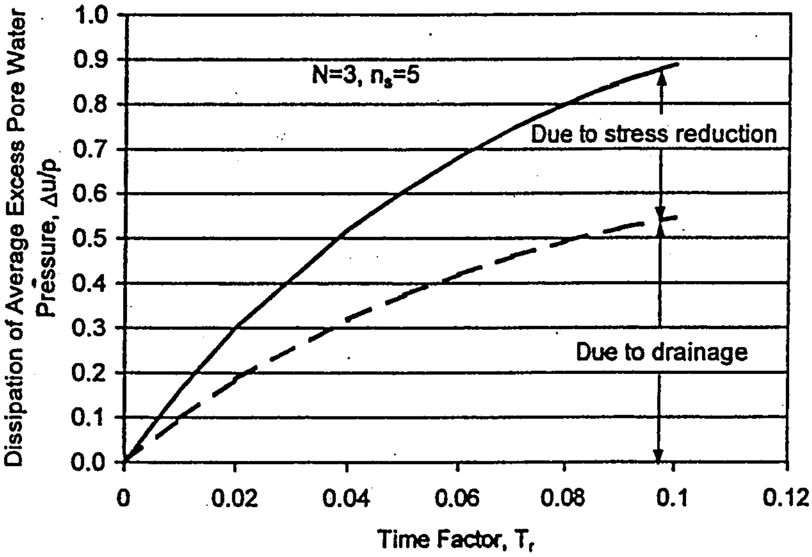


Fig. 22 Dissipation of excess pore pressure (after Han and Ye 2001)

The time dependant behaviour of granular pile reinforced ground supporting a rigid raft has been analysed by Balaam and Booker (1981) based on Biot's rigorous theory of consolidation. For the two dimension (radial and vertical) flow and consolidation, the radial coefficient of consolidation, c_r, is defined as

$$c_r = \frac{k_h E_c (1-\nu_c)}{\gamma_w (1+\nu_c)(1-2\nu_c)} \tag{3}$$

where k_h is the horizontal coefficient of consolidation. The rate of settlement of the rigid raft with time factor, T_h ($=c_r.t/d_e^2$) for different values of the modular ratios is presented in Fig.23 for b/a ratio of 2.0. The degrees of consolidation of reinforced ground with different modular ratios are significantly larger compared to that of unreinforced ground (Barron's solution). As mentioned earlier, the reduction in the stress transferred on to soft soil with time adds to further increase in the rate of consolidation. The degree of consolidation is more than 80% for the modular ratio of 40 at a time factor of 0.01. Thus granular pile reinforced ground consolidates very rapidly and thus develops strength in a very short time. The variations in the stresses on the granular pile and the soft soil as consolidation progresses, i.e. with T_h, can be seen in Fig.27. Initially, the soft soil being incompressible carries slightly larger stress than the granular pile but with time a very significant part of the load is carried by the granular pile.

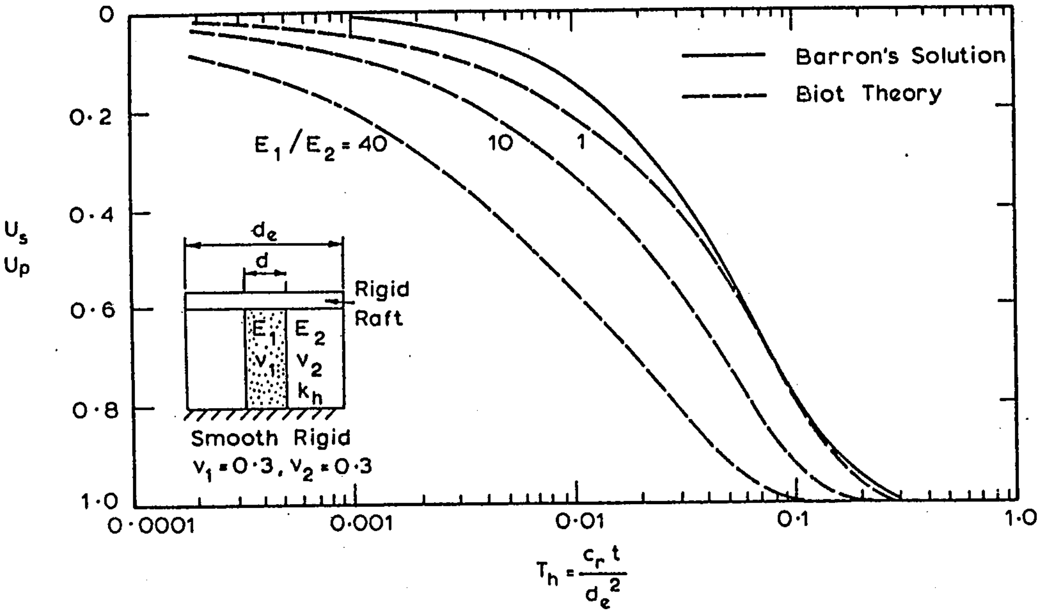


Fig. 23 Rate of settlement of rigid raft on granular pile reinforced ground (after Ballam & Booker 1981)

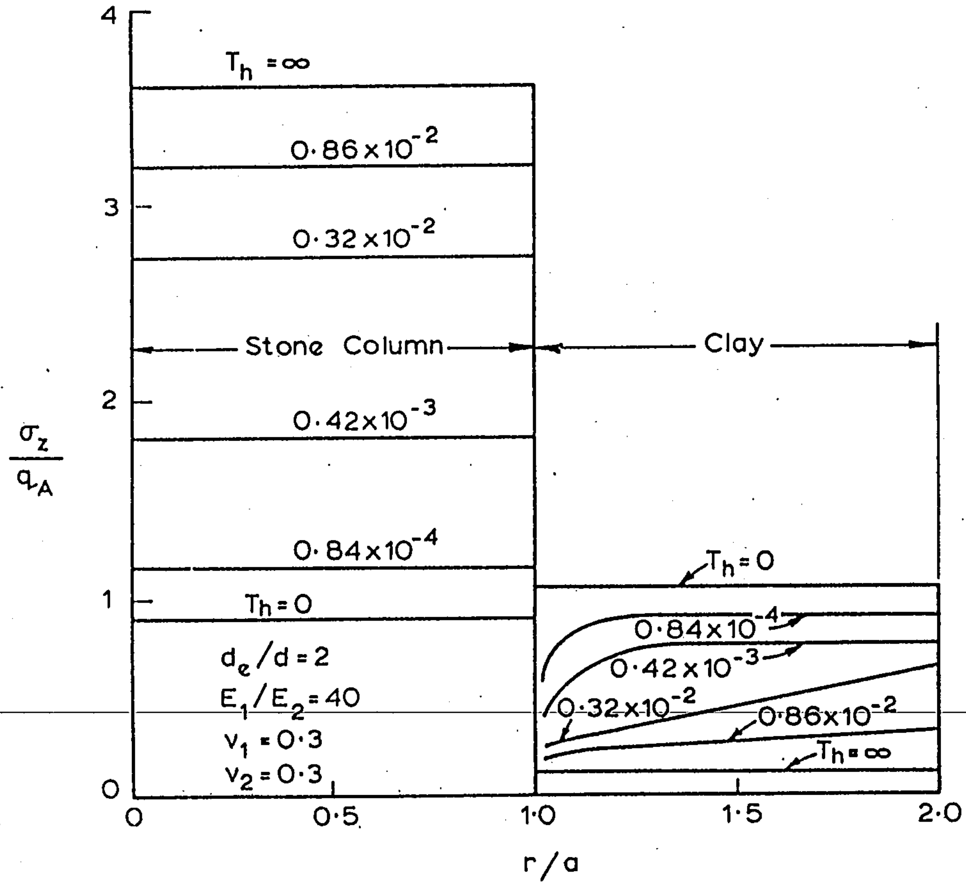


Fig. 24 Change of vertical stress with time (after Ballam & Booker 1981)

Han and Ye (2001) present a simplified approach account for the stress transfer along with radial drainage into granular pile in a unit cell. The modified or equivalent coefficient of consolidation, c_e*, is derived as

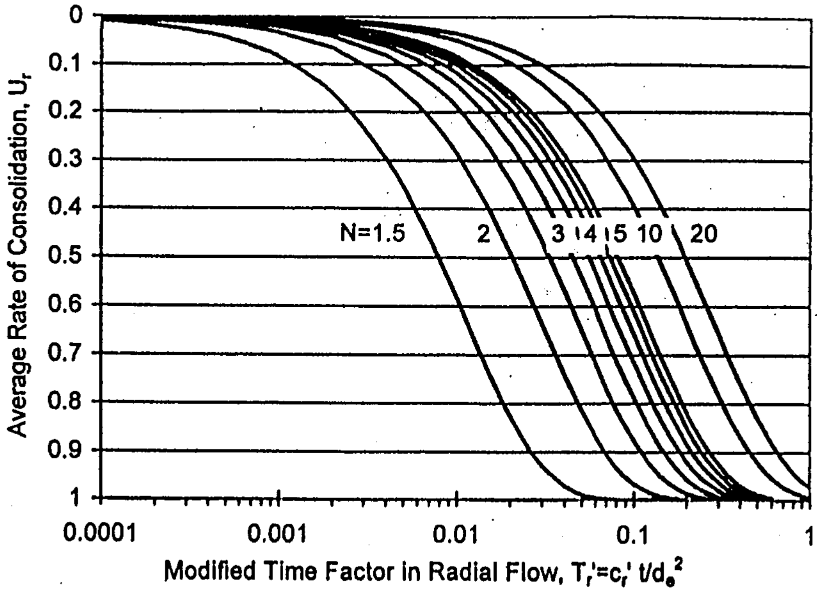


Fig. 25 design curves for rate of consolidation of reinforced ground (after Han and Ye 2001)

$$c_r^* = c_r\left(1 + n_s/(N^2 - 1)\right) \tag{4}$$

where c_r is the radial coefficient of consolidation. A similar correction can be made for the vertical coefficient of consolidation but the contribution of vertical flow to overall consolidation is relatively small and is usually neglected. Fig.25 presents the degree of consolidation versus modified time factor curves for ready use by practicing engineers. These curves are identical to those for vertical drains except that the time factor accounts for the relative stiffness and spacing of granular piles.

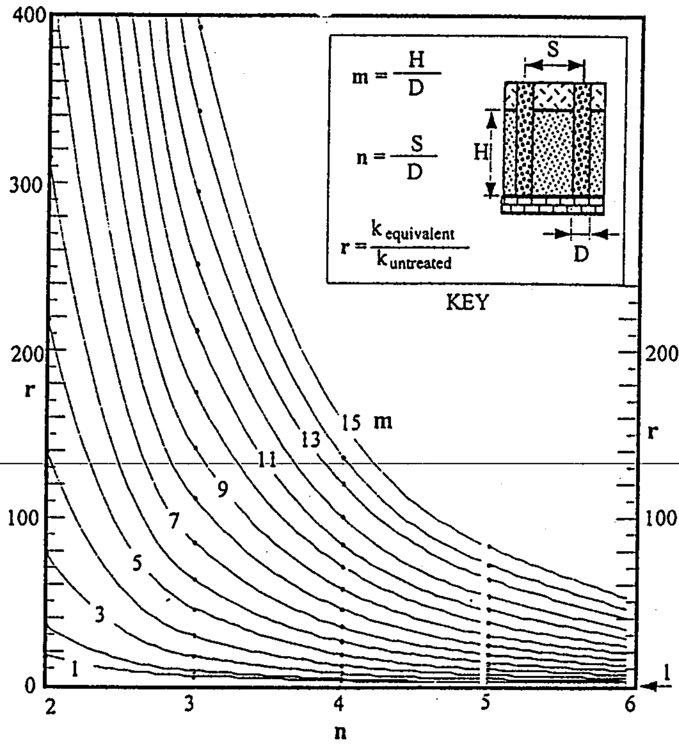


Fig. 26 Design chart for evaluation of k_{eq} (after Poorooshasb et al. 2000)

RESISTANCE AGAINST LIQUEFACTION

Granular piles help in mitigating earthquake induced liquefaction effects through one or more of these functions:

1. Granular piles, installed in to a very dense state, are not prone to liquefaction and replace a significant quantity of in situ liquefiable soil;
2. Granular piles modify the nature of earthquake experienced by the in situ soil;
3. Granular piles function as drains and permit rapid dissipation of earthquake induced pore pressures by virtue of their high permeability with the additional advantage that they tend to dilate as they get sheared during an earthquake event.

Based on the radial consolidation theory as applicable to granular pile treated ground, Poorooshasb et al. (2000) propose an equivalent coefficient permeability, k_{eq}, for the treated soil in terms of the permeability, k_{untr} of untreated ground, as $k_{eq} = k_{untr} \cdot t_{50}$ (for untreated ground)/t_{50} (for treated ground) where t_{50} values for the untreated and granular pile treated ground are the times for 50% degree of consolidation based on one dimensional and radial consolidation theories respectively. The ratio, r ($=k_{eq}/k_{untr}$) is derived (Fig.26) in terms of the spacing ratio, n (= S/d) where S and d are the spacing between and diameter of granular piles. The results presented are for triangular arrangement of drains but can easily be modified for square pattern.

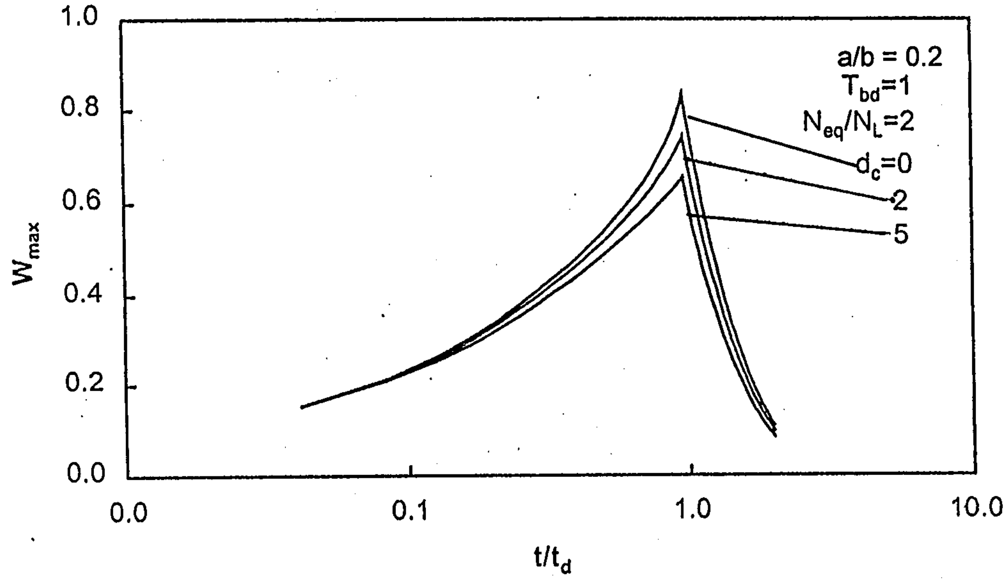


Fig. 27 Effect of dilation on maximum pore pressure ratio

Granular piles installed in loose sand deposits are often compacted to a relatively high density. Seismic forces, which tend to generate positive pore pressures in these deposits, cause an opposite effect, of dilation in the dense granular piles. During the seismic event, negative pore pressures that tend to get generated therein increase the gradient and permit rapid rates of drainage than otherwise. Madhav and Arlekar (2000) quantify this effect of dilating granular pile in mitigating liquefaction damage. The variation of maximum pore pressure ratio, W_{max}, with normalized time for different rates of dilation is depicted in Fig.27. The results compare well with those of Seed and Booker (1977) for no dilation. The effect of dilation rates in reducing the peak value of the pore pressure ratio can be clearly seen in the figure.

CONCLUSIONS

Deposits consisting of soft soils or loose granular deposits have low shear resistance, high compressibility, some times high sensitivity and low permeability. The Indo-Gangetic alluvial belt is in addition subject to moderate to high seismic activity. The structures founded on these deposits have low bearing capacity, suffer high settlements, long consolidation times and effected by liquefaction in case of loose granular deposits subjected to earthquake loads. Amongst the various ground treatment alternatives, granular piles or stone columns are the most preferred choice in many instances because of the ease of construction, proven applicability and utilisation of low cost natural materials.

Provision of granular piles in soft ground or loose granular deposits, leads to significant improvement in the overall response of the ground because of (i) reinforcement and (ii) densification effects, and (iii) increased rates of dissipation of excess pore pressures generated either during static or seismic loading. The paper analyses various improvement schemes, viz., with respect to strength and stability, settlement and consolidation and lastly in mitigating earthquake disaster mitigation through prevention of liquefaction. In particular, some studies on the strength, settlement, consolidation and liquefaction resistance of granular pile reinforced ground have been presented. Validation based on limited field data has also been given to give confidence to practicing engineers on the viability of using this technique.

REFERENCES

Aboshi, H. and Suematsu, N. 1985. Sand compaction pile method: State of the art paper, Proc. 3rd Geotech. Seminar on Soil Improvement Methods, NTU, Singapore.

Balaam, N.P. and Booker, J.R. 1981. Analysis of rigid rafts supported by granular piles. Int. J. for Num. & Anal. Methods in Geomechanics, 5:379-403.

Barksdale, R.D. and Bachus, R.C. 1983. Design and construction of stone columns. Rep. No. FHWA/RD-83/026. Nat. Tech. Inf. Service, Springfield, Virginia.

Barron, R.A. 1948. Consolidation of fine-grained soils by drain wells. Transactions ASCE, 113:718-754.

Bergado, D.T., Anderson, L.R., Miura, N. and Balasubramaniam, A.S. 1996. Soft Ground Improvement in Lowland and Other Environments. ASCE, New York, NY, USA:186-233.

Datye, K.R. and Nagaraju, S.S. 1975. Installation and testing of rammed stone columns. Proc. IGS Spec. Session, V Asian Regional Conf. on Soil Mech. & Found. Engrg., Bangalore, 101-104.

Han, J. and Ye, S-L. 2001. Simplified method for consolidation rate of stone column reinforced foundation. J. Geotech. & Geoenvironmental Engrg., ASCE, 127(7):597-603.

Enoki, M., Yagi, N., Yatabe, R. and Ichimoto, E. 1991.Shearing characteristics of composite ground and its application to stability analysis, Deep Foundation Improvements: Design, Construction and Testing, Ed. by M.I.Esrig and R.C.Bachus, ASTM STP 1089:19-31.

Juran, I. and Guermazi, A. 1988. Settlement response of soft soils reinforced by compacted sand columns. J. GTE, ASCE, 114(8):930-943.

Kamon, M. and Bergado, D.T. 1991. Ground improvement techniques. Prof. 9th Asian Reg. Conf. on Soil Mech. and Found. Engrg., Bangkok, 2:521-548.

Madhav, M.R. 1982. 'Recent developments in the use and analysis of granular piles', Proc. Int. Symp. on Recent Developments in Ground Improvement Techniques, Bangkok, 117-129.

Madhav, M.R. and Arlekar, J. N. 2000. Dilation of granular piles in mitigating liquefaction of sand deposits. 12th World Conf. on Earthquake Engineering, Auckland.

Madhav, M.R. and Van Impe, W.F. 1993. Load transfer through gravel bed on stone column reinforced soil. J. of Southeast Asian Geotech. Soc., 47-62.

Ohbayashi, J., Harada, K. and Yamamoto, M. 1999. Resistance against liquefaction of ground improved by sand compaction pile method. Earthquake Geotechnical Engrg., ed. by Seco a Pinto, Balkama, Rotterdam:549-554.

Poorooshsb, H.B. and Madhav, M.R. 1985. Application of rigid plastic dilatancy model for prediction of granular pile settlement. 5th Int. Conf. on Num. Methods in Geomechanics, Nagoya, 1805-1808

Poorooshasb, H.B., Noorzad, A., Miura, N. and Madhav, M.R. 2000. Prevention of earthquake induced liquefaction of sandy deposits using stone columns. Proc. Int. Symp. on Lowland Tech., Saga, 213-220.

Van Impe, W.F. and De Beer, E. 1983. Improvement of settlement behavior of soft layers by means of stone columns. Proc. 8th EC SMFE, Helsinki, 1:309-312.

Van Impe, W.F. and Madhav, M.R. 1992. Analysis and settlement of dilating stone column reinforced ground. Osterreichische Ing. und Arch.-Zeitschrift, 137:114-121.

SOIL IMPROVEMENT USING LIME AND PADDY HUSK ASH FOR CONSTRUCTION OF RURAL ROADS

N.Mohadevan, K.M.G.D.N.Dias, H.N.Seneviratne, A.G.H.J.Edirisinghe

Department of Civil Engineering, University of Peradeniya

Abstract

In many parts of Sri Lanka rural road network is not sufficiently developed to meet the current transport needs. In many instances this is due to non-availability of good aggregate at economic prices in rural areas. Therefore, it was suggested to adopt appropriate road construction techniques based on locally available material resources. One such method is to use lime and paddy husk ash to improve the stability and the strength characteristics of road bases or sub bases. This is particularly effective in clayey soils. In the present study, a series of laboratory tests has been carried out with soaked and unsoaked stabilized specimens prepared with different mix proportions to evaluate the effectiveness of this method under local conditions. Samples have been taken from three different locations in Kandy District and it has been demonstrated that insitu soil in these locations could be satisfactorily stabilized to obtain CBR values higher than what was required for rural road construction. This paper also analyses the suitability and the importance of the lime-paddy husk ash stabilization technique in local rural road development projects. At the end of this paper a field procedure has been suggested for the construction of trial roads to verify the findings from the laboratory studies given above.

INTRODUCTION

In Sri Lanka the road network has been classified into four main classes namely A, B, C, and D; in addition there are minor rural roads which branch off from C and D class roads and traverse interior to remote villages. The rural roads are generally maintained by Local Government Bodies, Development Boards and Plantation Sectors. These rural roads play a vital role in the farming activities and all other development work of the area. However, the development of these road networks is restricted due to limitation of the availability of good aggregate at economical prices and in most of the cases the roads have been constructed with poor quality materials.

When the road base does not have enough bearing strength it undergoes differential settlements (Fig. 1). The situation become worse during rainy seasons as the depressions form water pools or mud holes, which may eventually make the road unusable (Fig. 2). On the other hand, during the dry season, a large amount of dust is generated which becomes an environmental problem.

Fig. 1 Differential settlement in a rural road

Fig. 2 Depressions eventually become water pools

Considering the above factors, there is a necessity to consider alternatives, such as improvement of the quality of poor materials available. One of the cheaply available materials for the road construction is the insitu soil on the road site. There have been many researches reported in the literature (Josi et al. 1985, Smith 1984, Khanna 1986, Flaherty 1990) on studies carried out to find suitable methodology to stabilize insitu soils. One of the suggested methods was to use lime and paddy husk ash to improve the strength characteristics of the sub base or base. For a rural road of capacity 45-150 vehicles exceeding 3 tons, a 150 mm flexible pavement (road base) having a CBR value of 20% is recommended (Khanna 1986). Further this stabilized soil could be used as a fill material to repair the potholes and other depressions in the road.

This paper analyzes the applicability of the lime paddy husk ash stabilization technique to improve the stability and strength characteristics of the sub base of rural roads where existing material is clayey soil.

STABILIZING TECHNIQUE

In the proposed method lime and paddy husk ash mix acts as a bonding agent by cementing the soil particles through a chemical reaction known as Pozzolanic reaction. Here the clay particles consisting of SiO_2 and Al_2O_3 and paddy husk ash consisting of SiO_2 act as Pozzolone and, when mixed with lime ($Ca(OH)_2$) the following reactions take place.

$$Ca(OH)_2 \longrightarrow Ca^{++} + 2(OH)^-$$

$$Ca^{++} + 2(OH)^- + SiO_2 \longrightarrow CaO.SiO_2.H_2O$$

(Calcium silicate CSH)

$$Ca^{++} + 2(OH)^- + Al_2O_3 \longrightarrow CaO.Al_2O_3.H_2O$$

(Calcium Aluminate CAH)

According to the literature (Flaherty 1987, Khanna 1986) a soil should have the following geotechnical characteristics to be satisfactorily stabilized.

Table 1 Geotechnical characteristics recommended for satisfactory stabilization.

Parameter	Value (%)
Clay Content	10~30
Plasticity Index	20~30
Liquid Limit	~50

COMPRESSIVE STRENGTH OF TREATED SOIL

In India and some other countries the lime-paddy husk ash stabilizing technique is also used to manufacture bricks. In Sri Lanka this is done on a minor scale.

The research on compressive strength of 250 mm mortar cubes with various proportions of lime and ash had been carried out in United Kingdom laboratories on the sample collected from India and Nepal and the properties thus obtained are given below in Table 2 (Smith 1984)

Table 2 Compressive strength tests results (Smith 1984)

Country	*Sample No*	*Proportion of Constituents by Weight (Lime:Ash)*	*Compressive strength in MN/m^2*	
			7 Day, 20^0 C	*28 Day, 20^0 C*
India	*1*	*1:2*	*2.1*	*4.6*
	2	*1:2.5*	*8.0*	*10.5*
Nepal	*3*	*1:3*	*6.4*	*10.6*

LABORATORY TESTS

In the present study a series of laboratory tests has been carried out to evaluate the performance of stabilizing agents with the samples collected from three different locations. First particle size analysis tests and Atterberg limit tests have been carried out to check whether the soil has correct characteristics to be stabilized.

The geotechnical properties of the soil samples are given in Table 3.

Table 3 Geotechnical properties of soil samples considered for stabilization.

Sample No	Location	Clay Content (%)	Liquid Limit (%)	Plasticity Index (%)
1	Hantana	10	52	22
2	Hal Oya	17	48	22
3	Udadumpara	20	48	23

From Table 1 and Table 3, it is clear that all the three samples have the required geotechnical properties.

Paddy husk ash derived from insitu burning was used for stabilizing samples 1 and 3 whilst that derived from controlled burning at 600^0 C, has been used for sample 2.

In preparation of specimens for samples 1 and 2, the mixtures have been prepared by adding lime and paddy husk ash together and the testing were carried out by Sirisena(1997) and Subentheran et al.(1999) respectively. In contrast, in testing sample 3 paddy husk ash and half of the required amount of water was added first and lime and the remaining water were added 24 hours later. This second procedure (Suresh 2000), described in detail in Appendix - A, was found to be more effective than the previous method. Further lime to paddy husk ash ratio is kept at 1:2 for all samples as recommended in the literature (Sirisena 1997). In addition seven more specimen have been prepared, three with untreated soil, two stabilized with lime only and the other two treated with paddy husk ash only.

DETERMINATION OF OPTIMUM MOISTURE CONTENT

The degree of compaction achieved is one of the major factors governing the stability and the strength characteristics of the stabilized soil, therefore it is important to prepare specimens at optimum moisture content to obtain the best results.

The results of compaction tests conducted on samples taken from the three sites are given in Table 4a-c.

Table 4a Compaction test results for Hantana housing scheme sample.

Ratio of mix Lime:Ash:Soil	*Optimum Moisture content/(%)*	*Maximum Dry density (Kg/m^3)*
0:00:100	*18.5*	*1600*
5:10:85	*22.0*	*1460*
6:12:82	*23.0*	*1440*
8:16:76	*26.0*	*1370*

Table 4b Compaction test results for Hal Oya sample.

Ratio of mix Lime:Ash:Soil	Optimum Moisture content/ (%)	Maximum Dry density (Kg/m^3)
0:00:100	15.	1550
5:10:85	26	1450
6:12:82	26	1430
7:14:79	27	1370
8:16:76	28	1340
9:18:73	28	1310
8:00:92	27	1440
0:16:84	18	1400

Table 4c Compaction test results for Ududumbara sample.

Ratio of mix Lime:Ash:Soil	Optimum Moisture content	Maximum Dry density/ (Kg/m^3)
0:00:100	17.95	1680
4:08:88	25.6	1510
5:10:85	24.8	1480
6:12:82	27.8	1470
7:14:79	31.5	1370

CBR TESTS

All the specimen prepared at optimum moisture content were subjected to CBR tests to determine the optimum mix ratio of the stabilizing agents. The results of soaked and unsoaked CBR tests at all locations are given in Figs. 2a-c and in Table 5.

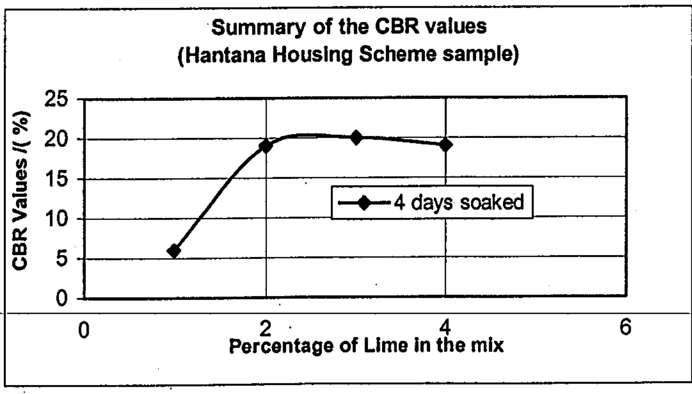


Fig. 2a

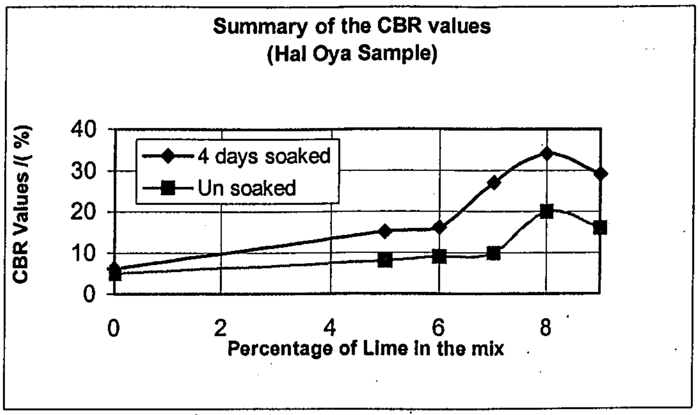


Fig. 2b

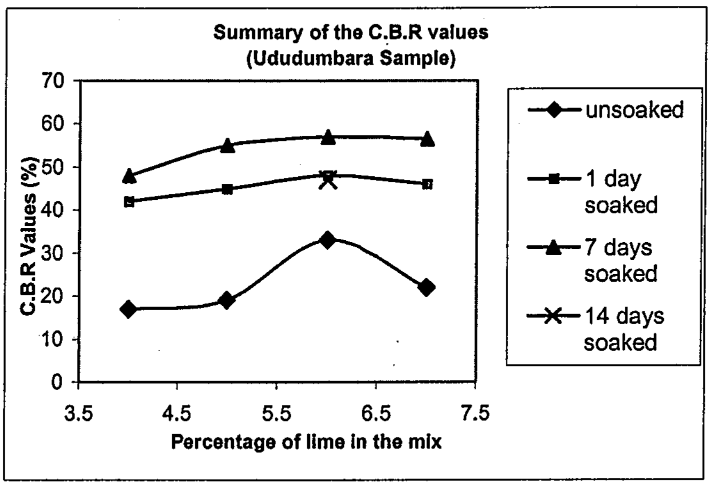


Fig. 2c

Table 5. CBR test results for un treated samples, samples treated with lime only and the samples treated with paddy husk ash only.

Sample No	Location	Untreated sample	CBR values (%)			
			Treated with lime only		Treated with paddy husk ash only	
			Un soaked	4 days soaked	Un soaked	4 days soaked
1	Hantana	6	-	-	-	-
2	Hal Oya	5	19	25	4	5
3	Ududumbara	12	-	-	-	-

DISCUSSION : LABORATORY TEST RESULTS

All three soils show improved strength after adding lime and paddy husk ash, and under optimum conditions CBR improved to a value acceptable for the design of a rural road pavement. For specimen prepared from soil samples 1 and 3, CBR values shows a peak of 20% and 57 % at 6% lime and 12 % paddy husk ash, and for the case of specimen prepared from sample 2 it was 34% at 8% lime and 16% paddy husk ash.

Higher CBR values have been obtained with Ududumbara (sample No 3) sample, this may be due to the special procedure followed to prepare the specimens. For the same sample there is a reduction in

CBR value with the 14 days soaked specimen, this may have been due to the softening effect caused by the increased moisture content. However, this value is well above the acceptable limit of 20% CBR.

For the case of sample taken from Hantana Housing Scheme (sample No 1), lower CBR values were obtained. The reason for this may be due to the low clay content in the sample, which may eventually reduce the amount of Pozzolone (SiO_2 and Al_2O_3) required for the stabilizing reaction . However, the optimum CBR value of 20% has been obtained with this sample, therefore the insitu soil at Hantana Housing Scheme could also be satisfactorily stabilize using the lime-paddy husk ash technique.

Hal Oya sample (sample location 3) has been treated with lime and controlled burnt paddy husk ash. In this case the tests were carried out to check the improvement with the control burnt paddy husk ash. Even though the geotechnical properties of this sample are somewhat similar to that of Ududumbara sample there is a significant difference in the CBR values. One of the possible reasons for this may be the special procedure followed to prepare specimen using the sample collected from Ududumbara.

Above results indicate the importance of adopting the correct procedure for stabilization. It also to a lesser extent demonstrate that insitu burnt paddy husk ash is a sufficiently good material for stabilization.

BENEFITS OF THE LIME AND PADDY HUSK ASH STABILIZATION

The benefits of the lime and paddy husk ash stabilization are as follows,

1. It improves the stability and the load bearing capacity of road bases.
2. It expedites construction, and saves time and money.
3. The strength of the stabilized base increase with time.
4. The stabilized road surface has ability to sustain small duration flooding.
5. The procedures are simple in technique, so can be easily carried out in the field.
6. The procedure can be easily adjusted to be more labour intensive thereby providing more employment opportunity to the local people.
7. Paddy husk ash is generally considered as a waste material. Disposal of it has become one of the problems faced by the rice mill owners and improper dumping of it may create environmental problems. Therefore, the adoption of this stabilization technique may provide some means of solving these problems by effectively utilizing the material, which is considered as a waste.

On the other hand large amount of dust may generate if the paddy husk ash is not carefully utilized. Therefore, necessary guidance should be given to the labourers involving in the stabilization process to minimize dust generation.

ECONOMIC CONSIDERATIONS

Sirisena(1997), carried out a cost analysis to evaluate the cost difference between lime and paddy husk ash stabilized roads and tarred or/cemented surfaced roads.He reported that up to 28% savings could be made using the lime and paddy husk ash stabilizing technique over conventional methods. This analysis had been carried out assuming that general type of machinery and resources are used in construction.

A cost analysis has been carried out by the authors assuming the use of labour intensive method with minimum use of machinery to suit rural conditions. The summary of the results is given below and the details of the analysis are given in appendix B.

To stabilize 1m length road having 2.5 m width,

		Amount/ (Rs)
[i] Lime - Paddy Husk Ash Stabilization (150 mm thickness)	=	721.00

[ii] 1:3:6 cement concrete excluding form work (100 mm thickness) = 1,040.00

[iii] Metal and tarring = 1,060.00
(As per RDA specification (4", 2", 3/4") metal with two coats of tarring for each layer)

Approximate savings from Lime and paddy husk ash technique (At current prices) 32% of Metal and tarring method

FIELD TRIAL AND FIELD PROCEDURE

Based on the methodology used in laboratory tests the technique of mixing prepared by Suresh(2000) is found to be the most effective. An appropriate field procedure based on this method and other literature is given in the appendix C.

CONCLUSIONS AND SUGGESTIONS FOR FUTURE WORK

It has been effectively demonstrated that lime and paddy husk ash stabilization can be used to improve the bases and sub bases of rural roads economically. At current prices cost savings up to 32% is possible when using this method. The method has several advantages including the low cost, use of locally available labour and material, less sensitivity to water penetration and environmental friendliness. Field trials should be conducted in the future to determine the optimum conditions for stabilization in the field and to fine-tune the procedure of stabilization to suit field conditions.

ACKNOWLEDGEMENT

The authors express their sincere thanks to Mr.A.Sirisena, Deputy Chief Secretary, Central Engineering Department, Central Province, Mr.M.Subentheran and Mr.Y.Thayaparan for their contribution, and Mr.P.Sureish, Assistant Director, Central Engineering Department, Central Province, for his assistance at various instances in carrying out the research.

REFERENCES

Craig, R.F., (1987). Soil Mechanics, 4th Edition, ELBS-London.

Flaherty, C.A.O.,(1987). Highways Vol 2, Highways Engineering , London.

Janarthan, G., Mohadevan, N., Parthiban, K., Seneviratnae, H.N., Edirisinghe, A.G.H.J., (2000). "SOIL STABILIZATION WITH LIME AND PADDY HUSK ASH" Project Day, Sri Lanka Geotechnical Society, 26-12-2000.

Joshi, R.C., Nagaraj, T.S., (1985). "Fly Ash Utilization for Soil Improvement", Symposium on Environmental Geotechnics and Problematic Soils and Rocks, Dec 1985,AIT, Bangkok, Thailand,

Khanna, P.N., (1986). Civil Engineering Hand Book, India.

Ministry of Highways, (2001). Highway Schedule of Rates in Metric Units, Ministry of Highways, Sri Lanka

Smith, R.,(1984). Rice Husk ash Cement- Progress in development and Application, London , 1984.

Sirisena, A.,(1997). " Design of Flexible Pavements of C & D Class and Village Type Roads in Sri Lanka Using Paddy-Husk-Ash and Hydrated Lime", MSc Thesis, Department of Civll Engineering, University of Peradeniya, Sri Lanka.

Suresh,P.,(2000) Internal report on soil stabilization with lime and paddy husk ash for rural road construction, Department of Civil Engineering, University of Peradeniya.

Sterwood, P.T., (1993). Soil stabilization with cement and lime: State-of -the-Art Review, Transport Research Laboratory, department of Transport, USA.

Subentheran, M., Thayaparan, Y., (1999). "Soil Stabilization With Lime and Paddy Husk Ash". Undergraduate Project Report No C/1999/06, Department of Civil Engineering, University of Peradeniya.

Appendix-A

A SPECIAL PROCEDURE TO PREPARE THE SPECIMEN FOR CBR TESTING.

A special procedure (Suresh 2000) has been followed to prepare the specimen for the lime and paddy husk ash stabilization of clayey soil. The steps involving in the special procedure are listed below,

1. Hand mix each batch of soil with half of the required moulding water and then allowed for 24 hours in a humid chamber.
2. Add the remaining water.
3. Mould the specimen by two-end static compaction.
4. Allow the specimen for another 24 hours in the atmosphere for preliminary curing.
5. Carry out the CBR test immediately for the un-soaked case CBR test, and for the soaked cases keep the moulded specimens in a 100% humid environment for the soaking period and carry out the CBR test.

Appendix-B

COST ESTIMATE FOR THE RAW MATERIALS
(site price)

Lime, paddy husk ash and water are the basic raw materials required for the proposed stabilization technique.

The site cost of these materials can be broken down as shown in the specimen calculation below.
Specimen Calculation:

(i) Lime(50 kg Bag):

Haul Distance 10 miles (assume)
(Transport by tractor/ trailor)

		Amount / (Rs)
Unit Price/Bag	=	175.00
Transport Cost	=	13.54
Loading Charges	=	2.60
Unloading Charges	=	2.40
Total for 1 Bag at site	=	193.54

(ii) Paddy Husk Ash (15 kg Bag)

Haul distance 1 mile by Tractor trailor

		Amount/(Rs)
Unit Price/Bag	=	0.00
Transport	=	2.47
Loading	=	1.30
Unloading	=	1.20
Total for 1 Bag at site	=	4 .97

(iii) Water		Amount/(Rs)
Cost for 1 gallon	=	0.10

Cost estimate to stabilize 10 X 2.5 m^2 area
(Thickness of the Sub Base –150 mm)

This cost estimate have been made assuming that the labour force is used for all the activities.

[a] Ash and Lime

Assuming dry density of the soil to be 1.8Mg/m^3

Weight of the Earth	=	6750 Kg
Mix Ratio – Lime : Ash : Soil	=	5 : 10 : 85
Mass of Ash	=	800 kg
Mass of Lime	=	400 kg
		Amount/(Rs)
Total Site Cost for Paddy Husk Ash (Rs)	=	265.00
Total Site Cost for Lime (Rs)	=	1550.00

[b] Labour /Machinery		Amount / (Rs)
1.Collecting Paddy Husk ash & put into gunny bags and tightening-53 bags 1 day unskilled labour	=	248.00
2.Softening the Soil (medium hard) Hand tractor 4 hours	=	800.00
3.Mixing the stabilizing agents, Hand tractor 4 hours @200	=	800.00
4. For spreading and manual mixing one day labour	=	248.00
5.Compacting the soil (Vibrating rammer – 60 kg) 8 hrs	=	1700.00
Total (Labour + Machinery) Cost	=	3786.00

[c] Surface Coating

Cold Bitumen using 1 l/m^2 including blending with sand at the rate of 250 m^3 /m^2 and brushing, cleaning and moisture the road surface 25@23.10 /m^2	=	580.00

[c] Water

Assuming the OMC of the soil is 28% and the Natural water content be 10% Mass of water	=	1430 kg
Cost associate with water	=	32.50

Total Cost (Basic Cost)	=	6213.00
13% profit	=	808.00
Total Cost for the stabilization	=	7021.00

Appendix-C

RECOMMENDED FIELD PROCEDURE:

A field procedure has been proposed for the construction of the trial road.

This procedure consist two parts, the first part is the preliminary analysis involving in checking the availability and the suitability of resources and the second part is the construction of the trial road base.

PRELIMINARY ANALYSIS

This preliminary analysis is a kind of reconnaissance survey to check the availability of the resources and the field conditions before the commencement of the construction/stabilization process. The key steps involving in the preliminary analysis are listed below.

I. Check the availability of the resources especially the paddy husk ash at a reasonable price at the site.

II. Check the Geotechnical properties of the insitu soil:

Mechanical analysis and Atterberg limit tests have to be carried out to check the geotechnical properties. For small scale projects suitability of the soil can be roughly determined by threading method.

III. Estimate the required quantity of stabilizing agents, labour and other overhead charges. (An estimate for 10 X 2.5 m^2 pavement is given in appendix B.)

FIELD PROCEDURE : Construction of a stabilized road base with lime - paddy husk ash mix (Labour intensive)

The following steps are recommended to carryout for the construction of the trial roads.

1. Loosening the insitu soil up to the required depth (150 mm). (machinery –hand tractor)
2. Rake the soil at the mixing locations to distribute the stabilizing agents.
3. Distribute the paddy husk ash at the raked locations and mix it thoroughly with the half of the required water. (machinery –hand tractor)
4. Cover the locations with water proof material. (Sheets, polythene etc.)
5. In the following day distribute the lime and water explained for the case of paddy husk ash.
6. Again thoroughly mix the layer. (machinery –hand tractor)
7. Compact the layer up to recommended density; here again field specimens have to be taken to determine the degree of compaction.
8. In the following day (3rd day) Water curing has to be carried out and from the fourth day traffic can be allowed over the pavement while sufficient wetting (curing) is maintained for another six days.
9. Proper drainage should be provided to ensure that no water logging in the pavement area.
10. After ten days apply surface coating.

As stated in the conclusion these field procedures should be fine tuned after determining the optimum field conditions for the stabilization.

CEMENT STABILIZED SOILS FOR EROSION AND SEEPAGE CONTROL OF IRRIGATION STRUCTURES.

K.M.G.D.N.Dias, H.N.Seneviratne, A.G.H.J.Edirisinghe

Department of Civil Engineering, Faculty of Engineering, University of Peradeniya.

ABSTRACT.

A study has been undertaken to find the applicability of stabilization to improve soils available for construction of canals and canal linings. A series of classification, wetting and drying, permeability, and unconfined compressive strength tests was performed on a soil sample taken from Wahalkada area to investigate the suitability of cement stabilization in canal construction and to find the optimum cement content for stabilization. In addition to the above tests slake durability tests was conducted to find suitability of this test for determining erosion resistance of treated soil. The tested soil had British classification of SFL, Sand with fines of low plasticity. Cement contents of 2%, 4%, 6%, 8% and 10% by weight were used in the series of tests.

The wetting and drying tests show that 6% cement content is the optimum value for durability of the stabilized soil. However, it is observed that addition of 4% cement brings down the permeability of the mixture after 7 days to an impervious level suitable for canal construction, which is in the order of 10^{-8} m/s. This is of a three-fold reduction of the permeability of the untreated sample.

It was also observed 5% cement is necessary to increase the strength of the soil to a satisfactory level. However placing a clay lining above the soil may bring down this requirement to about 4% of cement. Slake durability test also produced some interesting results but indicated a slightly higher cement content of 8% for optimum stabilization. Further tests should be conducted to come to a firm conclusion in this case. Considering the series of test results the use of 6% of cement or the use of 4% of cement with a suitable technique such as surface protection of the canal lining using a clay lining may be satisfactory for design purposes. However, a field trial is necessary to verify the laboratory findings and to find the optimum field procedures.

INTRODUCTION.

Most of the irrigation structures such as small and medium high dams and main canals, branch canals and field canals are made of earth. If the soil is to be suitable for such construction it should have low permeability, high erosion resistance, higher durability, good workability, and less dispersibility.

When suitable material is not available engineers will have to consider other alternatives such as concrete dams and concrete lined canals. Irrigation structures using concrete would not be favourable in many cases when considering the cost factor. Soil modification either by mechanical or chemical stabilization would be a wise choice.

Soil stabilization is the process by which natural soils and aggregates are improved for engineering use. More commonly soil stabilization is recognized as the improvement of natural materials by adding a small proportion of stabilizing agent/s, normally less than 10%. Cement, lime, bitumen, paddy husk ash and some chemicals such as sodium additives are the commonly used stabilizing agents.

Selecting cement as the stabilizing agent has the principal advantage that almost all soils are amenable to this technique. The durability of soil cement is of a high order and its strength is known to increase with age. In common practice cement stabilization is often considered economically viable with low and medium plasticity gravels and rock aggregates. Single sized material with a low uniformity coefficient (less than 3) requires high cement contents to achieve required strength because of their high void ratios. In this case cement will act as both filler material and bonding agent, as the pores are to be replaced by cement. Well-graded soils having uniformity coefficient more than 5 can be stabilized more economically as they have low void ratios.

Materials containing organic matter cannot be stabilized with cement because it inhibits the setting of the cement. The safer upper limit is 2% although soils with 3-4% organic matter have also been successfully stabilized with cement. Certain clayey soils are not suitable for cement stabilization due to the difficulty of pulverizing the soil and due to high amount of cement required. It is desired that clay content is restricted to less than 5%.

The use of the cement-stabilized soils for irrigation structures began in 1940's. Bonny test section on the Bonny reservoir in Eastern Colorado, which was built in 1951, was the first structure, which proved, cement stabilized soil as a low cost, durable construction material for dam construction. According to the tests carried out on Bonny test section, Holtz & Walker (1962) found that 12.5%-13.5% cement by volume is required to assure 6% weight loss after 12 cycles of wetting and drying test (ASTM D559-44) for silty fine sand and silty fine to coarse sand.

Kahir et al. (1988) found that a soil cement specimen should posses a minimum unconfined compressive strength of 1.724 MN/m^2 after 7-day curing to withstand stress induced by the sun and rain.

PROJECT AREA CONSIDERED.

Most of the irrigation canals of Wahalkada Scheme, which is in Anuradapura district, are earthen canals. The duty of this scheme is 8 Ac.ft/Ac during Maha season which is very much higher than the specified value of 3 Ac.ft/Ac and the actual island average of 4.5 Ac.ft/Ac. Therefore, the above scheme was selected for the present study.

METHOD OF STUDY.

Disturbed soil samples for laboratory experiments from Wahalkada area were collected according to BS 1377 Part 1. Mechanical analysis, atterberg limit tests and standard proctor compaction tests were performed on samples to classify the soil. Falling head permeability test was carried out to find the coefficient of permeability. All the tests were repeated two times and the average of the results were taken.

Standard proctor compaction tests were carried out on specimens to find out the optimum moisture content and maximum dry density. Unconfined compression tests were carried out according to ASTM D 1632-63 and ASTM D 1633-84. The diameter of the specimens used was 100 mm and height to diameter ratio was 2:1. Wetting and drying tests were carried out according to ASTM D 559-82 and ASTM D 558-82 to determine the minimum amount of cement required in soil-cement to achieve degree of hardness adequate to resist field weathering.

Slake durability apparatus, which is conventionally used to find the resistance offered by rock samples to weakening and disintegration when subjected to two standard cycles of drying & wetting. The specimens are placed in perforated drums and material dislodged after 200 revolutions was measured to obtain the slake durability index. Slake durability test which was proposed by Franklin and Chandra (1972) was carried out according to ASTM D 4644-00 to find out the suitability of this test for determining erosion resistance of the treated soil. The slake durability apparatus is shown in Figure 1.

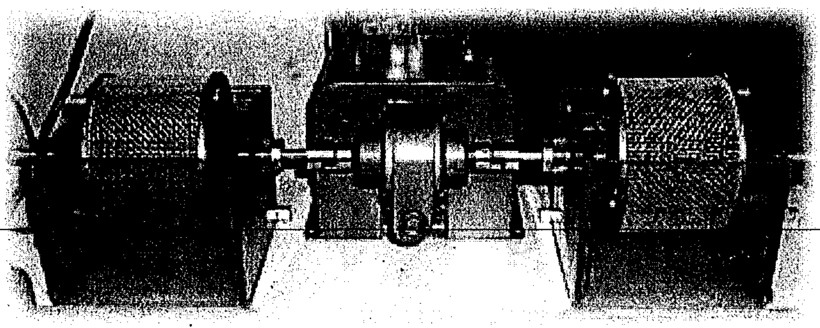


Figure 1. Slake Durability Apparatus.

RESULTS

The soil selected for the study consists 8% of clay, 11% of silt, 79% of sand and 2% of gravel. Liquid limit is 23% and Plasticity index is 6%. The soil selected falls under the category of Silty or Clayey SAND with low plasticity SFL according to BSCS.

According to the standard Proctor compaction test results shown in Table 1 optimum moisture content is 9.5% and maximum dry density is 1.99 Mg/m^3 for the natural soil. The optimum moisture content decreased to 8.5% and maximum dry density increased to 2.03 Mg/m^3 with addition of 6%-10% cement to the natural soil.

Table 1. Optimum moisture content and maximum dry density of soil-cement mixture.

Cement content by weight/ %	Optimum moisture content/ %	Maximum dry density/ (Mg/m^3)
Untreated	9.5	1.99
2	9.0	2.00
4	9.0	2.02
6	8.5	2.03
8	8.5	2.03
10	8.5	2.04

Weight losses after 12 cycles of stabilized samples subjected to wetting and drying test (ASTM D 559-82) are given in Table 2 and Figure2. It was not possible to carry out this test for untreated sample, as its strength is too low for making test specimens. The wetting and drying test result show that percentage loss after 12 cycles decreased with the addition of cement. As shown in Figure 2 percentage loss after 12 cycles decreased rapidly from 30.3% with 2% cement to 5.1% with 6% cement. Further increment of cement content showed only a little variation in weight loss.

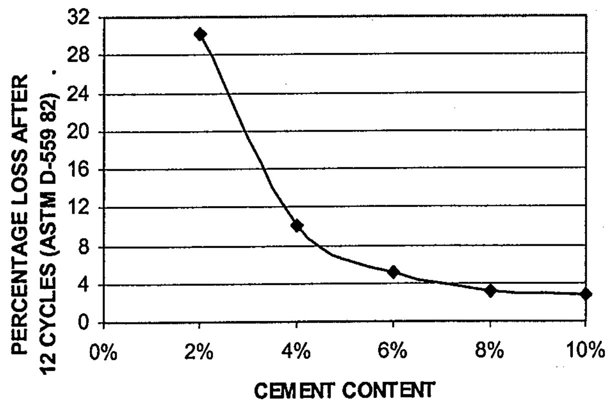


Figure 2. Variation of loss with cement content.

Table 2. Weight loss after 12 cycles of wetting and drying test.

Cement content by weight / (%)	Percentage loss after 12 cycles
2	30.3
4	10.1
6	5.1
8	3.1
10	2.8

The weight loss of samples for each wet-dry cycle is shown in Figure 3. It can seen from the figure that with 2% and 4% cement the soil cement loss per cycle increased with each cycle and with cement content 6% and greater the weight loss per cycle was approximately constant.

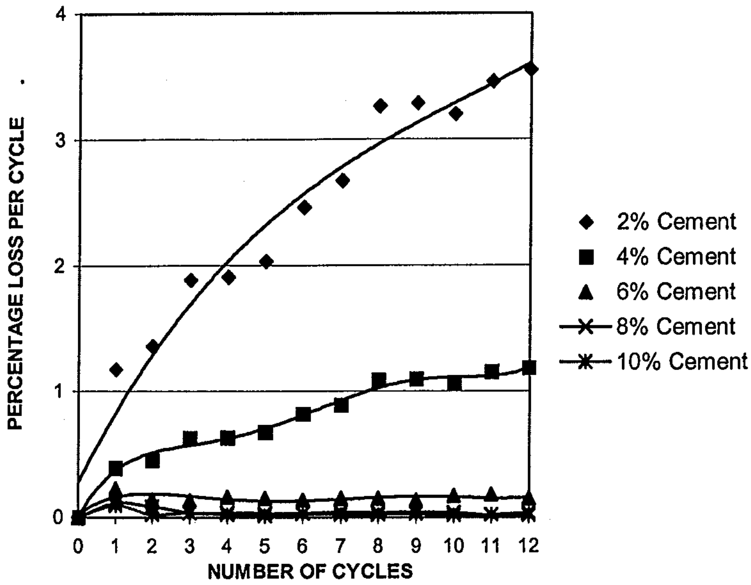


Figure 3. Comparison of soil cement loss per cycle.

Falling head permeability test was carried out on untreated specimens and 2% and 4% cement added specimens. Permeability was measured for 7 days and results are shown in Table 3 and Figure 4. The results show a reduction in permeability with time and after about four days the coefficient of permeability reaches a constant value for untreated specimen. This may be due to gradual saturation of untreated specimen. It is observed that addition of 4% cement brings down the permeability of the stabilized soil after 7 days to an impervious level suitable for canal construction, which is in the order of 10^{-8} m/s. This is of a three-fold reduction of the permeability of the untreated sample.

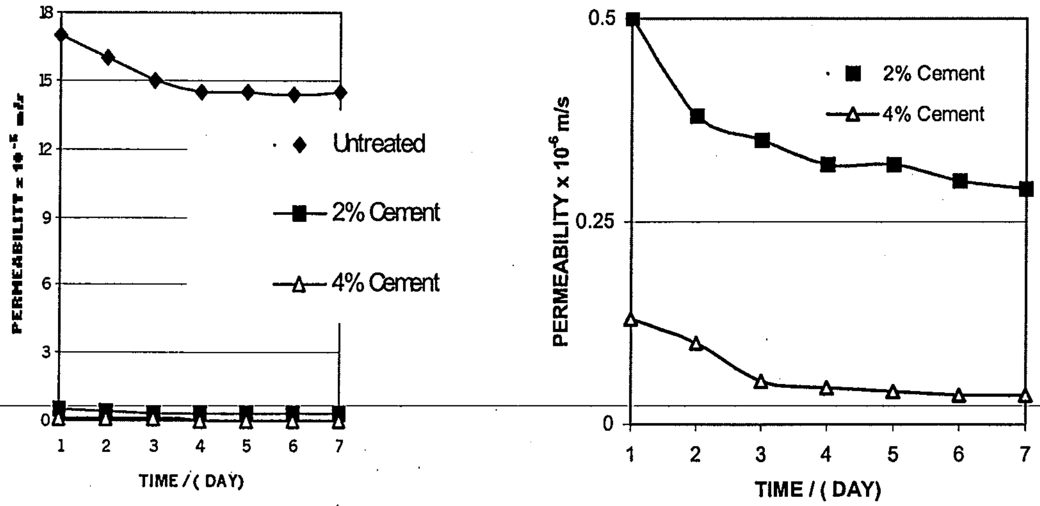


Figure 4. Variation of permeability with time.

Table 3. Variation of coefficient permeability in m/s

Day	Cement content by weight		
	Untreated	2%	4%
1	17.0	0.50	0.130
2	16.0	0.38	0.100
3	15.0	0.35	0.053
4	14.5	0.32	0.045
5	14.5	0.32	0.040
6	14.4	0.30	0.036
7	14.5	0.29	0.036

The unconfined compressive strength results (Table 4 and Figure 5) showed an increase in strength with the increase in cement content and with time. It showed a higher strength gained after 7 days curing when compared with 1 day cured sample; the gain of strength beyond seven days was not substantial when compared with gain within the first seven days.

According to the strength limit proposed by Khair et al. (1988) about 5% cement is required to satisfactorily stabilize this soil to prevent degradation by sun and rain. However, if a protective lining can be placed above the soil, to prevent degradation even 2% cement may satisfactorily improve the strength of the soil.

Table 4. Variation of compressive strength in MPa

Curing period / Days	Cement content by weight				
	2%	4%	6%	8%	10%
1	0.128	0.0972	1.495	2.292	3.303
7	0.486	1.585	2.96	3.808	5.377
28	0.652	1.791	3.376	4.643	5.998

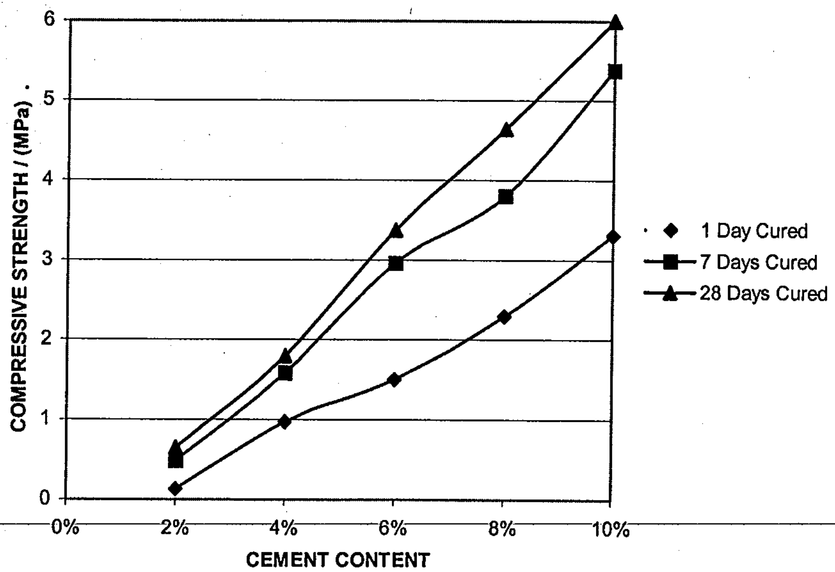


Figure 5. Compressive strength test results.

Slake durability test results are shown in Table 5 and Figure 6. Slake durability test results indicated a slightly higher optimum cement content of 8% for stabilization; in contest the wetting and drying test indicated an optimum value of 6% cement. It also showed an increment in slake durability index with addition of cement. The values increased rapidly up to a cement content of 8% and the gain beyond 8% cement content was not significant. The condition of the specimens after 2 cycles of wetting and drying test cycles in the slake durability apparatus are shown in the Figure 7.

Table 5. Variation of slake durability index in %

Curing period / (Days)	Cement content by weight				
	2%	4%	6%	8%	10%
1	0.00	24.30	49.96	68.69	74.53
7	9.12	49.32	75.14	88.54	93.35
28	20.00	54.74	79.55	91.84	95.58

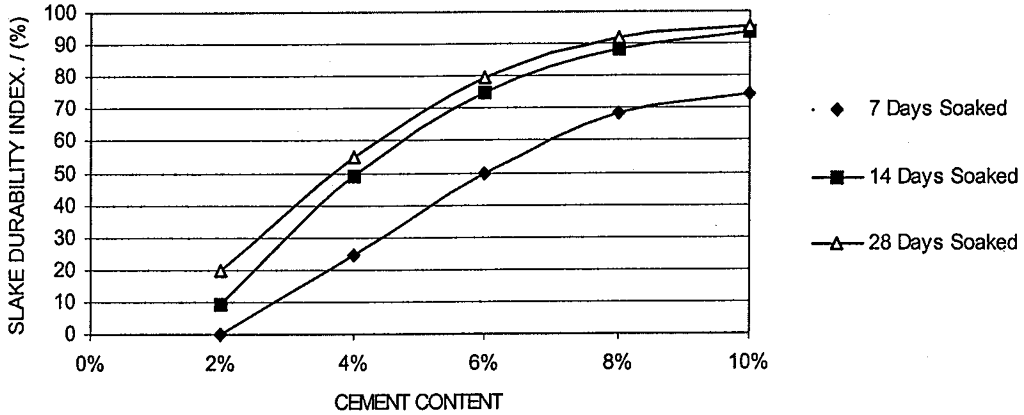
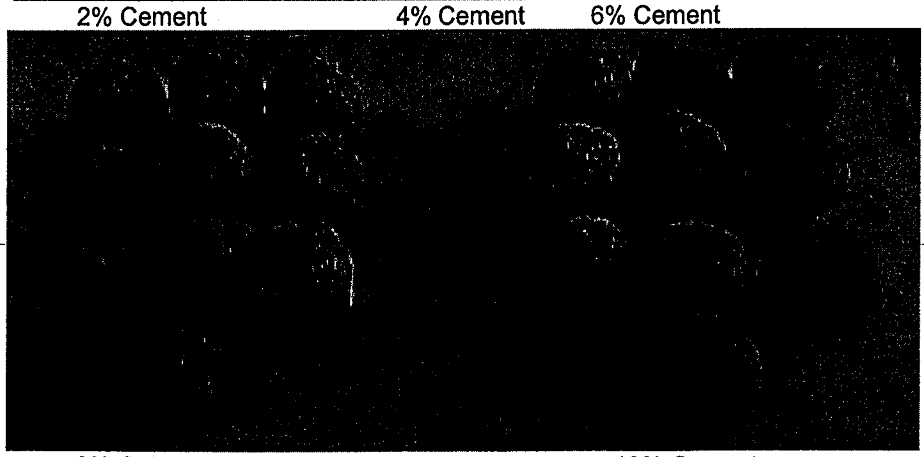


Figure 6. Variation of slake durability index.

2% Cement 4% Cement 6% Cement



8% Cement 10% Cement

Figure 7. Slake durability test samples after two cycles of wetting and drying.

CONCLUSIONS & RECOMENDATIONS.

The experimental results demonstrate that Wahalkada soil could be effectively stabilized using cement. A cement content of 4% for satisfactory improvement of permeability of the soil and 5% for strength was adequate. However for erosion control 6% of cement was required as given by wetting and drying test and 8% cement required as given by slake durability test. The results demonstrated that the soil stabilize even with 4% cement provided that erosion control could be achieved through some other means.

The laboratory results also demonstrated that slake durability test also can be used to find the suitability of cement-stabilized soil as an erosion resistance material. However more research is necessary to come to a firm conclusion in this case.

Field trials should be conducted to verify the above results under field condition and also to find appropriate field construction techniques. Field trials can be used to test the effectiveness of using a low content of cement for stabilization with erosion resistance provided by some other means for example by using a thin clay lining. In local context this may also prevent accidental damage to the embankment or canal lining by actions of humans and animals specially buffaloes.

REFERENCES.

Baghdadi Z.A. and Shihata S.A. (1993), "On the durability and strength of soil-cement", Ground Improvement Journal, Vol. 3, pp 1-6.

Craig R.F., Soil Mechanics handbook fifth edition.

Division of Geotechnical & Transportation Engineering, Asian Institute of Technology, (1984), " Laboratory manual for rock tasting". pp 40-44.

Holtz W.G. and Walker F.C. (1962), "Soil-Cement as slope protection for earth dams". Proceedings of the American Society of Civil Engineers, Journal of the soil mechanics and foundations division, Vol. 88, December, SM 6, pp 107- 134.

Kadiyali R. (1989), "Principles and practice of highway engineering", pp 459- 467

Khair A., Nallurri C. and Kilkenny W.M. (1991), " Soil-cement tiles for lining irrigation canals" Journal of the Irrigation and Drainage Systems-5, pp 151-163.

Moh Z.C. Chin Y.P. and Cang S.C. (1967), " Cement stabilization of lateritic soils", Proceedings of the third Asian regional conference on soil mechanics and foundation engineering, pp 42-46.

Shen C.K. and Akky M.R. "Use of cement stabilized soils in canal lining", Proceedings the fifth Southeast Asian conference on soil engineering, Bangkok Thailand pp 401- 411.

Shen C.K. and Akky M.R. "Erodibility and durability of cement stabilized loam soil" Transport research board no. 501, pp 54-58.

Za-Chieh Moh. (1962) "Soil stabilization with cement and sodium additives", Proceedings of the American Society of Civil Engineers, Journal of the soil mechanics and foundations division, Vol. 88, December, SM 6, pp 81-105.

REINFORCEMENT OF GROUND AND SOILS - PRINCIPLES AND MECHANISMS

Madhira R. Madhav

Professor of Civil Engineering
Indian Institute of Technology, Kanpur-208 016.

ABSTRACT

Reinforcement of ground and earth structures is probably the most popular form geotechnical practice these days. The practice of geotechnical engineering has received significant boost because of the rediscovery of the principles of reinforcement. The paper reviews the principles and practices of reinforcement in geotechnical engineering and the mechanics of reinforcing the ground. In particular, the function of the reinforcement in the forms of compression, shear, flexure and tension are discussed. The principles enunciated are valid for reinforced slopes as well.

INTRODUCTION

The present era is one of innovation. In geotechnical engineering, recent advances have provided a variety of ground treatment techniques (Table 1) that facilitate construction in any difficult ground conditions or with any kind of soil inclusive of even waste materials. Materials used for reinforcement (Table 2) also have a wide spectrum from strong steel, corrosion resistant and non-biodegradable geosynthetics to the so-called waste materials such as tyres, used plastic, etc.

Table 1 PRINCIPLES AND TECHNIQUES FOR ENGINEERING OF GROUND

PRINCIPLE	*TECHNIQUE*
Driving	Nails & Micropiles
Injection	Lime/Cement Mixed Piles
Deep Mixing	Dry or Wet Jet Grouting
Dynamic Methods	
a. Vibrations	Vibro-compaction & Vibro-replacement
b. Heavy Tamping	Dynamic Replacement, Dynamic Replacement & Mixing
c. Vibratory Probes	Vibro-wings & Resonant Compaction
Boring & Dynamic Methods	Rammed Stone Columns
Near Surface Constructions	Reinforced earth Walls, Reinforced Embankments, Reinforced Foundation Beds

Table 2 Reinforcement Materials

	TYPE	*TECHNIQUE*
1.	Historical	Reeds, mats, timber, etc.
2.	Natural	Granular Piles / Stone Columns Sand / Gravel Compaction Piles
3.	Traditional Civil Engrg. Materials	Lime/ Cement Piles, Nailing, Micro-piles, Anchors, Dowels, Embankment Piles, Slope Stabilising Piles, Grouting
4.	Geosynthetics	Reinforced Earthwalls, Reinforced Embankments, Embankments on Soft Ground, Reinforced Foundation Beds
5.	Waste Materials	Used Tyres, Fly Ash, etc.
6.	Combinations	Combinations of the above and several other methods.

OBJECTIVES

Improvement of the engineering properties of ground or a soil mass (Bergado et al. 1996) implies any one or more of the following objectives :

1. Imrovement of strength for increase in stability, reduction of earth pressures or increased resistance to erosion;
2. Reduction of settlements or deformations consequent to increase in stiffness;
3. Control of volumetric changes due to environmental effects - swelling and shrinkage;
4. Increase or reduce permeability as per the requirement of the project;
5. Increased resistance to effects of liquefaction;
6. Improved performance with respect to cyclic loading, alternate cycles of wetting and drying etc., and
7. Account for natural variability in soils to achieve relatively lower factors of safety.

Amongst the various alternatives available (hydraulic, physical, chemical, mechanical, densification, replacement, confinement, reinforcement, etc.) for dealing with either a difficult site or soil of inadequate strength, reinforcement is one that revolutionized the practice of geotechnical engineering. Soft soils are reinforced by granular piles (stone columns) soil lime or cement piles, inclusions created by deep mixing or jet grouting, to build medium to heavy structures including high embankments (Table 1). Loose sands are reinforced by vibro -compaction or heavy tamping, while expansive soils by anchors or tension resistant members. Slopes are reinforced (Fig. 1) by piles, nails, anchors, dowels, reticulated micropiling, etc. Thus ground and slopes are reinforced by a large variety of methods and concepts, the reinforcing elements being installed by driving, compaction, grouting, etc.

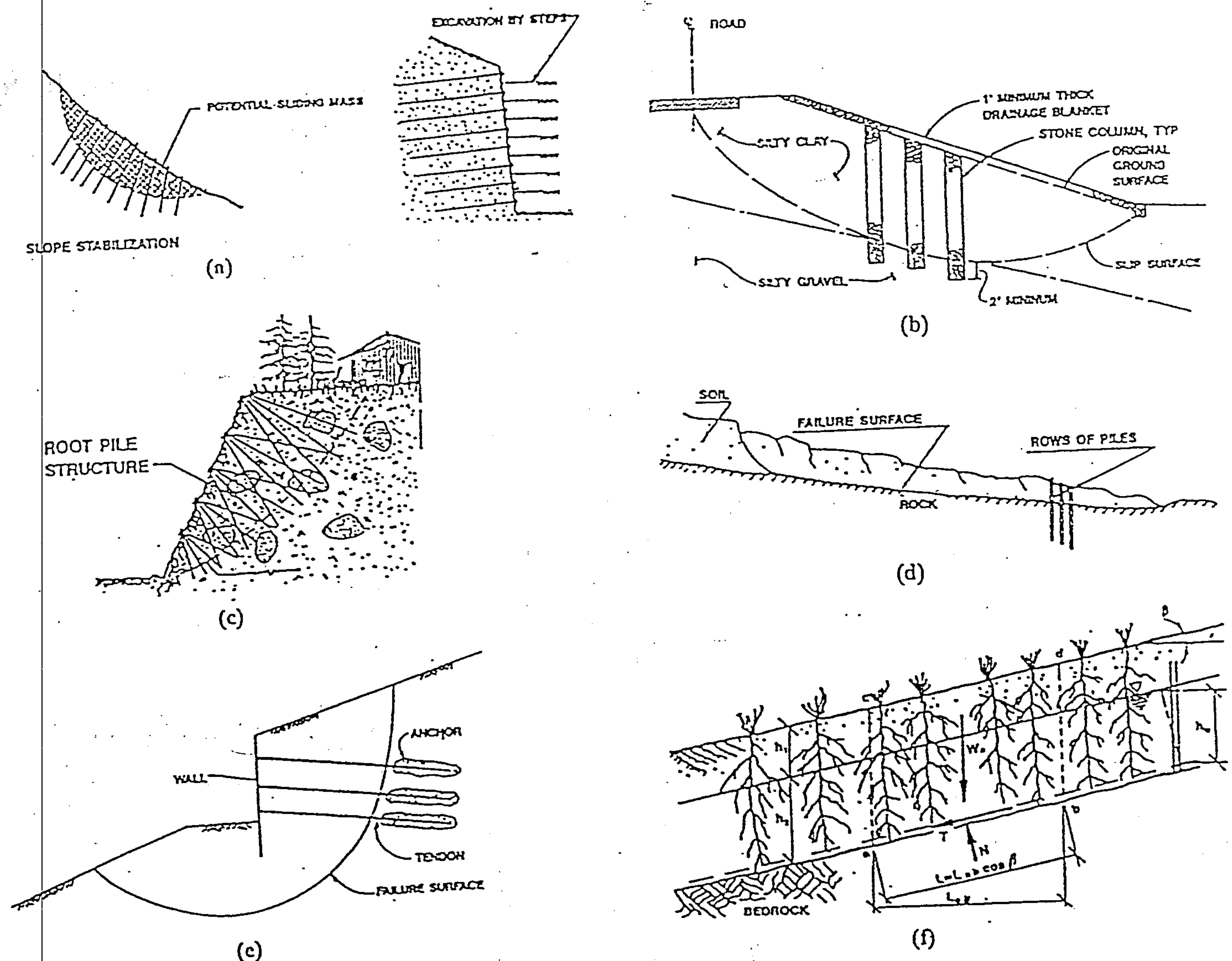


Fig. 1 Examples of Reinforcement of Slopes

Conventionally, earth retaining structures have been built external (Fig.2a,b & c) to the soil being retained and designed based on the theory of earth pressures of Coulomb or Rankine. However, there has been a remarkable growth in innovativbe technologies and products to retain soil and to construct earth structures. Amongst the various techniques available now, the concept of reinforcement (Fig. 2d to h) is the most popular. Earth structures, viz., retaining walls and embankments, are built by the concept of incremental burial of the reinforcement.

Often, two or more techniques are combined to achieve faster and more economical construction of earth structures. Thus as shown in Fig. 3a, basal reinforcement below an embankment is coupled with vertical drains (Fig. 3b) to achieve faster rates of consolidation or with piles (granular or otherwise) (Fig. 3c) to reduce settlements. Fig. 4 depicts the relation between time and required reinforcement tension. Unreinforced condition requires very large times for the in situ soil to gain sufficient strength to bear the embankment or structural loads. If the rate of consolidation is accelerated, less reinforcement tension needs to be provided.

Very soft almost fluid like reclaimed ground can be stabilized with reinforcement (Fig. 5) Ramu (2001) describes, models and analyses the process of reclamation of soft ground. Fig. 6 displys basal reinforcement beneath a sea wall / causewy. Several instances of reinforcing soft ground (Fig. 7) prior ro tunneling also are available.

MECHANISMS

The inclusions provide reinforcement action in compression, shear, flexure / bending or tension depending on the mechanism to be achieved. Thus granular piles reinforce soft ground by functioning s relatively stiff compression elements. Nails increase stability of slopes by the tension mechanism while dowels provide increased shear resistance across the failure surface. In most earth structures, the tensile resistance of the reinforcing element is the one that provides for the improved performance of the facility.

REINFORCING ELEMENTS

Traditionally, reeds, mats and timber were used to reinforce soils as in nearly three thousand year old Juggarats in Iraq.Currently (Table 2), granular materials, lime of cement mixed or jet grouted soils, for reinforcement in compression or shear. Steel and galvanized iron bars, strips or grids / meshes and geosynthetic sheets, meshes, grids, nets, etc. are being utilized for reinforcing soils and earth structures in tension and flexure. Anchors (Fig. 8) strengthen soils by mobilizing passive resistance. Reinforced concrete elements provided to increase stability of slopes and embankments function predominantly in flexural mode.

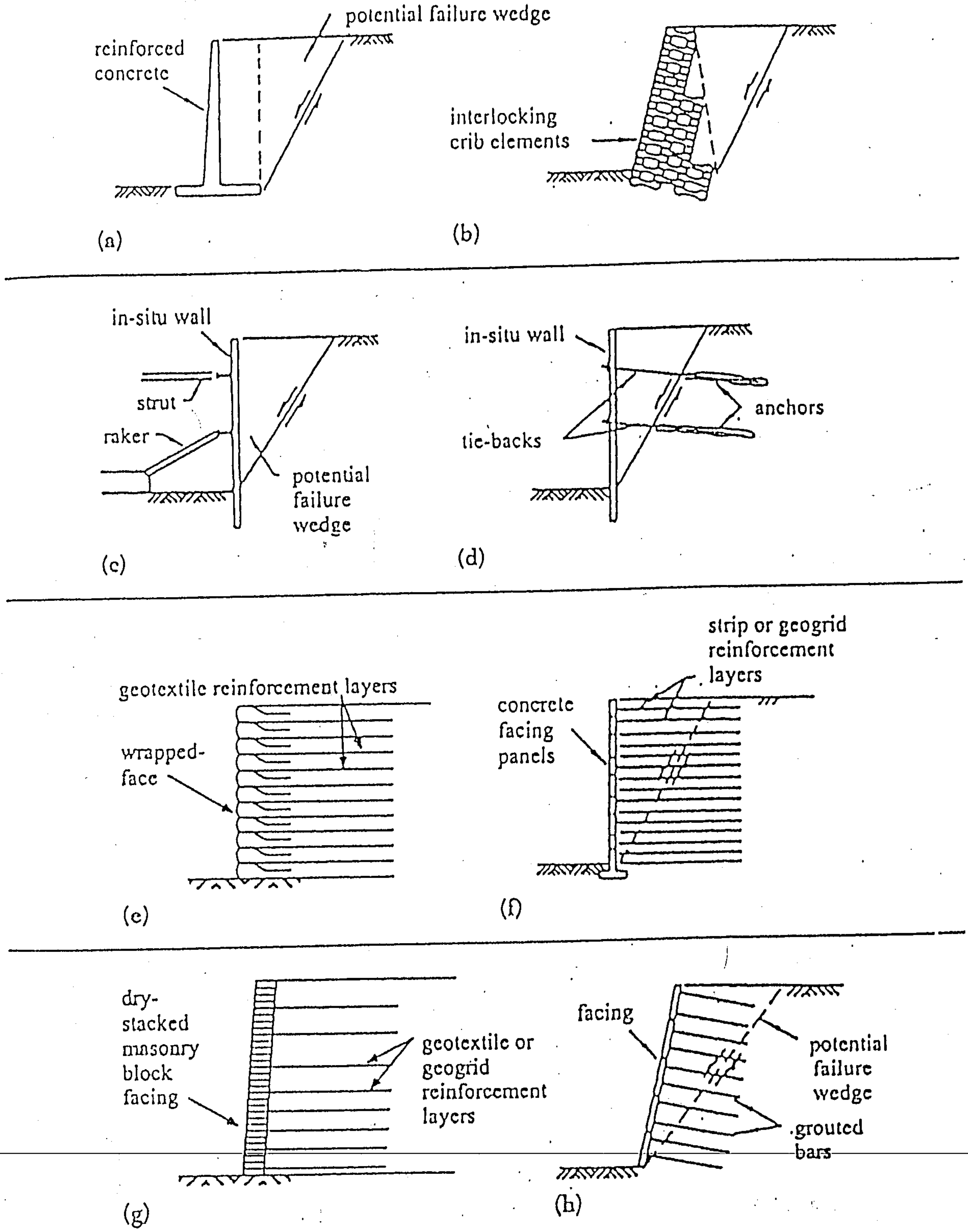


Fig. 2 Earth Retaining Structures; (a) Cantilever; (b) Crib; (c) Braced; (d) Tie-back (Anchored); (e) Wrapped Geosynthetic Reinforced; (f) Reinforced Wall; (g) Reinforced Segmental Wall; (h) Soil Nailing

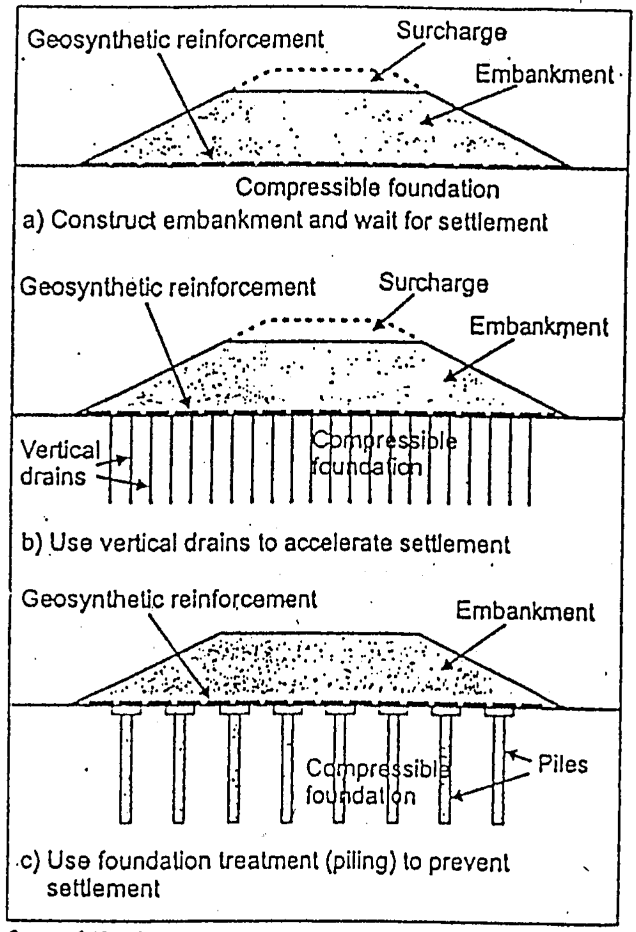


Fig. 3 Basal Reinforced Embankment with (a) Surcharge; (b) Vertical Drains and (c) Piles

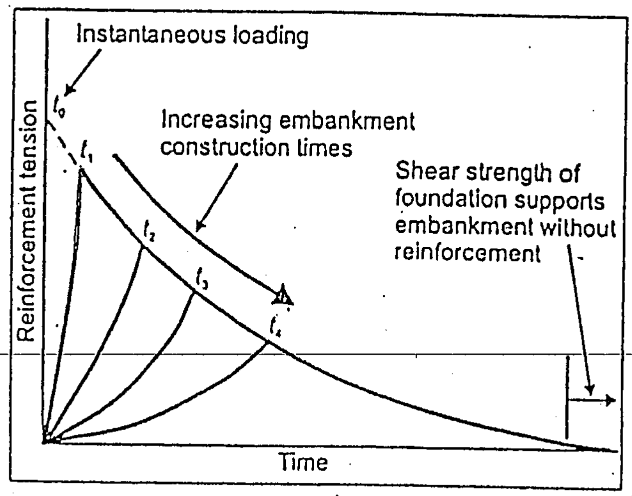


Fig. 4 Effect of Loading Rate on Maximum Required Tension in the Reinforcement

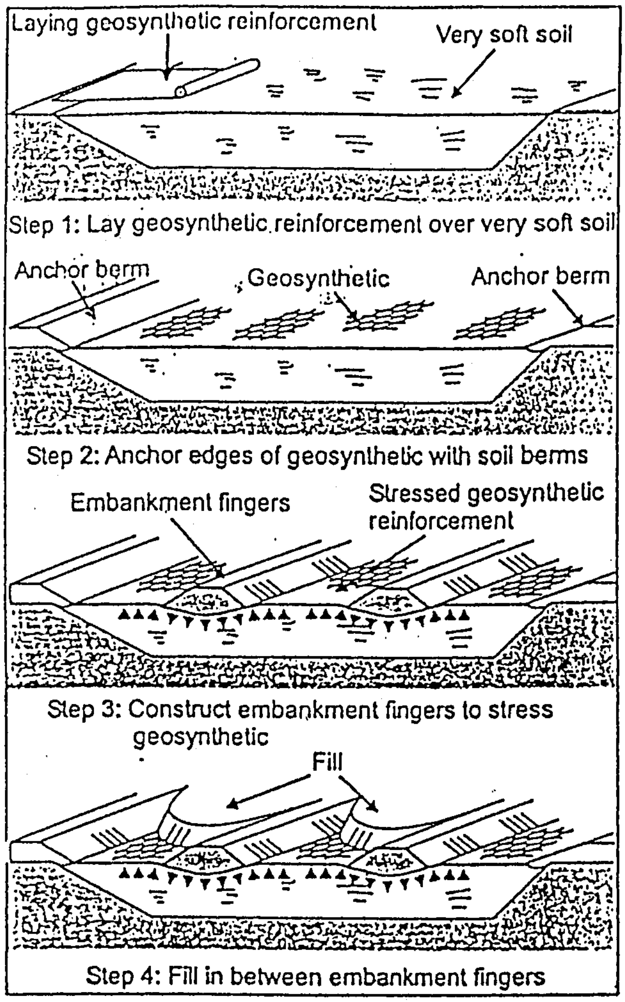


Fig. 5 Reclamation of Super Soft Ground with Geosynthetic Reinforcement

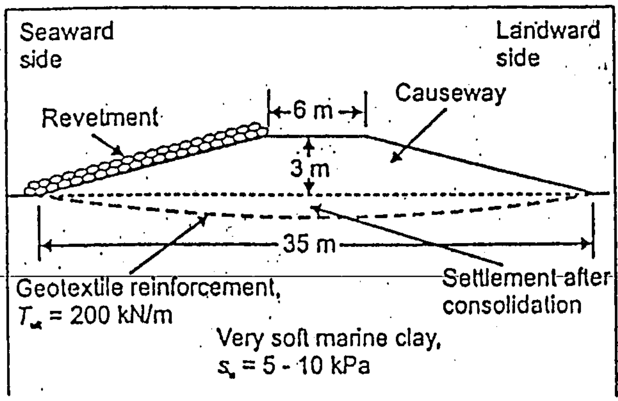


Fig. 6 Basal Reinforced Causeway/ Seawall

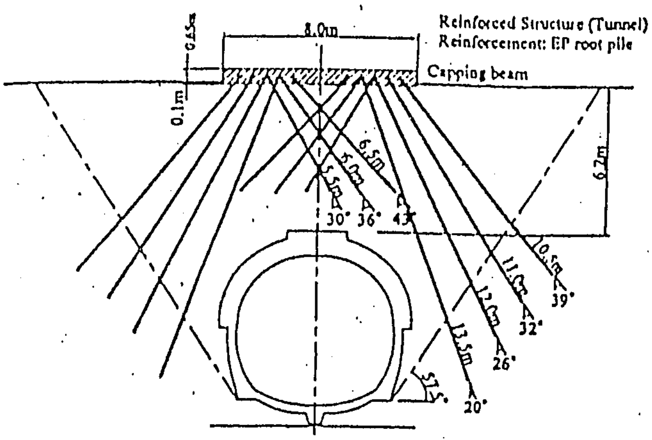


Fig. 7 Reinforced Ground Support for Tunnelling

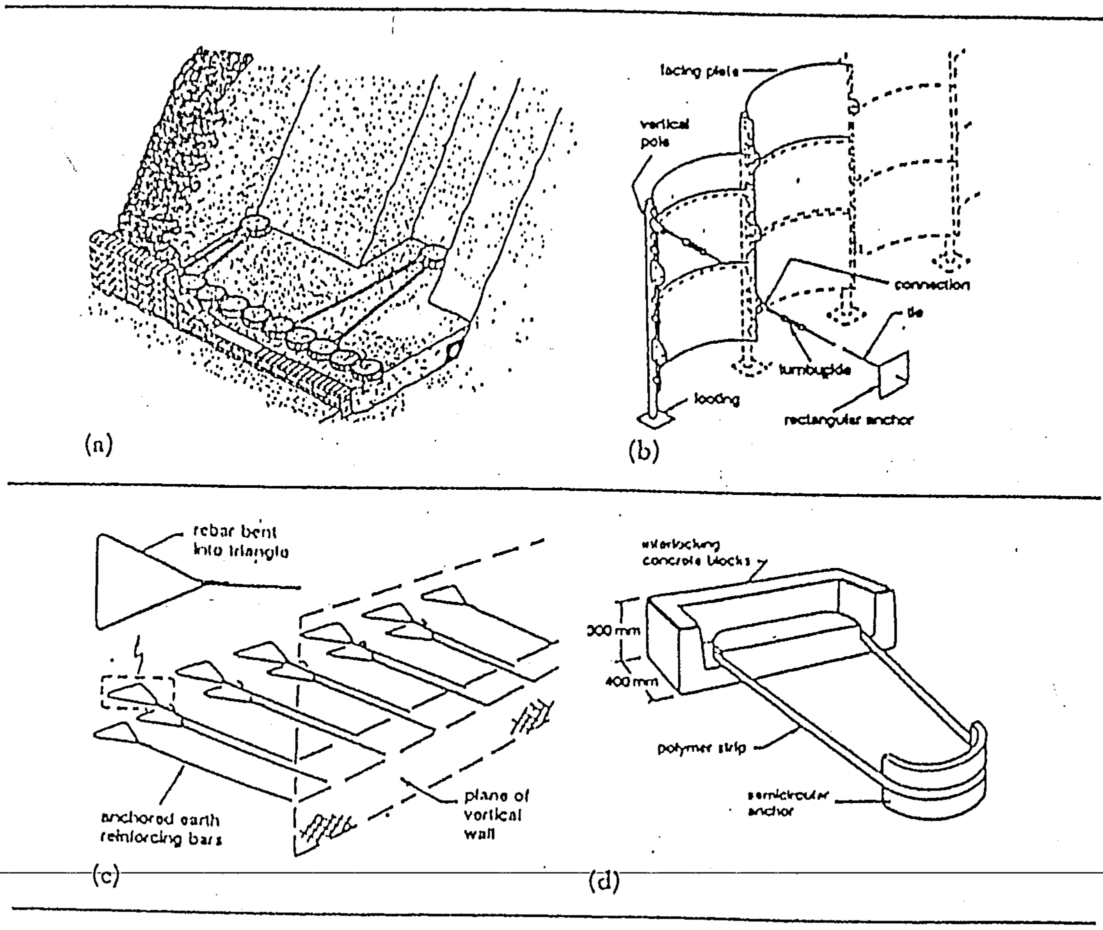


Fig. 8 Anchored Wall Systems; (a) Waste Tyres and Geosynthetic; (b) Anchor Plates; (c) Triangular Rebar and (d) Semicircular Anchor

REINFORCEMENT - GROUND / SOIL INTERACTIONS

COMPRESSION

Granular piles, lime or cement columns strengthen and stiffen soil by sharing larger percentages of load or stresses by virtue of their higher strength and stiffness characteristics. Fig. 9 depicts the stress concentration over a reinforcing element. The loads are transferred to these elements by a stiff granular pad laid over soft soils. Madhav and Van Impe (1993) present a design chart for the estimation of percentage of load transferred to granular columns in terms of relative stiffness of the granular pad and modular ratio of the granular pile and soft ground (Fig. 10). In the figure, K_R ($=k_{gp}/k_s$) is the ratio of relative stiffnesses of the granular pile, k_{gp}, and the soft soil, k_s, and λc ($=k_s d^2/G.H$) where d is the diameter of the granular pile and G and H the shear stiffness and thickness of the granular pad respectively. Stiffer the pad and / or the granular pile and softer the soft ground larger percentages of load are carried by the reinforcement. However, these reinforcing elements transfer their load back to the soil at depth through shaft and bearing resistances.

SHEAR

Reinforcing elements not only contribute to reduce settlement but also increase the resistance of the reinforced soil. Fig. 11 illustrates the stress concentration effect on stresses carried by granular pile providing increased resistance to the ground. Increase in the vertical stress on the inclusion results in increased normal stress on the failure surface and thus increased shear resistance. If the soft soil surrounding the granular pile consolidates, the confining stresses on the granular pile further increase proving for enhanced strength to the reinforced soil.

TENSION

Basal Reinforcement at the Interface of Embankment and the Soft Ground. Unreinforced embankments built on soft ground (Fig. 12 a) tend to fail by lateral spreading. The lateral outward shear stresses (Fig. 12b) reduce the bearing caacity (Fig. 12c) of the ground leading to bearing failure. The layer of reinforcement provided at the base of the embankment (Fig. 12d) not only resists the outward shear stresses (Fig. 12e) but also provides for increased bearing capacity (Fig. 12f) by virtue of mobilizing inward shear stresses. A very similar mechanism operates in reinforced subgrades beneath pavements (Fig. 13)

Reinforcement provided in retaining walls, generates tension due to the lateral and downward movement of the unstable soil mass (Fig. 14) The tension is transferred to the part of reinforcement in the stable soil region. If the length of reinforcement in this region is inadequate to resist the pullout forces anchors may be provided to take care of the additional force. The anchor resists the pullout force by passive or bearing resistance (Fig. 15). The bearing resistance available is strongly dependent on the angle of shearing resistance of the granular material.

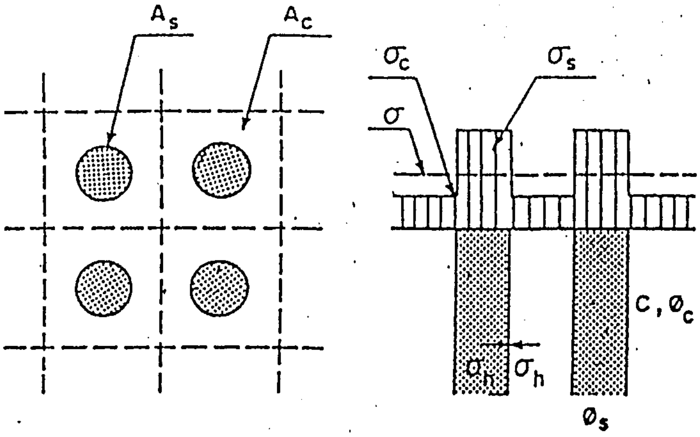


Fig. 9 Stress Concentration on Granular

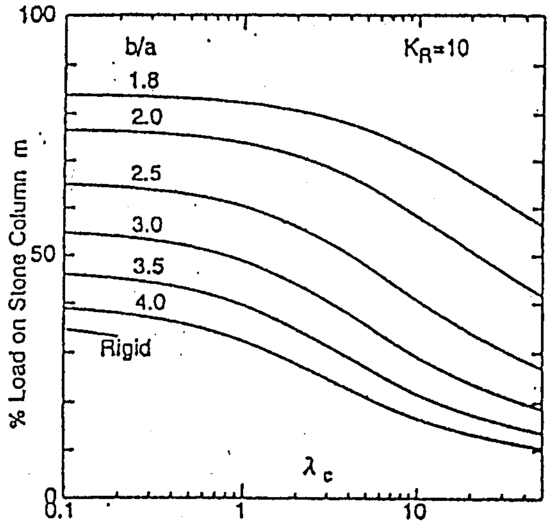


Fig. 10 Effect of Stiffness of Granular Pad on Percentage Load on Granular Pile

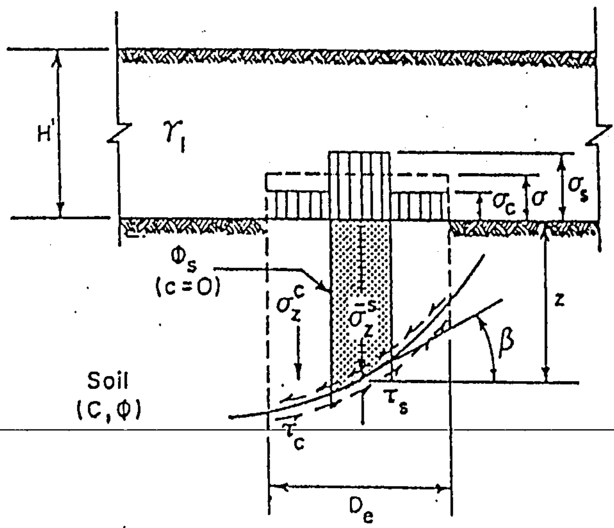


Fig. 11 Effect of Stress Concentration on Shear Resistance on Granular Pile

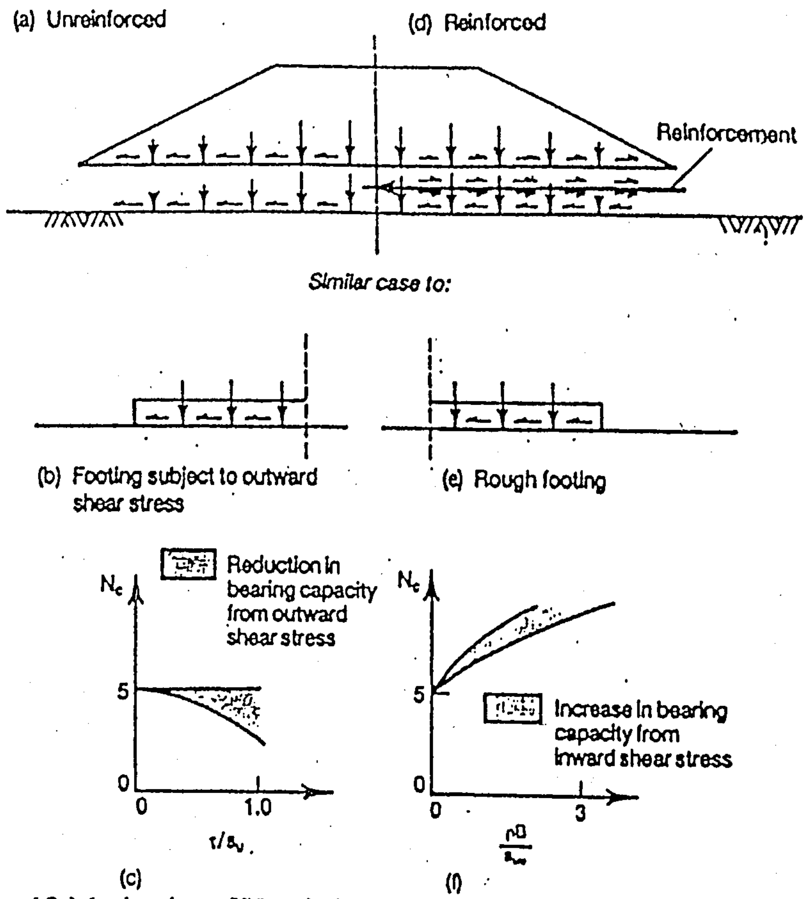


Fig. 12 Mechanics of Unreinforced and Reinforced Embankments

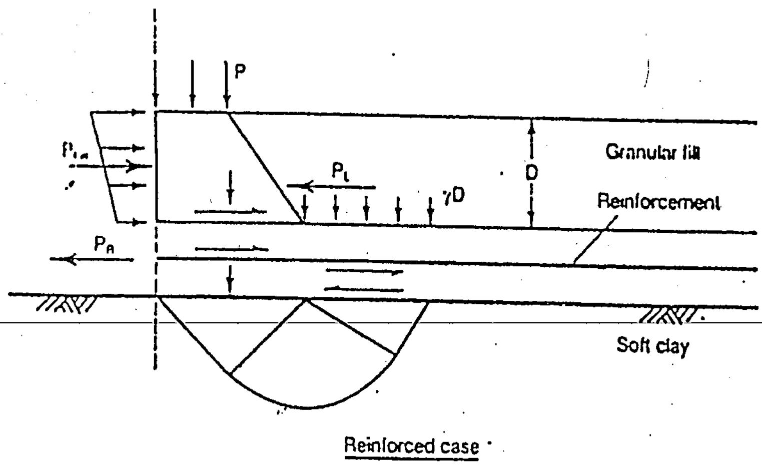


Fig. 13 Mechanics of Reinforcement in Reinforced Foundation Beds

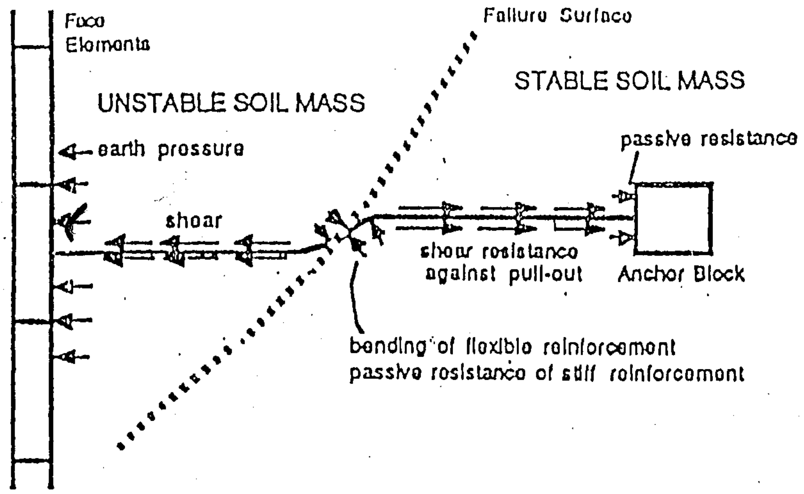


Fig. 14 Mechanics of Reinforcement in Reinforced Earthwalls

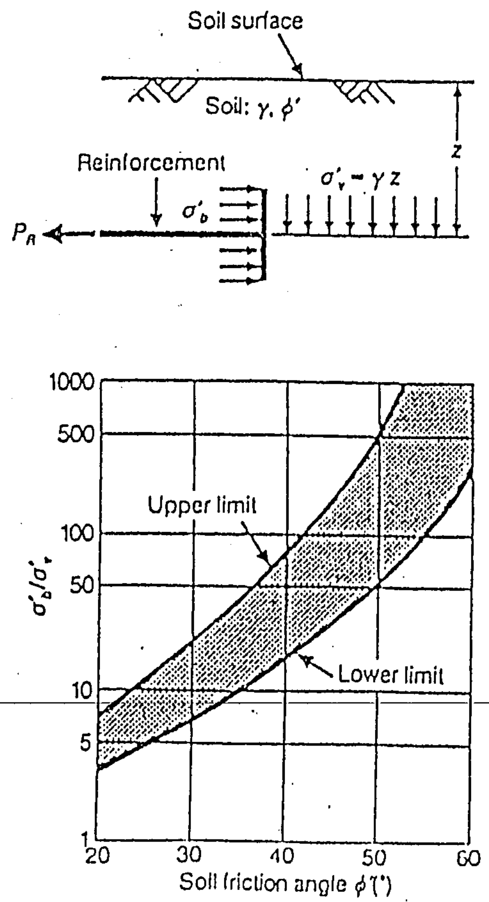


Fig. 15 Mechanics of Bearing and Estimation of Bearing Stress

FLEXURE

Reinforced Foundation Beds; Pitchumani (1995) Madhav and Pitchumani (1996) and Pitchumani and Madhav (2000) analyse the reinforcement strip interactions in terms of both shear and flexure. Stiff reinforcements placed normal to the applied loads, resist transverse deformations through flexure. If the reinforcement is rigid, large interaction normal stresses are mobilized (Fig. 16) beneath the center of the reinforcement and the reinforcement deforms as a rigid body. Relatively flexible reinforcement bend and deform accordingly. Pitchumani and Madhav (2000) quantify the effect of relative flexibility on the reduction of settlement.

Reinforced Piled Embankments; Geosynthetic reinforced embankment founded on piles was first used in Asia in 1982 (Tan et al. 1985). Conventional piled embankments require large pile caps to transfer the embankment load on to the piles. Often batter or raking piles are necessary to carry the horizontal loads. Instead if the reinforcement is provided at the base of the embankment, smaller sized pile caps would suffice to capture the load from the embankment. Moreover, the reinforcement obviates the need for batter piles. Two and three dimensional Mechanisms of load transfer through arching actions are shown in Fig. (17). Smaller amounts of tension are required in the former case as the pile cap is in the form of a strip. Similarly, larger the spacing, larger would be the required tension in the reinforcement.

REINFORCED EARTH

Earth fills as in retaining structures (Fig. 18) can be supported by reinforced earth walls instead of structural support through gravity, cantilever or counterfort walls. Thus we have reinforced earth walls, gabions, crib walls, etc. which consist of prescribed earth fill along with appropriate reinforcing structural components such as metallic or polymeric strips, sheets, meshes or boxes. Retaining structures can be built by anchoring as well. A combination of geosynthetic reinforced walls with rigid facing (Fig. 19) is commonly used in Japanese railways.

Varadarajan et al. (2000) present a comprehensive design procedure for reinforced embankment based on limit equilibrium method. Inaddtion to the usual factors, the variation of undrained strength with depth, large foundation depth and the effect of reinforcement on rotational failure mechanism are considered.

DEFORMATIONS OF REINFORCED STRUCTURES

A large variety of factors (Fig. 20) affect the performance of reinforced structures, viz., climatic conditins, construction practice, reinforcement - soil geometry, external loads, local geomorphology, groundwater and internal environment (soil and reinforcement characteristics & durability). The effect of level of deformation on the performance of the structure is presented in Fig. 21. In most cases, the deformations increase with time. If the rate of increase is slow and the deformations stabilize after some time (Stage 1), the structures are stable and serviceable. If the rate of deformation is relatively high but the deformations stabilize after some time, the structures could be stable but unserviceable (Stage 2). In Stage 3 deformations both the rate and magnitude of deformations are so high as to make the facility unstable and unserviceable. Thus the designs of reinforced structures and ground require checks on deformations as well as for stability.

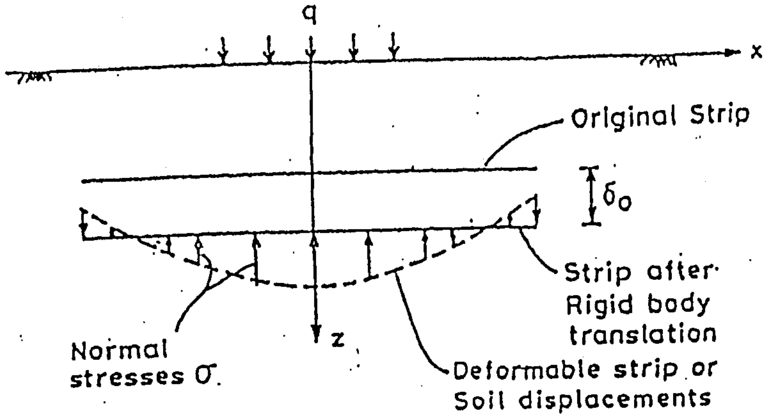


Fig. 16 Flexural Mechanism of Reinforcement

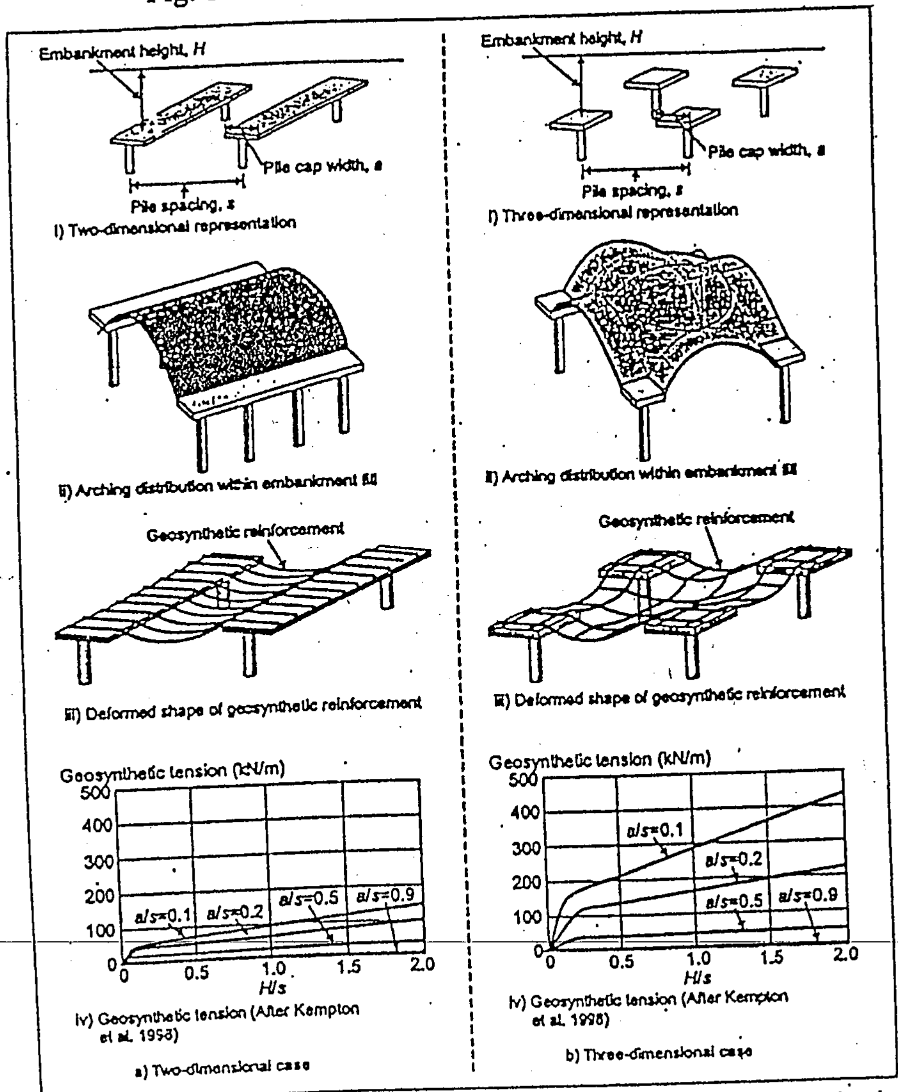
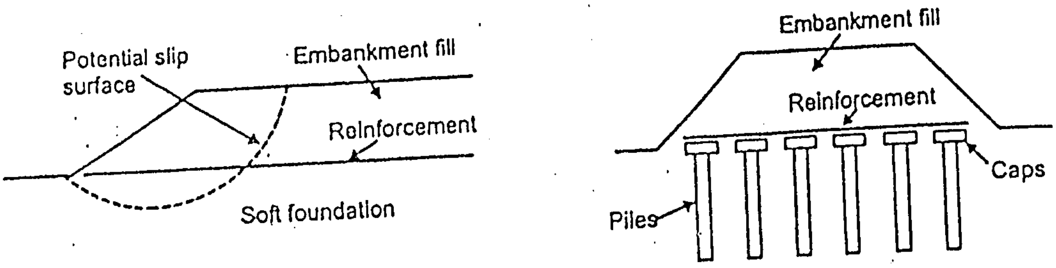
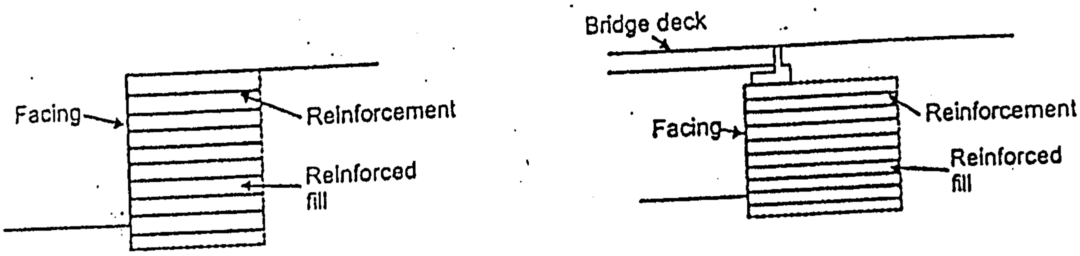


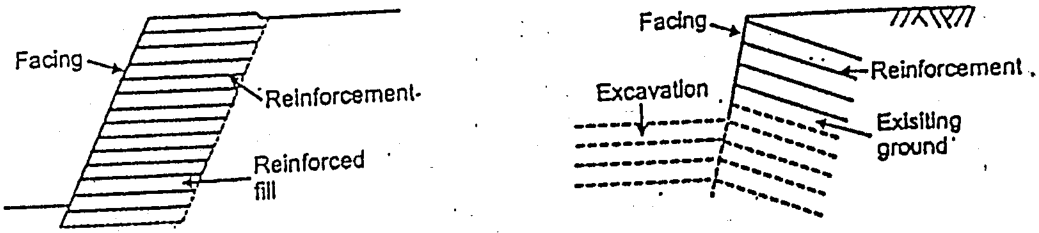
Fig. 17 Two and Three Dimensional Arching Mechanisms in Geosynthetic Piled Embankments



a) Basal reinforced embankments over soft foundations



b) Reinforced walls and abutments



c) Reinforced fill and soil nailed slopes

Fig. 18 Different Reinforced Earth Structures

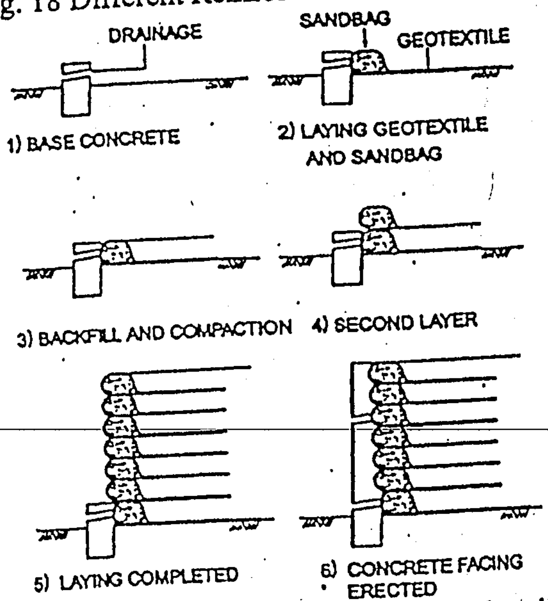


Fig. 19 Rigid Facing Reinforced Earthwalls

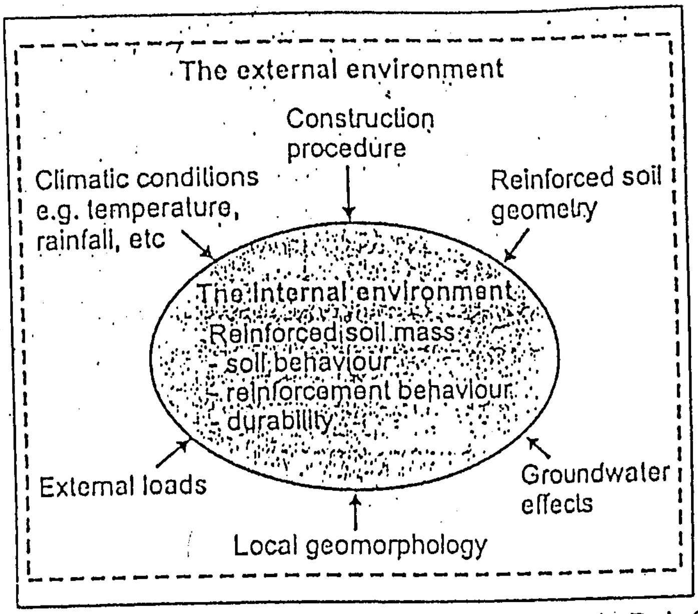


Fig. 20 Factors affecting Deformations of Reinforcement in Reinforced Earth Structures

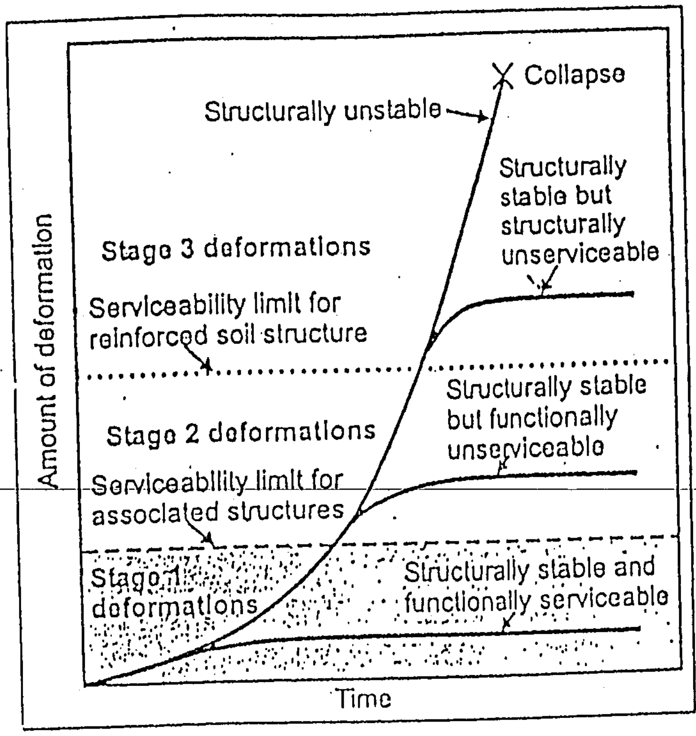


Fig. 21 Limits of Deformations for Performance of Reinforced Earth Structures

CONCLUSIONS

The practice of reinforcement of ground or soil, even though ancient, has received considerable attention in the last twenty five to thirty years. In current geotechnical practice, a very lrge percentage of works utilize some form of reinforcement. Ground is reinforced in compression and shear with granular or lime mixed piles, grouted bolts / micropiles, slopes and earth structures such as retaining walls and abutments in shear and with the reinforcement mobilizing tension. Unstable ground ahead of tunneling is stabilized by reinforcing or stitching the ground.

Reinforcement of ground is achieved by several mechanisms operating singly or together. The mechanisms are compression, shear, flexure, and shear. The former two are utilized in reinforcing soft ground with inclusions while the latter two in reinforcing soils or fills. The above principles are also valid in case of reinforced fill slopes.

REFERENCES

Bergado, D.T., Anderson, L.R., Miura, N. and Balasubramaniam, A.S. 1996. Soft Ground Improvement. ASCE Press, New York, 427p.

Lawson, C.2001. Performance related issues affecting reinforced soil structures in Asia. Int. Symp. on Earth reinforcement, IS Kyushu, Preprint Vol., pp 63-99.

Madhav, M.R. and Pitchumani, N.K. 1996, Numerical modeling and analysis of reinforcement - strip interactions. Proc. 2nd Int. Conf. Computer Applications, Research and Practice, Bahrain, pp 579-586.

Madhav, M.R. and Van Impe, W.F. 1993. Load Transfer through a gravel bed on stone column reinforced soil. Geotechnical Engrg., J. of Southeast Asian Geotechnical Soc., Vol. 25, No. 2. pp. 47-62.

Pitchumani, N.K. 1995 Reinforced foundation beds; Soil reinforcement interaction analyses, doctoral thesis, IIT, Kanpur, 278p.

Pitchumani, N.K. and Madhav, M.R. 2000. Vertical displacement interaction between soil and reinforcement strip. Ind. Geotech. J., Vol 30, No.3, pp 133-155.

Ramu, K. 2001 Modelling approaches for and analysis of reclamation process and response of reclaimed ground, Doctoral thesis, IIT, Kanpur, 344 p.

Varadarajan, A., Sharma, K.G. and Aly, M.A.A. 2000. Design of reinforced embankments by limit equilibrium and finite element methods, IGJ, Vol 30, No.3, pp.100-132.

Improvement of organic soil by Dynamic Replacement

H. S. Thilakasiri, Dept. of Civil Engineering, University of Moratuwa, Moratuwa, Sri Lanka.

Abstract

This paper describes the application of a relatively new treatment method called Dynamic Replacement (DR) to improve soft organic soils by formation of a grid of sand columns using heavy tamping. A brief history of the use of heavy tamping in ground improvement is presented while the lack of analytical tools for selection of the factors attributing to effective implementation of DR process is highlighted. A summary of the development of a finite element algorithm for simulation of DR process is demonstrated and the use of an instrumented pilot DR treatment program results are used to validate the applicability of the developed finite element algorithm. Furthermore, some of the correlations developed from the results of the pilot test program were also described.

Introduction

Improvement of mechanical characteristics of loose to medium cohesionless soil by repeated application of very high intensity impacts to the surface, namely dynamic compaction (DC), had been in practice over a long period of time. The use of this heavy tamping technique for improvement of soft cohesive soils was not scientifically developed until1974 when Louis Menard introduced Dynamic Consolidation (DC). Publication of his research, Menard (1974) stressed four main points associated with implementation of Dynamic Consolidation (DC) for soft cohesive soils: (a) the compressibility of saturated soils due to the presence of micro-bubbles; (b) the gradual liquefaction under repeated impacts; (c) the change of permeability of a soil mass due to the creation of fissures and; (d) subsequent gaining of strength with time (thixotropic recovery). With the successful application of Dynamic Consolidation (DC) to improve soft cohesive soils in various projects around the world various attempts were made to modify the DC technique for better improvement. Kruger et al. (1980) used a modified version of DC called Dynamic Replacement (DR) to improve very-soft-to-soft cohesive soil deposits. In this method, a sand blanket is first placed on the ground surface for confinement against surface heaving during tamping process. Subsequently large diameter tapering granular columns are formed in the soft layer at a certain spacing by several phases of pounding and infilling of tamping print location with granular material. Each phase of tamping continued until the rate of penetration of granular charges become relatively insignificant, thereby subjecting the ground adjacent to Dynamic Replacement (DR) also to a certain degree of compaction. Creation of large number of compacted sand columns in the soft soil deposit not only increased its stiffness but also increased the drainage characteristics as well. Lo et al. (1989) successfully used a modified version of Dynamic Replacement (DR) to improve a highly organic cohesive soil deposit in a construction site in Singapore. In this modified method, called Dynamic Replacement and Mixing (DRM), essentially the treatment process was carried out in two phases. During the phase one a grid of granular columns were installed in the organic layer with use of low energy pass similar to Dynamic Replacement (DR). Subsequently, in the second phase the installed sand columns in the low energy phase were imparted with sufficient high energy pounding to cause jets of sand to be ejected from them into the surrounding peaty soil by a process resembling claquage. At the end of the high energy phase of the treatment process, a sand blanket with peaty clayey pockets underlained by a sandy peaty clay layer was formed. Thus, the above the treatment process is named Dynamic Replacement and Mixing (DRM)

Scope of the present research

The design of a successful Dynamic Replacement and Mixing (DRM) project depends on the selection of: (i) hammer weight and the drop height; (ii) thickness of the sand blanket; (iii) spacing of the sand columns and; (iv) number of blows per print location for both high and low energy passes. Selection of these parameters for a given treatment project is normally carried out by conducting monitored field trials prior to the actual treatment program. This procedure is both time consuming and costly and creates problems if the initially selected hammer size and the drop height has to be changed according to the findings of the field trial. Due to these reasons there are two key areas in which analytical methods are to be strengthened: (i) Selection of the hammer size and the drop height prior to the actual treatment program and; (ii) The assessment of quality control during the actual treatment program.

Available analytical techniques for analyzing Dynamic Replacement (DR)

A well developed analytical model, taking into consideration of the complex behavior of the ground response to dynamic forces, is yet to be introduced for simulation of Dynamic Compaction (DC) or Dynamic Replacement (DR). There are number of analytical models with limited capabilities available for prediction of surface stress and crater depth during Dynamic Compaction. The methods Scott and Pearce (1975), Lysmer and Richart (1969), Brandal Sadgorski (1977) and Mayne and Jones (1983) are capable of predicting surface dynamic stress during dynamic compaction while the numerical methods proposed by Chow et al (1992) and Thilakasiri et al. (1996), taking into account the wave propagation through the soil, are capable of predicting both surface stress and crater depth. Among the simplified methods available to estimate pore pressure distribution during dynamic compaction are the methods proposed by Corapcioglu et al. (1993) and Gunaratne et al. (1996). Moreover, there are number of empirical correlations available for estimation of the zone of influence and the amount of improvement achieved due to dynamic compaction. Leonard et al (1980) proposed that up to a depth of $1/2\sqrt{Wh}$ below the impact surface, where W is the drop weight in metric tonnes and h is the height of drop in meters, the SPT values will be increased by at least 3 to 5. Poran et al. (1992) proposed the use of the Dynamic Stiffness Modulus (DSM), the slope of the tangent of the loading portion of the impact stress Vs relative displacement (Displacement/equivalent diameter of the drop hammer) curve to predict the rate of densification due to dynamic compaction. Due to the limited applicability of the available analytical methods in reliable simulation of Dynamic Replacement (DR), a coupled finite element algorithm was developed to simulate DR phase of the DRM process.

Proposed Finite element algorithm to simulate Dynamic Replacement (DR).

It is common knowledge that any analytical solution to simulate DR must account for such complexities as pore water flow, change of soil properties and above all, the excessive deformation induced by a high energy impact. Contrary to a geometrically linear problem where the obvious choice is a fixed coordinate system, the selection of the reference coordinate system plays a significant role for a geometrically nonlinear problem. Due to the geometrical nonlinearity of the displacement field associated with Dynamic Replacement (DR), Updated Lagarngian cylindrical coordinate system is used in the finite element formulation. As explained in Thilakasiri et al.(2001) an axisymmetrical equations of motion are developed separately for the soil skeleton and the pore water fluid considering the volumetric compressibility of the two phases. A finite element mesh consisting of four node isoparametric elements is used to specially discretize the above equations while the solution is steered in the time domain by the Newmark's explicit time integration. It is observed from both small scale testing in the lab and large scale modeling that the hammer punches through the surrounding soil during the tamping operation. A separate slide line algorithm is developed considering the forces transferred from solid to solid, solid to fluid and fluid to fluid across the discontinuity created by the punching of the soil column. As for the constitutive relations, Cylindrical cap model introduced by DiMaggio and Sandler (1972) is used for sand and the modified Cam Clay model is used for organic soils. A series of laboratory testing programs on the sand and the organic soils were conducted to estimate the relevant material parameters for the constitutive models and the slide line algorithm. The computer program DYCOM, capable of predicting the deformation field and the pore pressure distribution around a tamping location under multiple blows, was developed following the theoretical aspects explain above. In order to investigate the applicability of the developed algorithm for dynamic replacement and to develop new correlations applicable for DR, a field test program was devised.

Pilot testing program

The objective of the field testing program, a full scale DRM treatment program of a peaty soil layer, was to better understand the mechanism of DRM and to obtain actual field test results for later analysis. The site selected for the pilot test program was located on the Interstate I-4 on the alignment of the proposed west-bound on-ramp at exit 13 in Plant City Florida. The area selected for pilot test program was cleared of all vegetations and an approximately 1m thick sand blanket was placed over it. Cone Penetration Test (CPT) at the pilot test location, conducted prior to the testing program, indicated a 1m thick sand blanket overlying a 2m thick organic layer followed by a dense silty sand layer up to the weathered rock layer. The field testing program consisted of tamping a pilot test location, instrumented with pore pressure transducers to measure pore pressure development and inclinometers to measure lateral movement as shown in Figure 1, with an hammer instrumented to measure the acceleration. Inclinometers I/S-1, I/S-2, I-3 and I-4 were installed 1.2m, 1.8m, 2.4m and 3.6m respectively from the center of the pilot test location. The pore pressure transducers P1 and P2 were installed in a borehole, 1.2m from the center of the pilot test location at depths of 1.8m and 3.6m respectively while two other pore pressure transducers P3 and P4 were installed in a borehole 2.4m from the center of the pilot test location at depths of 2.4m and 3.6m respectively as shown in Figure 1.

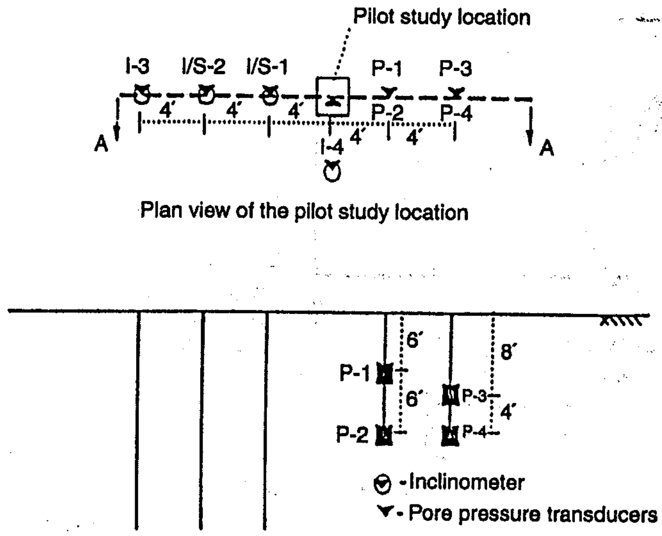


Figure 1. Instrumentation at the pilot test location

The hammer used was fabricated from twenty five 49mm thick 600mm x 600mm square steel plates welded to a 50mm x 150mm steel bar along the centerline of the weight.

The pilot test program consisted of 5 number of low energy blows with a hammer drop height 12m and 15 number of high energy blows with a hammer drop height of 18m. During each blow the hammer acceleration and pore pressure from four pressure transducers were electronically obtained and after each blow the crater depth, inclinometer readings and CPT readings at the impact location were taken. The crater created by each blow was filled with sand before the next blow was applied. From the cone penetration test conducted at the pilot test location showed that the sand column has penetrated the entire soft organic soil layer and the diameter of the sand column at the top was about six feet. Plate load tests carried out using a 750mm diameter plate at the center of the column gave settlement of 25mm, 50mm and 75mm for bearing pressures of 335Kpa, 478Kpa and 621Kpa respectively.

Analysis of the results of the pilot test program

The computer program developed, DYCOM, was used to predict the hammer acceleration and, pore pressure development of transducer P1, shown in Figure 1, after the application of the first blow. The deformed axisymetric finite element mesh after the first blow is shown in Figure 1. Comparisons of the predicted and measured hammer acceleration and pore pressure generations of transducer P1 are shown in Figures 3 & 4.

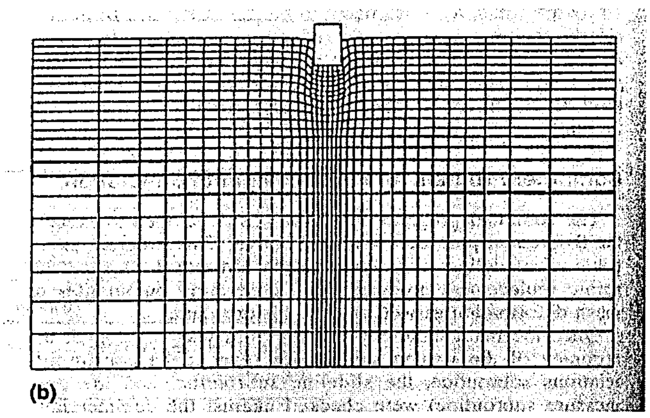


Figure 2. Deformed finite element mesh after the first blow

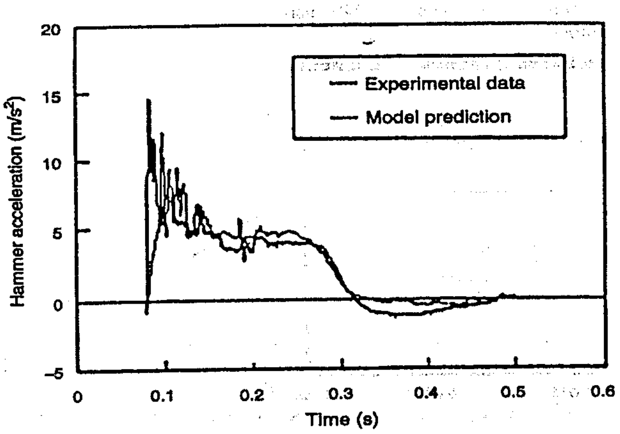


Figure 3. Predicted and experimental hammer acceleration during the first blow.

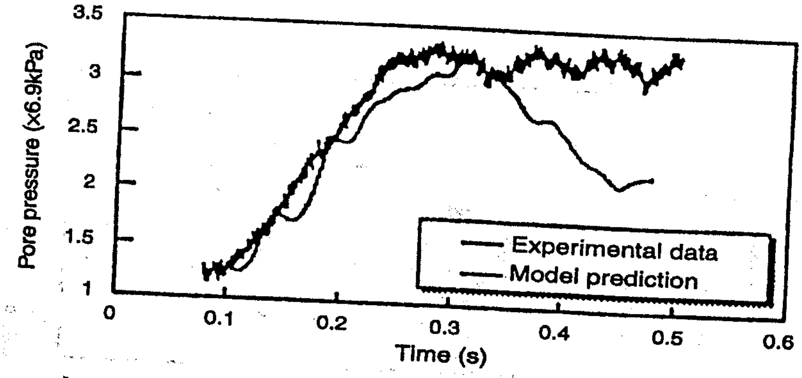


Figure 4. Predicted and experimental pore pressure generation at P1 during the first blow

Comparisons of the observed measurements with the program predicted locations of the inclinometer I/S-1 after first, second and forth blows are shown in Figures 5 to 7.

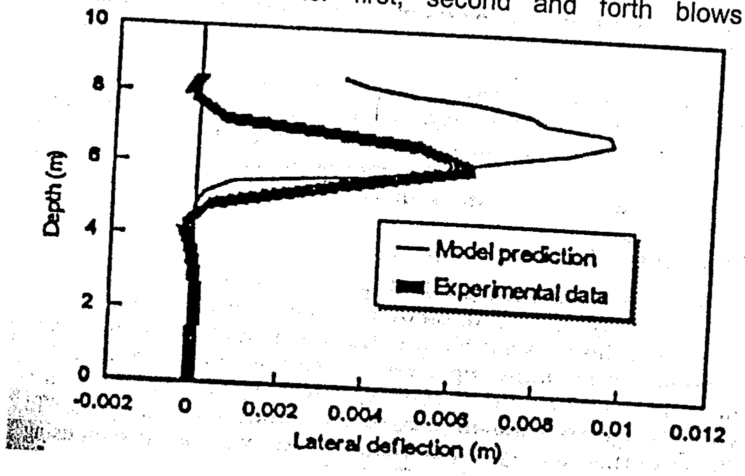


Figure 5. Lateral displacement of inclinometer I/S-1 after first blow

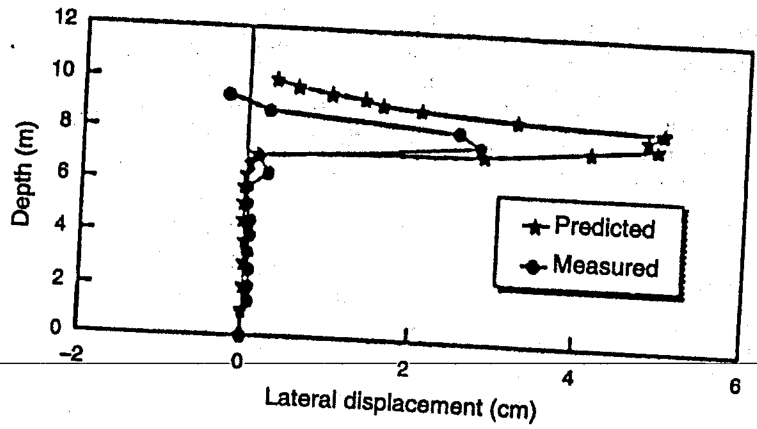


Figure 6. Lateral displacement of inclinometer I/S-1 after second blow

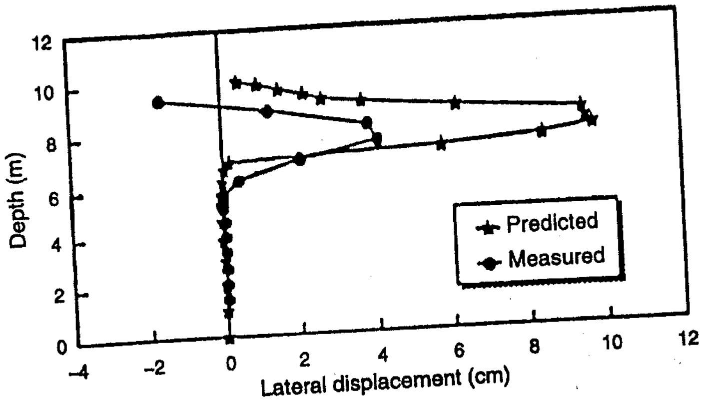


Figure 7. Lateral displacement of inclinometer I/S-1 after forth blow

Figures 7 & 8 show the lateral displacement of the inclinometer I/S-2 after second and forth blows.

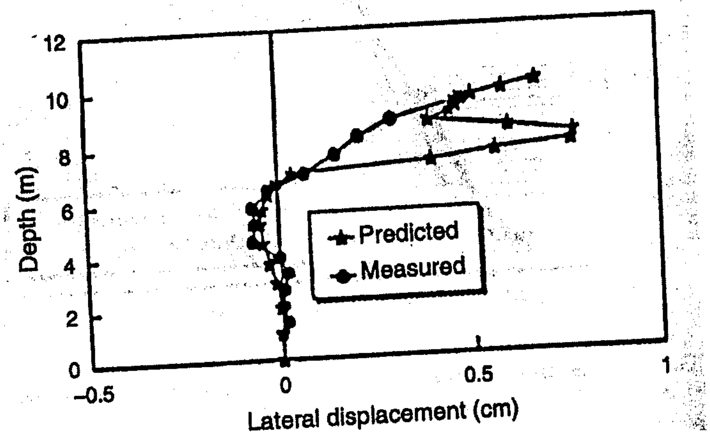


Figure 7. Lateral displacement of inclinometer I/S-2 after second blow

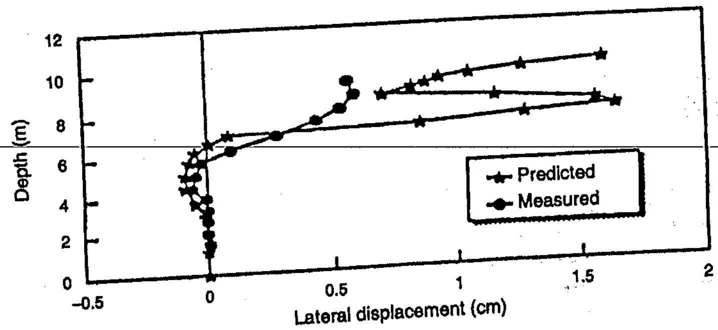


Figure 8. Lateral displacement of inclinometer I/S-2 after forth blow

Moreover, the program predicted location of the top of the organic soil layer after first four blows are shown in Figure 9 with the actual measured depth from the CPT.

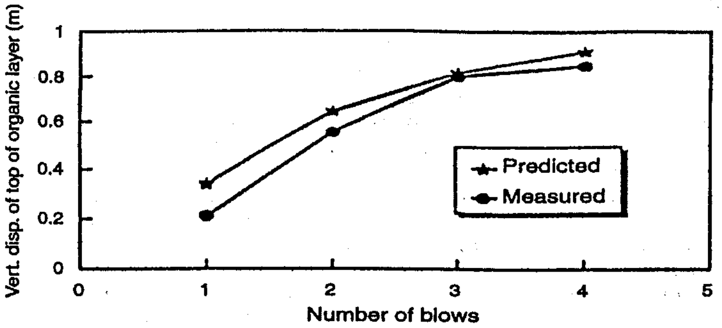


Figure 9. Predicted and measured depth to the top of the organic soil layer after first four blows

A typical impact stress Vs relative displacement (Displacement/equivalent diameter of the drop hammer) curve is shown in Figure 10. The dynamic stiffness modulus (DSM)), the tangent of the initial loading portion of the impact stress Vs relative displacement curve, were obtained for all the acceleration records of the hammer.

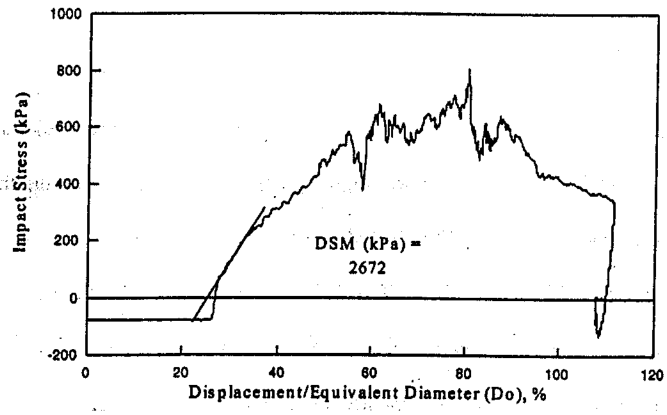


Figure 10. Typical impact stress Vs relative displacement curve

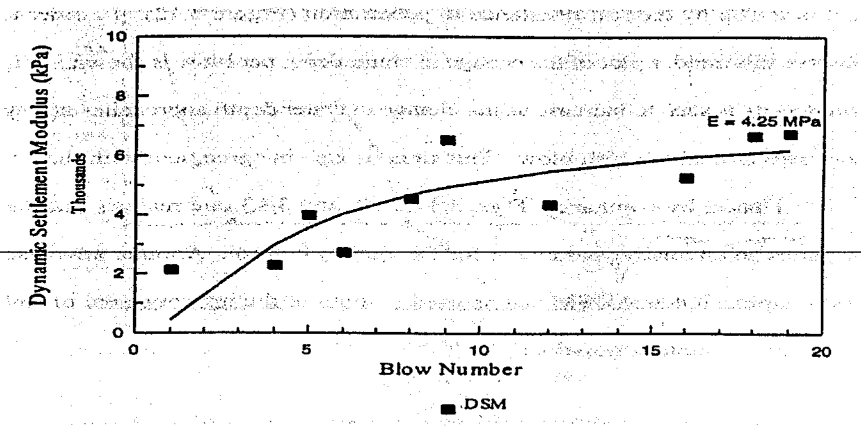


Figure 11. DSM vs blow number for the pilot study location

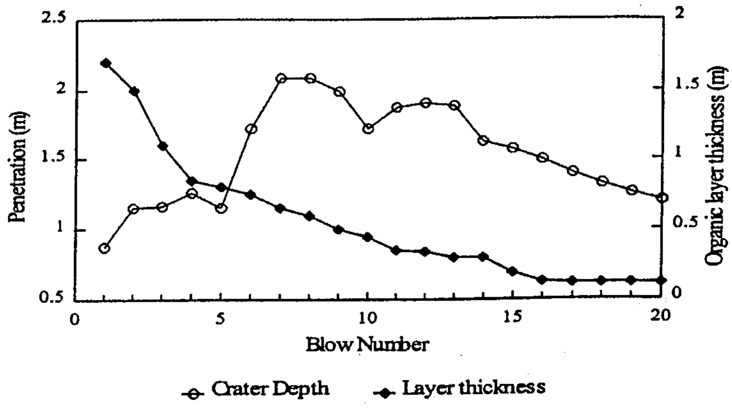


Figure 12. Measured crater depth and depth of organic layer vs blow number

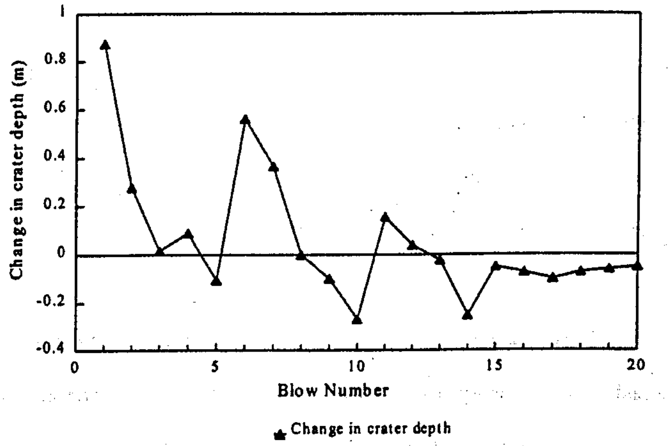


Figure 13. Change in crater depth vs blow number for the pilot study location

The variation of the DSM with the blow number is shown in Figure11. As it is evident from Figure 11, the DSM doesn't increase after the fifteenth blow. Figure 12 shows the crater depth and thickness of the organic layer Vs blow number while Figure 13 shows the rate of change of crater depth Vs number of blows. It is clear from Figures 12 that when the DSM reaches saturation the column penetration is maximum and the subsequent energy applied is completely used for lateral bulging of the sand column. Stinnette (1996) based on Figure 13 concluded that the constant rate of change of the crater depth is an indication of the saturation of the DSM.

Cost effectiveness

The cost of improvement of the soft organic layer using DR is compared with the other alternative treatment methods to investigate the cost effectiveness of the DR process. The cost for improvement of a given area using three different treatment methods, namely, demucking, surcharging and DR are considered for comparison. Gunaratne et al. (1997) gave the cost comparison shown in Figure 14.

Moreover, the program predicted location of the top of the organic soil layer after first four blows are shown in Figure 9 with the actual measured depth from the CPT.

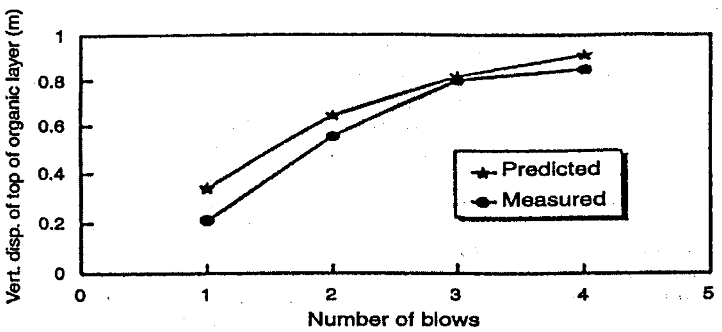


Figure 9. Predicted and measured depth to the top of the organic soil layer after first four blows

A typical impact stress Vs relative displacement (Displacement/equivalent diameter of the drop hammer) curve is shown in Figure 10. The dynamic stiffness modulus (DSM)), the tangent of the initial loading portion of the impact stress Vs relative displacement curve, were obtained for all the acceleration records of the hammer.

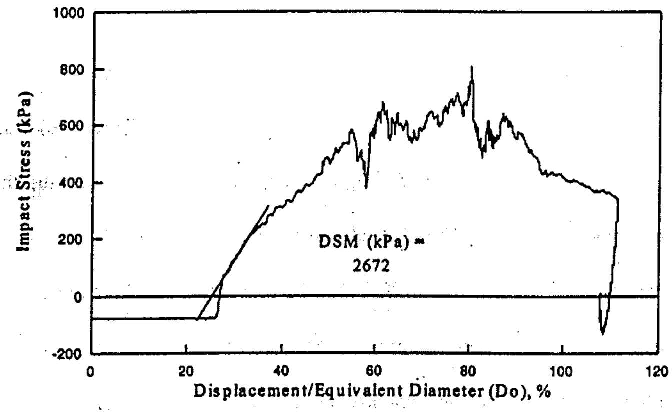


Figure 10. Typical impact stress Vs relative displacement curve

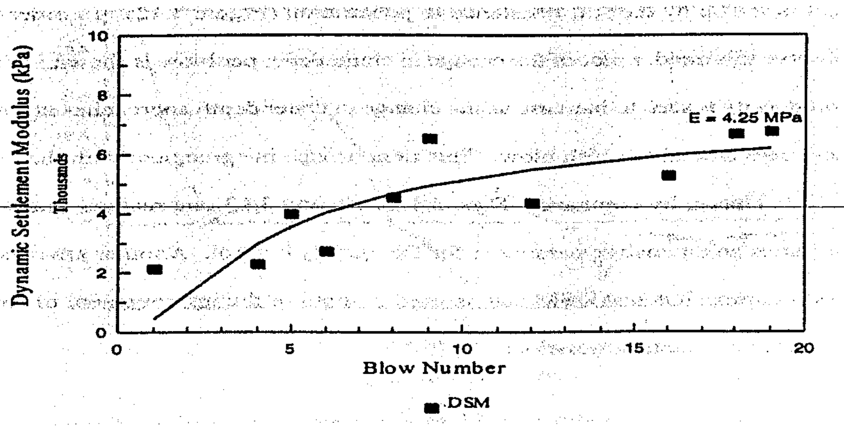


Figure 11. DSM vs blow number for the pilot study location

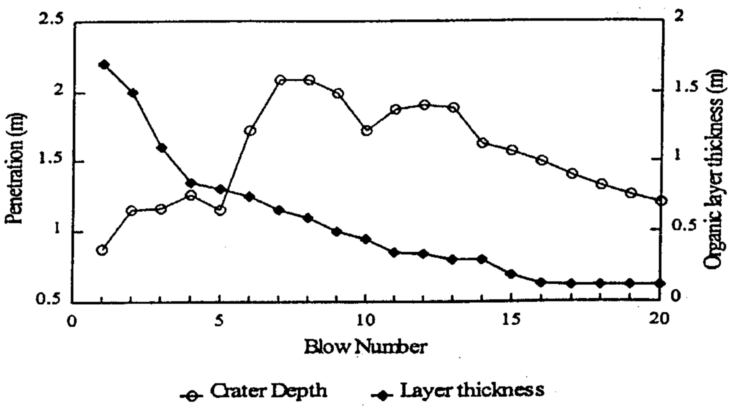


Figure 12. Measured crater depth and depth of organic layer vs blow number

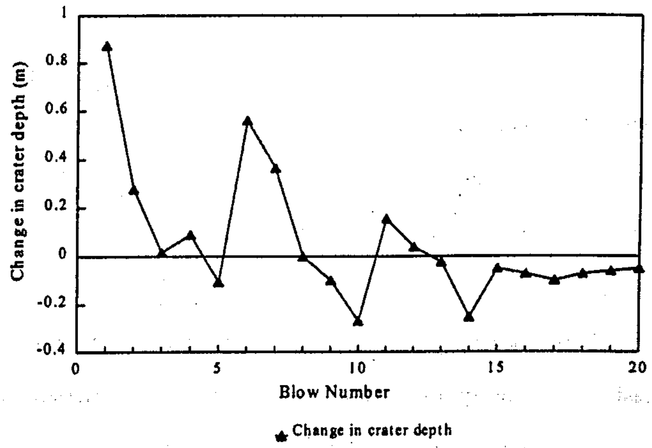


Figure 13. Change in crater depth vs blow number for the pilot study location

The variation of the DSM with the blow number is shown in Figure11. As it is evident from Figure 11, the DSM doesn't increase after the fifteenth blow. Figure 12 shows the crater depth and thickness of the organic layer Vs blow number while Figure 13 shows the rate of change of crater depth Vs number of blows. It is clear from Figures 12 that when the DSM reaches saturation the column penetration is maximum and the subsequent energy applied is completely used for lateral bulging of the sand column. Stinnette (1996) based on Figure 13 concluded that the constant rate of change of the crater depth is an indication of the saturation of the DSM.

Cost effectiveness

The cost of improvement of the soft organic layer using DR is compared with the other alternative treatment methods to investigate the cost effectiveness of the DR process. The cost for improvement of a given area using three different treatment methods, namely, demucking, surcharging and DR are considered for comparison. Gunaratne et al. (1997) gave the cost comparison shown in Figure 14.

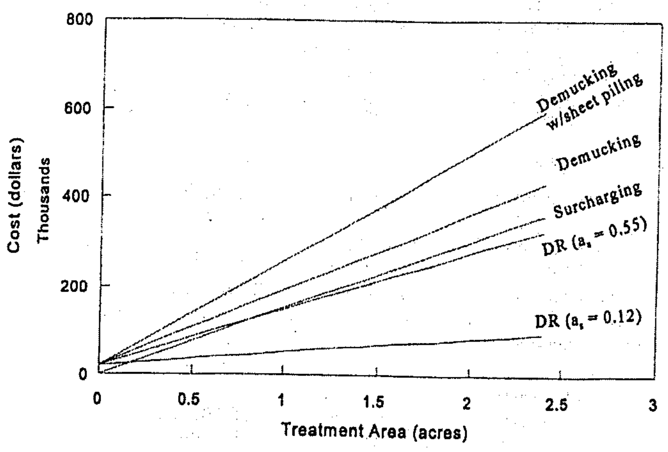


Figure 14. Cost comparison between different ground improvement methods

Conclusions

The pilot test conducted at Plant City, Florida confirmed the feasibility of Dynamic Replacement (DR) as a viable alternative for improvement of soft organic soils. The instrumented pilot study program showed the penetration and lateral bulging of the sand column due to Dynamic Replacement (DR) and at the end of the tamping (after 5 low energy blows and 15 high energy blows) a 1.5m diameter sand column penetrating the entire soft layer was formed. Furthermore, a finite element algorithm, considering kinematics of both the solid phase and the pore fluid, was developed for modeling Dynamic Replacement (DR) process. Subsequently, the finite element algorithm developed, together with the material parameters extracted from laboratory tests, was utilized to compare the model predicted hammer acceleration, lateral deformation, and pore pressure generation in the neighborhood of the impact location with the corresponding quantities measured during the full scale pilot test program. Reasonably good agreement was obtained between the measured and the predicted results confirming the applicability of the proposed algorithm for modeling Dynamic Replacement (DR). A reasonably good correlation between the Dynamic Stiffness Modulus (DSM) and the rate of change of the hammer penetration was observed during the pilot test program. Finally, cost effectiveness of the Dynamic Replacement (DR) was demonstrated by comparing the unit cost of DR with the other alternative ground improvement techniques.

Acknowledgement

The above research program was conducted with the funding provided by the Florida Department of Transportation (FDOT) and under the supervision of Prof. M. Gunaratne, Associate Professor, Department of Civil and Environmental Engineering, University of South Florida, Tampa, Florida.

References

1. Chow, Y.K., Young, D.M., Young, K.Y., and Lee, S.L.,"Dynamic compaction analysis", *J. Soil Mech. Found. Div. ASCE.*, 118, No08, 1989
2. Corapcioglu, M.Y, Mathur,S., and Bear, J., "Dynamic compaction of saturated porous columns" *J. Soil Mech. Found. Div. ASCE.*, 119, No08, 1993
3. DiMaggio, F.L, and Sandler, I.S, "Material models for granular soils", *J. Eng. Mech. Div. ASCE*, 97, 03,935-950.

4. Gunaratne, M., Maringanthi, R., Thilakasiri, H.S., Mullings, G., Stinnette, P., and Kuo, C., "Study of pore pressure induced in laboratory dynamic consolidation", *J. of Computers and Geomechanics*, Vol. 18, No. 2, 127-143, 1996.
5. Gunaratne, M., Mullings, G., Stinnette, P., and Thilakasiri, H.S., "Stabilization of Florida organic material by dynamic replacement", *Final Report prepared for Florida Department of Transportation*, April 1997.
6. Kruger, J.J., Guyot, C., and Morizot, J.C., "The dynamic substitution method", *Proceedings of the International Conference on Compaction*, Paris, pp. 339-343, (1980).
7. Leonards, G.A., Cutter, W.A., and Holtz, R.D.,"Dynamic compaction of granular soils", *J. of Geotech. Engrg Div. ASCE.*, 106,01,35-44, 1980
8. Lo, K. W., Oui, P.L., Yong, and S. L. Lee, "Dynamic Replacement and Mixing of organic soils with sand charges" *J. Soil Mech. Found. Div. ASCE.*, 116, No10, 1463-1482, 1989.
9. Lysmer, J. and Richart, F.E., "Dynamic response of footings to vertical loading", *J. Soil Mech. Found. Div. ASCE.*, 92, No01, 65-91
10. Mayne, P.W., and Jones, J.S., "Impact stresses during dynamic compaction", J. of Geotechnical Engineering, 109 (10), 1342-1346, 1983.
11. Menard, L., and Broise, Y. "Theoretical and Practical Aspects of Dynamic Consolidation", *Geotechnique*, London, England, Vol. 25,No 1.,1975, pp.3-18
12. Poran, C.J, Heh, K.S., and Rodriquez, J.A, "Impact behavior of sand", Soils and Foundations, 32,04, 81-92,1992.
13. Stinnette, P " Database Management system for organic soils", Ph.D. Dissertation, University of South Florida, Tampa, Florida,USA.,1996
14. Thilakasiri, H.S, Gunaratne, M., Mullins, G., Stinnette, P. and Jory, B., 'Analytical and Experimental Investigation of Dynamic Compaction Induced Stresses.' *International Journal for Numerical and Analytical Methods in Geomechanics*, 20, 753-767 (1996*a*)
15. Thilakasiri, H.S., "Analytical Simulation of Dynamic Replacement of Florida Organic Soil", Ph.D. Dissertation, University of South Florida, Tampa, Florida,USA.,1996
16. Thilakasiri, H.S, Gunaratne, M., Mullins, G., Stinnette, P. and Kuo, C.,"Implementation aid for dynamic replacement of organic soils with sand", *J. Geotech. And Geoenvironmental Engrg, ASCE*, 127,01,25-36,2001.