Deformation properties of lime cement stabilised soil in the working state

Propriétés de déformation en phase active pour les sols stabilisés à l'aide de chaux et de ciment

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ABSTRACT: In situ methods for determination of the deformation modulus and the coefficient of permeability of lime cement columns are presented together with a numerical study of clay-column interaction. The study focuses on the parameters that affect time dependent consolidation of lime cement stabilised clay and the factors that are of importance for the interaction between a column and the surrounding soft clay in the working state.

RESUME: Des méthodes in-situ pour la détermination du module de déformation ainsi que du coéfficient de perméabilité sont présentées, ainsi qu'une étude numérique du comportement réciproque colonne-argile. L' étude est concentrée sur les paramètres qui influencent la consolidation des argiles stabilisées à l'aide de chaux et de ciment en fonction du temps, et les facteurs importants pour le comportement réciproque entre une colonne et l'argile molle environnante durant la phase active.

1 INTRODUCTION

Stabilisation of clay with lime cement columns has lately become one of the most frequently used methods for soil improvement of soft clays used for the foundation of highway and railway embankments. The method is cost effective, as it results in an increased factor of safety against failure as well as a drastic reduction of settlements. The number of installed lime cement columns in Sweden has increased dramatically during the last ten years.

The theories used for design were originally developed for lime columns (Broms, 1984). Today the major part of all columns are manufactured by using a mixture of lime and cement, which results in somewhat different stress-strain and strength properties. Hence, the need for modified design theories and modern quality control methodologies have been actualised. As a part of this work ,a new "Guidance for project planning, execution and inspection" has been developed by Carlsten and Ekström (1995).

In order to intensify research and development regarding lime cement columns, the foundation Swedish Deep Stabilisation was started.

During the last few years, a number of projects have been carried out at Chalmers University of Technology, mainly regarding the material properties of lime cement stabilised clay. At present the research is focused on time dependent consolidation of soft clay stabilised with lime cement columns.

2 STATEMENT OF PROBLEM

The uncertainty in the calculation of settlements and how they develop with time has been fairly large. In combination with a simplified method of analyses, this has in many cases led to a conservative design. A study of a number of case records has also shown this to be the case. The project described in this paper aims at a better prediction of the deformation of lime cement stabilised soil under working loads.

Methods for determining the properties of soft clay are well established while corresponding methods for lime cement stabilised soil still are under development. Below in situ methods for determination of the deformation modulus and the permeability are presented. Results are given and compared with values obtained from laboratory tests. Finally results from a numerical study of soil-column interaction under working load are reported.

3 MODULUS OF DEFORMATION

Determination of the modulus of a lime cement column in the field would require extensive full-scale embankment load tests. As a compromise, a testing method was used where a single fairly short column was axially loaded to failure. The principle of the set up is given in Figure 1, where also the geometrical values are given.

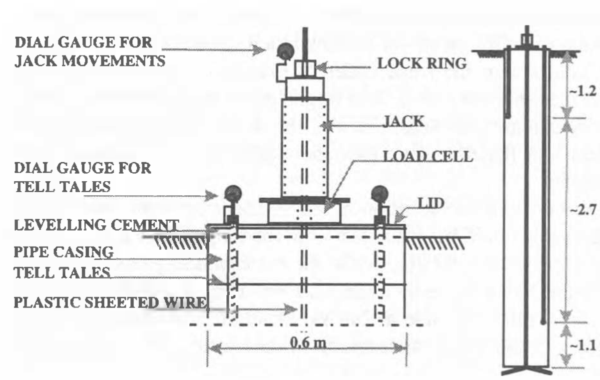

Figure 1 Set up of the equipment used for measuring the deformation modulus of lime cement columns.

The bottom plate and the central wire were installed simultaneously with the mixing of the soils (Holmqvist, 1995). Before the load test, two holes were drilled in the column and tell tales were installed. The load test was then carried out by applying a tension force to the wire, thus compressing the column. During the test, deformation was monitored at the top and at the bottom of the column and by the two internal tell tales. Typical data recorded are given in Figure 2. With four measuring points, three parts of the column can be analysed. Difficulties were however experienced with the bottom part, where the anchor probably penetrated to a certain degree into the column. The upper part of the column had a somewhat different quality, as the clay there originally consisted of dry crust. Therefore, the middle part is used here to illustrate the results.

When evaluating the results, the skin friction between the column and the clay must be taken into account. The maximum value of the skin friction was taken as the undrained shear strength (a pullout test of the hole column showed that the undrained shear strength of the clay was a good measure of the skin friction at failure). A deformation modulus was evaluated assuming fully mobilised skin friction and found to be in the range of 110 and 170 MPa with a mean value of 145 MPa. The boundary conditions along the periphery of the column for this load test lie between that of constant stress (modulus of elasticity, E) and that of no strain (oedometer modulus, M).

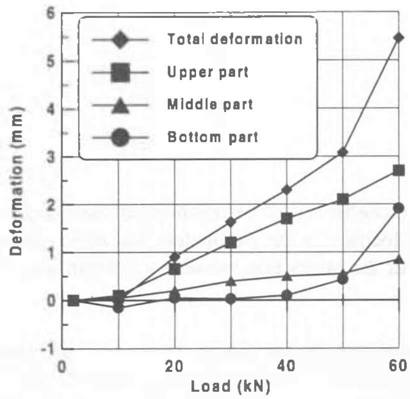

Figure 2 Typical result from an in situ load test of a lime cement column.

Samples were taken from the columns and tested in the laboratory. Ordinary unconfined compression tests were run on nine samples from three different columns. Moduli from these tests ranged from 45 to 110 MPa with a mean value of about 70 MPa.

When comparing the results above, it is important to consider that the boundary conditions are different in the field compared to the laboratory tests, which means that the laboratory values should be expected to be somewhat smaller than the field values although is difficult to say by how much. However, the moduli in the field are in the same order of magnitude as those obtained from the laboratory tests. It is important to point out that the clay here was extremely sensitive, that the in situ mixing was very efficient and that therefore extremely homogenous columns were obtained.

It should also be mentioned that a shear strength value, or a creep value could be evaluated from the field tests. These were in the order of 140-280 kPa, while the corresponding laboratory values were about twice as large, 220-420 kPa. Again the field values are on the safe side as full skin friction was assumed, resulting in a conservative assumption of the vertical stress in the column.

4 DETERMINATION OF PERMEABILITY

Both the modulus of compressibility and the permeability of the lime and lime cement columns have a large impact on the rate of consolidation of the composite soil.

The addition of lime usually increases the permeability of soft clay substantially. Åhnberg et al. (1995) indicated that the resulting permeability for different stabiliser contents depends on the clay type. Experiments have shown that higher permeability has been obtained in the case of clayey silt soil compared with clayey gyttja or pure clay. Terashi and Tanaka (1983) observed a reduction of the permeability of marine clays with increasing lime content.

The permeability will decrease when using lime cement mixture compared with the case of lime mixture only. However the stiffness of the treated soil will increase, which in turn leads to some kind of balance in the rate of consolidation.

In order to understand the settlement of the stabilised ground, the permeability of treated soil must be clarified.

In this study the permeability of treated soil was measured using both field and laboratory tests. It is expected that the properties of the treated soil will vary both with respect to depth and in cross section. A laboratory test usually determines the permeability of a small sample which may or may not be representative of the treated soil. For this reason rather large cylindrical samples with a diameter of 150 mm and a height of 300 mm as well as smaller cylindrical samples 50 × 100 mm have been tested. Most of the samples used in the laboratory were prepared by trimming field manufactured columns with a diameter of 600 mm. Also laboratory mixed samples were used.

The typical clay from Fjärås in Sweden was tested and found to have the following properties: slightly over consolidated clay with a natural water content between 60% and 80% and a sensitivity ranging from 10 to 15.

Table 1. Different sample types

Samples	Stabiliser* content (g)	Lime (%)	Cement (%)	Preparation method	Sample dimensions dxh(mm)
A	68	50	50	laboratory	50 × 100
B	78	50	50	laboratory	50 × 100
C	68	50	50	field	50 × 100
D	68	50	50	field	50 × 100
E	68	50	50	field	150 × 300
F	68	50	50	field	150 × 300
G	68	50	50	field	150 × 300
H	68	50	50	field	50 × 30

* grams per 1 kg clay material

4.1 *Laboratory test results*

The water flow in the treated soil is a function of different factors. The microscopic cracks in the treated soil system are probably the most important factor influencing the value of the permeability. In other words, it is expected that the coefficient of permeability depends on the stress level and perhaps also on the hydraulic gradient in the treated soil. For this reason tests were carried out using different cell pressures and different gradients.

Samples A and B were prepared in the laboratory and left to cure during 14 to 45 days. The samples were then placed in a triaxial cell and different cell pressures were applied. The permeability tests were performed using constant head test. The results are given in Figure 3.

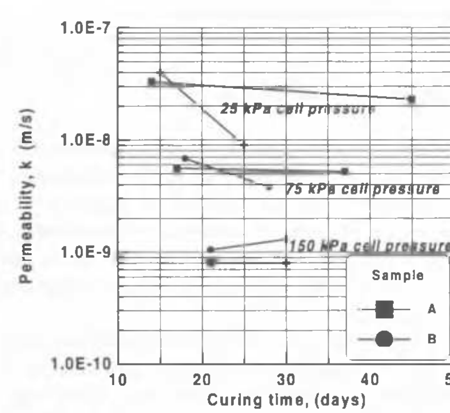

Figure 3. The effect of cell pressure on the permeability of treated soil.

The results above show that the permeability decreases with increasing cell pressure and to a certain degree with increasing time of curing.

Another test series consisted of samples taken from mixed- in-place columns. As was pointed out before, two types of sample sizes were used, see Table 1. The samples were taken from different depths. The applied cell pressure in the triaxial cell was selected to be equal to the horizontal pressure occurring at the depth at which the samples were taken, which in turn was equal to the initial effective pressure added to 80 kPa surcharge. In the case of samples C and D, isotropic conditions were assumed. While in the case of samples E to G, deviatoric stress was applied to show the effect of the actual vertical effective pressure.

The samples were subjected to the cell pressure between 2 and 4 days before applying the hydraulic gradient. A constant head method was used with different hydraulic gradients. The results for different cell pressures, deviatoric pressure and hydraulic gradients are given in Table 2.

Table 2. Permeability evaluated from triaxial tests (i = hydraulic gradient)

			Permeability, k (m/s)			
Samples	σ'_1 kPa	σ'_3 kPa	$i = 0.67$	$i = 2.0$	$i = 6.0$	$i = 20.0$
E	130	27	3.9E-9	2.4E-9		
F	186	75		2.0E-9		
G	186	75		3.2E-10	2.6E-10	2.9E-9
C	27	27		2.0E-9	3.7E-9	
D	100	100		2.1E-9	2.0E-9	2.1E-9

Table 2 shows that the permeability of the treated soil was about 2.0E-9 m/s, and that it was independent of sample size. The reason for this is probably that the quality and the homogeneity of treated soil were extremely good. However, in most cases it is difficult to obtain such quality. The tests made on the untreated soil show that the permeability of the untreated soil was about 4.0E-10 m/s. This means that the permeability of treated soil was approximately 5 times that of the untreated soil.

For samples E to G the sample size was 150 mm × 300 mm. It can be noticed that there is a weak tendency of decreasing permeability with increasing cell pressure. In the case of sample G, the permeability increases with increasing hydraulic gradient.

On the other hand, no relations were found between cell pressure or hydraulic gradient and permeability for samples C and D whose dimensions were 50 mm × 100 mm.

Many tests were performed to obtain the permeability of samples of type H. The results show rather large variety in the values of the permeability, between 1.0E-7 m/s and 1.0E-9 m/s.

4.2 *Field test results*

There exist no standard methods so far for testing and evaluating the permeability of the lime or lime cement column in the field. Hansbo (1994) carried out pressure-permeameter tests on soil stabilised using an admixture of 25 g lime and 35 g cement per kg of natural clay. He determined that the permeability of the treated soil ranged from 5.1E-8 m/s and 3.5E-7 m/s, with an average value of 1.4E-7 m/s. The method used here is called the packer test and it is similar to the method used for calculating the permeability in a rock mass. The packer test was performed in situ in an open borehole through a lime cement column. Two inflatable seals, called packers, were used to isolate a particular segment of the open borehole. In that part there was a perforated pipe. The packers were expanded against the wall of the borehole using water pressure. The water pressure applied must be high enough to act as a seal but at the same time too high a pressure has to be avoided to prevent expansion and cracking of the column. Figure 4 shows details of the packer instrument. The water discharge through the perforated pipe to the treated soil was measured using constant head pressure.

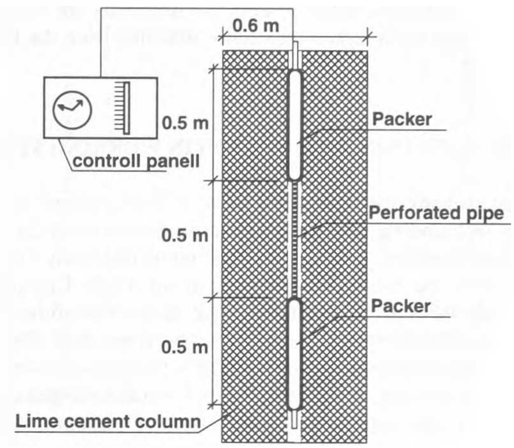

Figure 4 In situ instrument for determination of the permeability of lime cement columns.

It was assumed that the flow rate was stabilised by the end of the test, although this is generally not the case in practice. In other words the equation used to calculate the treated soil permeability was based on steady-state assumption. The equation can be written as:

$$k = \frac{1}{F} \cdot \frac{Q}{\Delta h} \tag{1}$$

where F is shape factor which varies depending on the assumed flow geometry.

It was assumed that the permeability of the untreated soil was low enough so that the flow through the lime cement column boundaries to the untreated soil could be neglected. As was pointed out previously the diameter of the treated soil is normally 0.6 m. This means that the radius of the boundary in the radial direction was limited. In order to find a shape factor used in equation 1, which was based on the above-mentioned conditions, a computer program SEEP was used. In order to verify the results obtained from the calculations with SEEP, a comparison was made with the results obtained by Brand and Premchitt (1980). This comparison gave almost identical results. Preliminary results show that the shape factor for the geometry used and for the treated soil boundary conditions is approximately equal to 0.45. The permeability of treated soil calculated using equation 1 is shown in Figure 5. The columns were selected close to each other in order to reduce the effect of soil type variation.

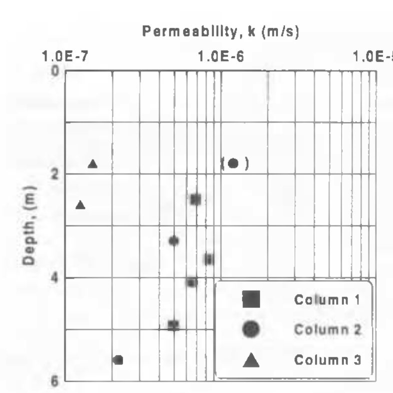

Figure 5 Permeability evaluated by the packer method.

Neglecting the measured value for column 2 at the depth 1.8 m, due to the observation of some leakage, the results from Figure 5 show that the permeability varies from 1.0E-7 to about 8.0E-7. By comparing the results from the laboratory tests with the results from Figure 5, it may be expected that the permeability of the lime cement columns obtained from the field tests are about two orders of magnitude higher than those obtained from the laboratory tests.

5 SOIL - COLUMN INTERACTION IN WORKING STATE

In order to study the interaction between lime cement columns and the surrounding untreated clay, a parameter study has been performed (Liedberg et al. 1996). The numerical study was performed with the finite element program PLAXIS. Calculations were made for axial symmetry as well as for two dimensional geometries. The study was concentrated on interaction effects on stresses and deformations due to stiffness variations between the column and the clay, and due to various installation geometries. Furthermore, the study included the effect of various roughness ratios at the clay-column interface on the transfer of stresses into and out of the columns.

All calculations were made assuming an ideal elasto-plastic Mohr-Coulomb material for the columns as well as for the clay. In this paper results are given only for the final stage, and the development of stresses with time is not dealt with. The model studied included a typical Swedish clay soil profile with a constrained modulus of compressibility M which increased from about 280 kPa in the vicinity of the surface to about 700 kPa at a depth of 15 meters. The angle of internal friction ϕ' of the clay was assumed to 30° and the cohesion intercept c' to 2 kPa. The properties for the various lime cement column qualities used in the study are given in Table 3 as relative qualities. Based on observations of generated pore-water pressure from several triaxial tests on in situ samples (Bergwall and Falksund 1996), the failure criteria was assumed to be drained.

Table 3 Stress-strain and strength properties of lime cement columns

Relative quality	M (MPa)	c' (kPa)	ϕ' (deg.)	ν
Low	10,5	30	35°	0,3
Normal	17,5	50	35°	0,3
High	35	100	35°	0,3
Very high	175	175	35°	0,3

5.1 *Calculations for a single column*

The calculations were made for various installation geometries in accordance with the model given in Figure 6.

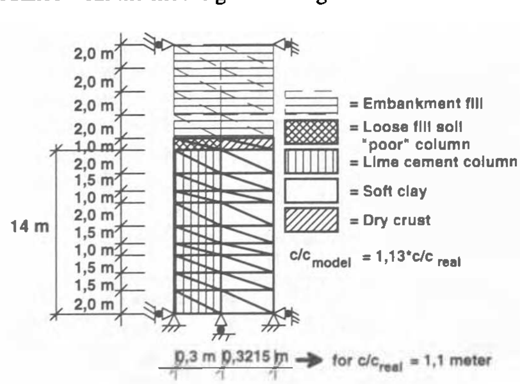

Figure 6 Principle finite element mesh for the axial symmetrical calculations.

The shear stresses along the periphery of the columns are given in Figure 7. As can be seen, the shear stresses in the final stage are close to zero. The commonly used assumption that horizontal planes remain horizontal seems to be valid, except for the upper and lower part where some load transfer occurs.

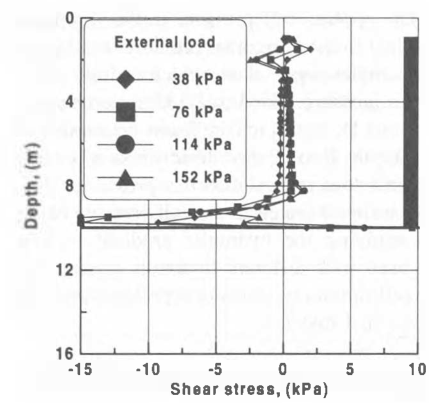

Figure 7 Shear stresses along the periphery of a floating column of "normal" quality for different external stresses.

The shear stresses shown in Figure 7 are positive when loads are transferred from the clay into the column and negative when the load carried by the column is transferred to the surrounding clay. Assuming drained conditions in the column, plastic zones develop from the top of the column and down for a given load increase in the column. Therefore, negative shear stresses can occur even along the upper part of a column as an effect of plastic deformation of the column.

It is interesting to note that the transfer length does not significantly increase with increasing load even though the plastic zone in the column increases. Calculations have also shown that the transfer length is little affected by the roughness ratio at the clay column interface. However, the load carried by a floating column decreases with decreasing roughness ratio, as shown in Figure 8. The consequence of this will lead to a larger portion of the load being carried by the surrounding clay which in turn will lead to larger settlements.

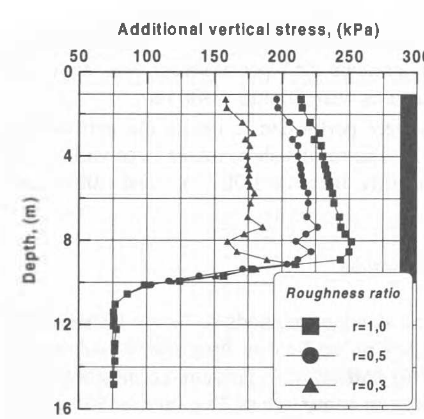

Figure 8 Additional stresses in a 10 meter long floating lime cement column with low quality for various roughness ratios and a constant external load of 76 kPa.

For an increase of the plastic zone in the lime cement column a larger part of the load will be carried by the clay, which will lead to larger settlements. In other words the quality of the column plays an important role. If the quality is high, i.e. high strength and modulus, in relation to the stress level, plastic deformations will not occur and, hence, a larger part of the load will be carried by the column, which will result in smaller settlements.

Figure 9 shows how the quality of the column affects the deformations in the floating lime cement column as a function of external stress without taking the settlement in the unstabilised clay under the column into account. The diagram shows a drastic reduction of deformations with increasing quality of the column.

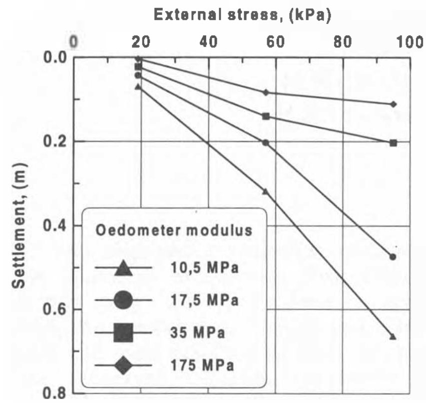

Figure 9 Column deformations as a function of stress and quality of the column.

5.2 *Calculations for two dimensional geometry*

In the 2D-study the columns were modelled as continuous column walls of infinite length so that the soil-column system had the same vertical stiffness per unit length as in the real 3D-case. Thus columns with a diameter of 0.6 m were modelled as 0.3 m wide walls, see Figure 10.

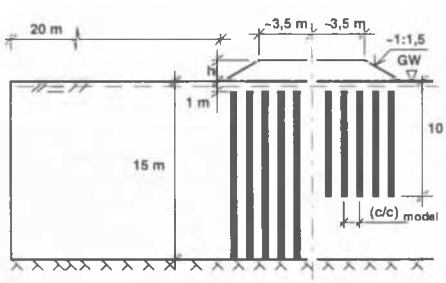

Figure 10 Schematic figure of the 2D-model.

For a column located under the toe of an embankment, which is exposed to certain horizontal deformation, such an assumption may lead to an overestimation of the horizontal deformation due to the relatively lower bending stiffness of the column walls compared to the real columns. The results from the simulations show that a column under the toe of an embankment is exposed to larger horizontal displacement if the column is installed as a floating column compared to a column installed to a firm bottom.

Furthermore, the vertical stresses in a floating column under the toe of the embankment are relatively larger than in the case of a column installed onto a firm bottom. In Figures 11 and 12, the distributions of additional vertical column stresses are given as a function of depth for three different locations under the embankment. Stress distributions are given for columns located under the middle of the embankment as well as for columns located under the edge and under the toe. As a consequence of relatively larger horizontal displacements and stress concentrations in a floating column at the toe, such a column is more stressed than a toe column installed onto a firm base. The stress concentration at the toe means that the column installation should be looked upon as a load with a stiffness rather than a flexible load, which is the most common assumption when it comes to the calculation of additional stresses in the untreated clay under the floating columns.

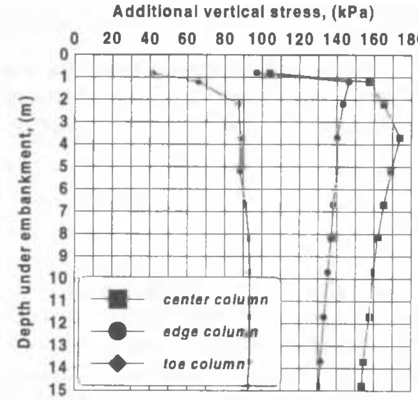

Figure 11 Vertical additional stresses in three different columns in a section of a three meter high (57 kPa) embankment. The columns were of normal quality and installed onto a firm bottom.

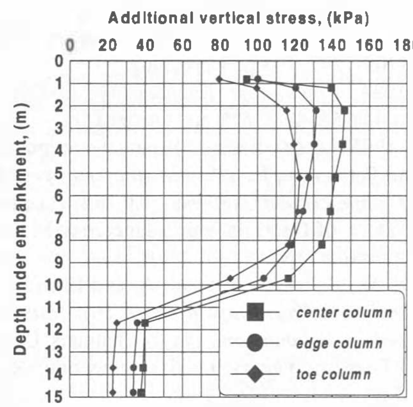

Figure 12 Vertical additional stresses in three different floating columns in a section of a three meter high (57 kPa) embankment. The columns were of normal quality.

6 CONCLUSIONS

The in situ method for determination of the modulus of deformation seems promising. However, further testing is needed in order to fully understand the stress-strain properties under field conditions of lime cement columns. In particular, columns with zones of lower quality where failure occurs locally are of interest.

The in situ method for determination of field permeability gives a more realistic picture of the permeability of lime cement columns, since macro effects and the effects of the installation are taken into account. On the other hand the heterogeneity of the permeability needs to be addressed further.

The numerical study of clay-column interaction showed that:

1 Horizontal planes remain horizontal except for a shorter upper and lower zone over which load transfer occurs.
2 The roughness along the periphery of the column does not significantly affect the transfer length. However, the magnitude of the additional stresses is affected and, hence, also the settlements.
3 The deformations in clay stabilised with lime cement columns strongly depend on the stresses carried by the columns and, hence, also by the column quality.
4 Columns at various locations under an embankment are exposed to various stresses and deformations. In particular stress concentrations occur for floating columns located under the toe.

7 ACKNOWLEDGEMENT

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REFERENCES

Bergwall, M. and Falksund, M. 1996. *Permeabilitetens och styvhetens inverkan på konsolideringsförloppet i kalkcementpelarförstärkt lera (Consolidation of lime cement columns - permeability and stiffness),* Chalmers University of Technology, Dept. of Geotechnical Engineering, M.S.C - thesis 1996:1, 105p.

Brand, E.W. and Premchitt, J. 1980. *Shape factors of cylindrical piezometers.* Géotechnique 30, No. 4, 369-384.

Broms, B. 1984. *Stabilisation of soil with lime columns.* Design Handbook, Third Edition, Lime Column AB, 51 p.

Carlsten, P. and Ekström, J. 1995. *Kalk- och kalkcementpelare, Vägledning för projektering, utförande och kontroll (Lime- and lime-cement columns, Guidance for project planning, execution and inspection).* SGF (Swedish Geotechnical Society) Report 4:95, ISSN 1103-7237, 103 p., Linköping.

Hansbo, S. 1994. *Foundation Engineering.* Elsevier Science B.V., ISBN 0-444-88549-8, 519 pp, Amsterdam.

Holmqvist, L. 1995. LC Markteknik, Personal communication.

Kernell, H. and Pettersson, B. 1996. *In situ measurement of permeability of lime cement columns.* Chalmers University of Technology, Dept. of Geotechnical Engineering, M.S.C - thesis unpublished progress report.

Liedberg, N. S. D., Baker, S., Smekal, A. and Ekström, J. 1996. *Samverkan mellan kalkcementpelare och lera (Interaction between lime cement columns and clay).* Chalmers University of Technology, Dept. of Geotechnical Engineering, Report No. B1996:1, 107 p.

Terashi, M. and Tanaka, H. 1993. *Settlement analysis for deep mixing method.* Proc. 8th European Conf. On Soil Mech. And Foundation Eng., Helsinki, Vol. 2, pp. 955-960.

Åbjörn, S. and Linnér, R. 1995. *Kalkcementpelares deformationsegenskaper - Provbelastning i fält (Deformation properties of lime cement columns - load tests in situ),* Chalmers University of Technology, Dept. of Geotechnical Engineering, M.S.C - thesis 1995:4, 85 p.

Åhnberg, H., Johansson, S.-E., Retelius, A., Ljungkrantz, C., Holmqvist, L. and Holm, G. 1995. *Cement och kalk för djupstabilisering av jord. En kemisk-fysisk studie av stabiliseringseffekter. (Stabilising of soil using cement and lime. - A study of soil improvements effects).* Swedish Geotechnical Institute, Linköping, Sweden, Report No. 48, 213 p.