Lime-cement columns on the 'Svealand' rail link: Performance observations

Piliers en chaux et ciment sous la ligne ferroviaire 'Svealand': Observation de la performance

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ABSTRACT: A reinforcement project using lime-cement columns was carried out on the "Svealand" rail link. The strength properties of the columns are compared using different field and laboratory tests. In addition, complete columns to a depth of 8 m were extracted. In the article, comparisons are made between settlements, obtained using measurements and two calculation methods, depending on how parameters are chosen. The calculation model normally used in Sweden (intended for lime columns) is discussed, as is the effect of the successive shortening of the columns which has been introduced.

RÉSUMÉ: Une technique de renforcement utilisant des piliers en chaux et ciment a été mise en oeuvre sur la liaison ferroviaire "Svealand". La résistance de ces piliers est comparée sur la base de différents essais effectués sur le terrain et en laboratoire, complétés par l'extraction de piliers entiers, descendant jusqu'à une profondeur de 8 m. L'article présente des comparaisons de tassement, à l'aide de valeurs obtenues par mesure et par deux méthodes de calcul selon les paramètres choisis. Le modèle de calcul normalement utilisé en Suède (applicable aux piliers en chaux) est également discuté, ainsi que l'effet du raccourcissement progressif des piliers auquel on a commencé à avoir recours.

1. INTRODUCTION

Lime-cement columns were used on the "Svealand" rail link, a single-track railway situated in the vicinity of Stockholm. The track runs between Södertälje and Arboga. The total length is 120 km, where 80 km are new construction and 40 km reconstruction. The installations were carried out in March 1994. The railway was built on an embankment 2 m in height, which was constructed in February 1995. The clay in the area is up to 25 m thick and equal numbers of columns descend to 5, 10 and 15 m depth, see Figure 1. The primary purpose of the columns is to reduce settlement. A comprehensive follow-up study of this project is in progress and parts of it will continue during 1997.

The columns have a diameter of 600 mm and a c-c spacing of 1.0 m at ground level. The lime-cement mix is 50-50 % by weight, which makes the columns stronger and stiffer than if lime alone is used. The amount of lime-cement is 23 kg/m column. The stabilization medium used in Sweden in recent years is almost exclusively a mix of lime and cement.

2. PROPERTIES OF THE SOIL AND COLUMNS

2.1 *Geotechnical properties of the soil*

The soil layer sequence consists of 1 m of fill at the top. The fill is from an old embankment which lay 0.3 - 0.4 m above the surrounding ground surface. Organic- clay is found at a depth of 1 - 7 m, overlying a clay layer up to 17 m thick. Lower down the clay is varved. Below this level, till is on rock. The clay is normally consolidated. In the follow-up area, the total clay depth is about 18 m. Soil properties are shown in Table 1.

Table 1. Soil properties

Soil	Depth (m)	τ_{fu} (kPa)	w (%)	γ (kN/m³)	w_L
Organic-clay	1-7	6-15	90-120	14-15	0.90-1.35
Clay	7-18	14-20	60-90	15-17	0.55-0.95

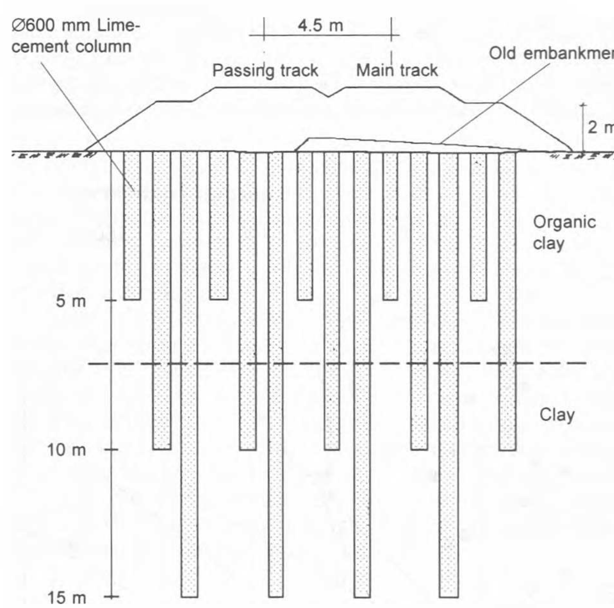

Figure 1. Cross-section of railway embankment and lime-cement columns.

2.2. *The properties of the columns*

The properties of the columns were determined using several different methods both in the field and the laboratory. In Sweden, tests are often carried out in the laboratory at the design stage where soil samples from different depths are mixed with stabilizers to assess the shear strength of the columns. In this case, soil from different depths have been mixed with lime and cement (50 % each by weight). Unconfined compression tests have been carried out 7, 20, 56 and 180 days after the mixing. The results are shown in Figure 2.

In the field, lime column soundings and inverted lime column soundings were used to confirm shear strength of the columns. These methods are usually used in Sweden to check the columns as described by Carlsten and Ekström (1995). Lime column soundings give the best result in the upper parts of the columns down to about 5 m depth. At greater depths, the probe can leave the column and give misleading results. Inverted lime column soundings are better at greater depths, since a cable has been pre-installed in the bottom of the column. The method can in some cases cause problems if the strength of the column is too high. In this project, a comparison between the methods has shown large differences. Inverted lime columns soundings have therefore been correlated with the results of lime column soundings and only half of the value of the shear strength has been counted. This is the result of this project but it does not allow any general conclusions concerning the correlations. An assessment of the sounding results is shown in Figure 2. One reason for the difference can be that an accumulation of stabiliser has occurred at the centre of the columns with pre-installed cables.

Tests in the field were carried out in six columns using core drilling 1, 5 and 9 months after the installation of the columns. The samples which were extracted have a diameter of 50 mm. The method is most suitable for high strength columns but an attempt was made here to apply the method to relatively soft columns. Eight months after installation, whole columns were extracted using a Finnish sampler which consisted of a two-chamber steel pipe 600 mm in diameter with a shutter at the lower end. The pipe was driven or pressed down by a pile driver. Two columns were extracted to a depth of 7 to 8 m. In one column, the pipe could be pressed down most of the way and the sample is therefore assumed to be less distorted than the others where driving was carried out. A number of smaller samples were extracted from the least distorted sample as well as three larger sample pieces with the full width and a length of 0.5 m each.

The extracted columns were inspected visually. It could be observed that the outer part of the columns consisted of an up to 20 mm thick crust. The distribution of the stabliser was similar along the length of the column, i.e. the column was equally thick along its length. When a cross-section of the lime-cement column was studied, it could be seen that the column consisted of well-mixed lime-cement and clay, and also pockets of pure clay, and layers of pure lime cement. The layers of stabiliser formed weakness zones or cracks in the tests.

Unconfined compression tests were carried out in the laboratory on the samples from the core drilling and the samples trimmed from the extracted columns. Samples taken with a core sampler showed low shear strength, less than 40 kPa and with many values around 5 kPa. The highest value was obtained for two samples taken with modified core drilling equipment, of 61 and 70 kPa respectively. No increase in shear strength with depth could be noted. Unconfined compression tests on the trimmed samples gave shear strength values varying from a few kPa up to a maximum of 150 and 210 kPa. A compilation is shown in Figure 2. Most of the results of the unconfined compression tests on core samples gave unreasonably low values and a comparison between these and the soundings gave no correlation between the results, i.e. high sounding resistance did not correspond to a high shear strength. There could be several possible reasons for the low shear strength values. In sampling in these relatively soft columns, there is a risk that hard pieces of the columns are broken loose and rotate with the core causing distortion of the samples. The method seems in this case not to give representative values of the properties of the columns.

In order to get a better idea of the properties of the columns, large scale triaxial tests were carried out with a diameter of 500 mm and a length of approximately 500 mm on lime-cement columns. The tests were carried out by the Danish Geotechnical Institute according to a programme drawn up by the Swedish Geotechnical Institute (SGI). In these tests, the most important goal was to obtain consolidation parameters for the columns. Both drained and undrained tests were performed according to a schedule shown in Figure 3. In step 2 the load was set at a level corresponding to the strainlevel of the final conditon. In step 5 and 8 the load is made to failure. The samples were taken at depths of 1.9, 2.4 and 6.4 m. Table 2 shows the parameters that have been chosen from the trials. The tests are described by Stensen-Bach et al (1996).

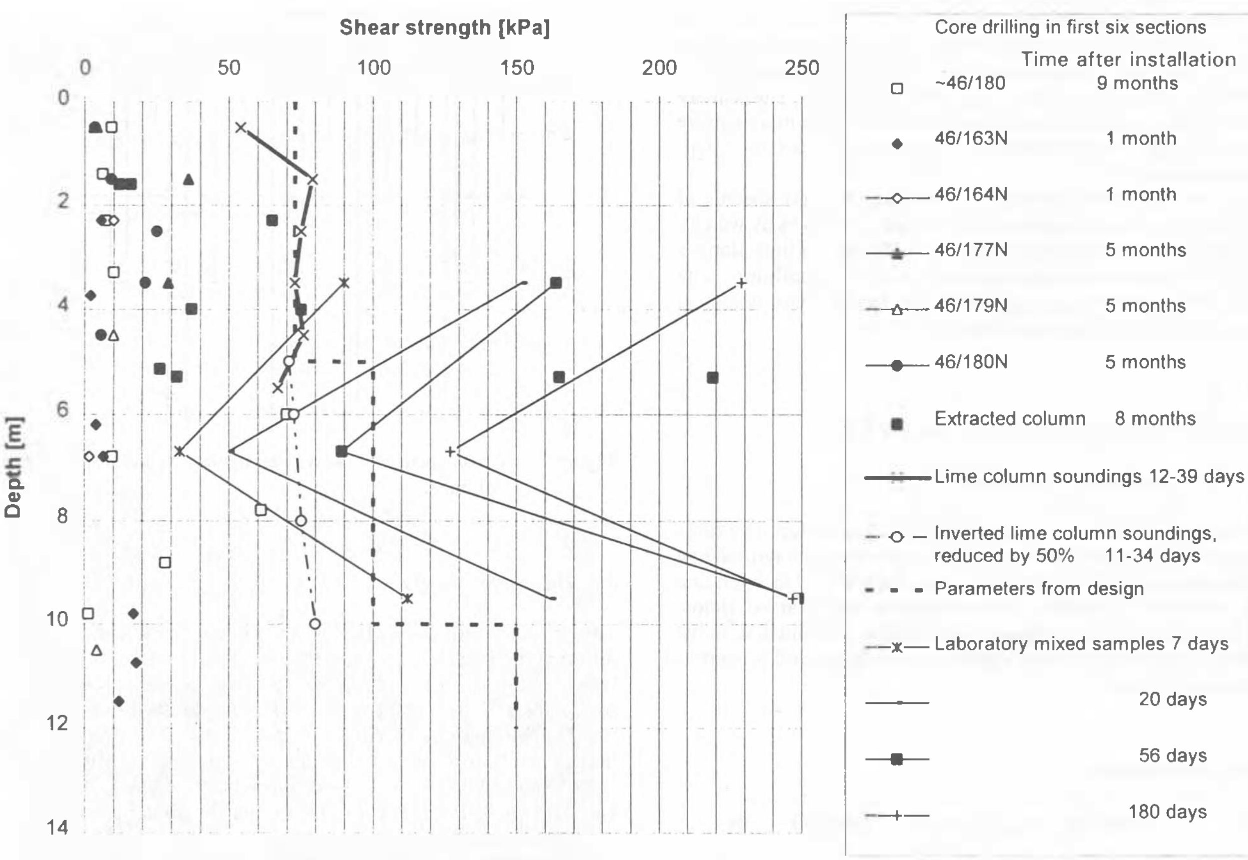

Figure 2. Diagram of shear strength against depth. Shear strength is determined by unconfined compression tests from core drilling, samples trimmed from an extracted column and laboratory mixed samples and evaluated results of soundings.

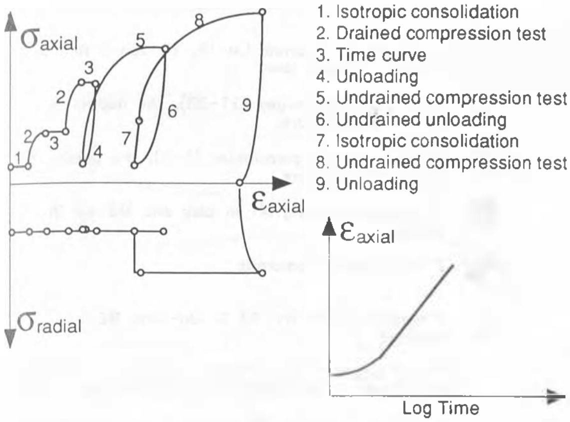

Figure 3. Schedule for carrying out triaxial tests with samples of diameter 500 mm. From Steensen-Bach et al (1996).

3. FIELD MEASUREMENTS

In order to follow up the project, measurements were concentrated to an area around section 46 + 180. Figure 4 shows the measurement station where settlements, horizontal movement, pore pressure and temperature were measured. Supplementary settlement observations have also been carried out in other sections.

3.1. *Settlements*

The actual settlements are monitored using horizontal measuring hoses, bellows hoses and magnetic screws. Four bellows hoses were installed in the clay and one in a column, while three magnetic screws were installed in the columns and one in the clay. The magnetic screws are installed at a depth of 5.5. m while the bellows hoses in clay extended the whole depth of the clay, i.e. about 18-20 m. In order to construct the railway embankment the magnetic screws had to be removed in January 1996. Three of five bellows hoses were removed in March 1996.

Figure 5 shows the bellows hose measurements 11 months after the construction of the embankment. The Figure shows that the columns have functioned as intended since the settlements were evenly distributed within the column-reinforced depth. The same tendency could also be seen on previous measuring occasions. The same also applies to the two measuring hoses which remain after 19 months' measurement. No increase in settlement can be registered at those levels where the columns ended, i.e. at 5, 10 and 15 m depth. Settlement is somewhat greater in the uppermost layers closest to the ground surface and under the columns.

Temperature measurements have been carried out. They show an increase in temperature of 5-6 °C between the columns in the upper part of the clay layer (depth 2.5 m and 7.5 m) just after the installation of the columns. At 12.5 m and 17 m depth almost no changes were observed. Inclinometer tubes were installed to determine the size of the horizontal movements during loading. The temperature gauges as well as the inclinometers are linked to an automatic measuring system. The inclinometers and temperature gauges were removed in January 1996.

The inclinometer measurements show very small horizontal movements. In the inclinometer tube 4 m from the middle of the bank, movements at most are about 25 mm and in the two other inclinometer tubes placed 6 and 8 m from the middle of the bank a few mm. This indicates that the lateral displacements are small.

3.2 *Excess pore water pressure measurements*

Measurement of pore pressure has been going on for two years, both before and after the loading of the embankment, both within the column-reinforced area but also at a reference station outside the column area. The pore pressure piezometers were connected to an automatic measurement station in August 1994, about five months after the installation of the columns. The results are shown in Figure 6. At 2.5, 7.5, 12.5 and 17 metres depth, two piezometers have been installed at each depth within the column-reinforced area and one piezometer at the reference station. Due to cable failures, some irregularities have occurred in the graph. In January 1996 all the piezometers were removed and then reinstalled. Before the loading in February 1995, a excess pore water pressure of a few kPa was apparent compared to a ground water table 0,8 m below the ground surface. The variations at the reference station under the period of measurement were 8 kPa at 2.5 metres' depth and 13 kPa at 17 metres' depth. At 7.5 and 12.5 m depth, the conditions are more stable. Excess pore water pressure in the underlying till varies and during some periods it corresponds to the ground surface. During the measurement period up to the construction of the embankment, i.e. from August 1994 to February 1995, a difference in pore pressure is seen between the column-reinforced area and the reference station. The pore pressure is higher in the column area than at the reference station except for at 2.5 metres below the ground surface where they are about the same. The difference in pore pressure was at most 16, 24 and 15 kPa at 7.5, 12.5 and 17 metres depth respectively, which is judged to be a result of the installation of columns.

Figure 7 shows the excess pore water pressure at different points in time against depth. Between 10 and 16 m in depth, the pore pressure falls relatively slowly. The drainage paths for the clay are relatively long, both to the columns (large c/c distance at these levels) and to the drainage layer in the lower part of the soil profile. Between 5 and 8 m, the reduction of the excess pore water pressure takes place more quickly. In the lower layers, the excess pore water pressure is affected by fluctuations in the underlying ground water reservior, which makes an assessment of the development over time of settlement in the clay layers more difficult.

Table 2. Parameters used for calculation of settlements.

	Parameters used in design				Parameters from triaxial test		
Hight of layer (m)	M_L *1) (kPa)	τ_{fu}, column (kPa)	α *2)	M_{column} *3) (kPa)	τ_{fu}, column (kPa)	Corre-sponding α *2)	M_{column} *3) (kPa)
0.80	350	75	50	3750	115	208	24000
2.20	260	75	50	3750	115	208	24000
2.00	250	75	50	3750	115	208	24000
2.00	270	100	75	7500	160	175	28000
3.00	235	100	100	10000	160	200	32000
2.50	280	150	125	18750	160	225	36000
2.50	330	150	125	18750	160	250	40000

*1) M_L = Oedometer modulus in clay in linear range between the preconsolidation pressure and the yield stress.
*2) α = an empirical soil dependent coefficient. Equation 1.
*3) M_{column} = Confined modulus of the column material.

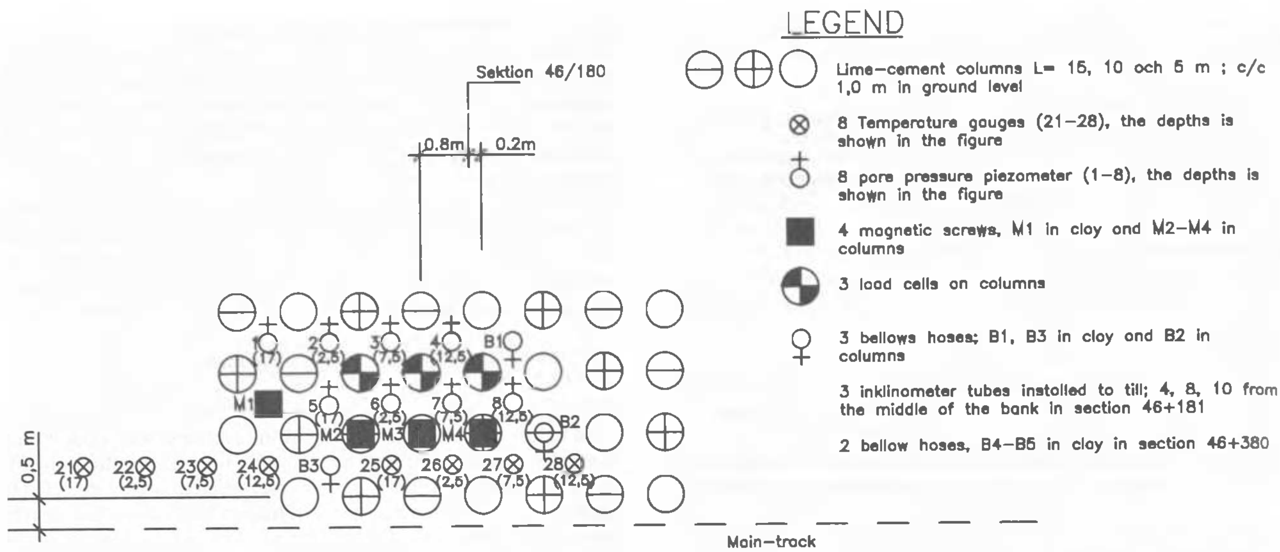

Figure 4. Measurement station at Section 46+180.

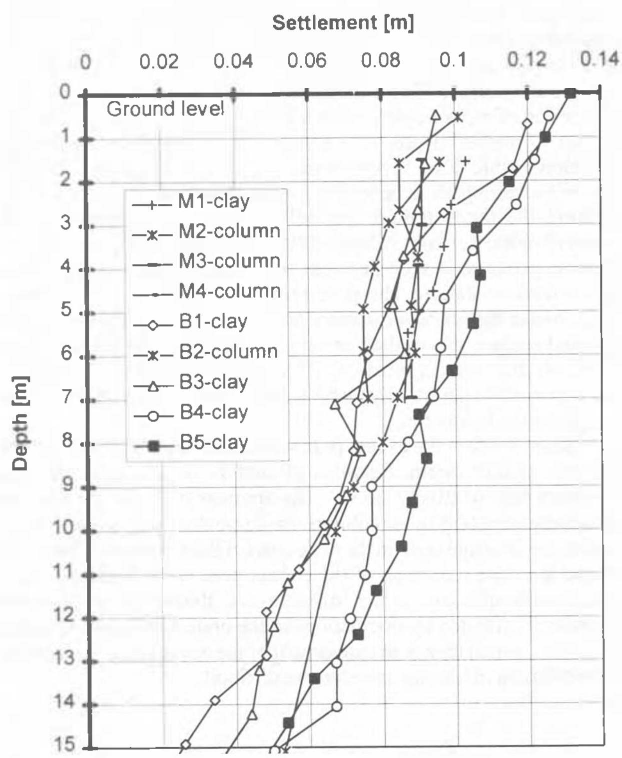

Figure 5. Measurement result for bellows hoses and magnetic screws in January 1996, 11 months after construction of the embankment.

Permeability measurements in clay were carried out at levels where pore pressure piezometers had been installed. The measurements gave an average value of about $2 \cdot 10^{-10}$ m/s. Compression tests in the laboratory indicated a permeability of about $4 \cdot 10^{-10}$ m/s.

4. COMPARISON BETWEEN CALCULATED AND ACTUAL SETTLEMENTS

4.1. *Calculated settlement*

A computer programme LIMESET (Carlsten 1989) were used at SGI to claculate the settlements.

The programme is based on the assumption that the settlements in the soil and the columns are equal at each level. The compression modulus in the columns, M_{column}, is normally based on an empirical approach:

$$M_{column} = \alpha \cdot \tau_{fu} \qquad (1)$$

where the undrained shear strength, τ_{fu}, is determined from unconfined compression test and the ratio, α, is an empirical soil dependent coefficient. The values used in the design phase are shown in Table 2 together with values from the triaxial tests.

The computer programme calculates a time period but is really only correlated against settlement at 80-90 % consolidation. In the article therefore, no comparison is made between calculated and actual time periods.

4.2 *Settlements occurring*

Compression measured 11 months after the embankment was filled, using bellows hoses and magnetic screws is shown in Figure 5. The Figureure shows that the settlements are of the same magnitude in and outside the columns. There is however a tendency to somewhat lower settlement in the columns. Figure 8 shows the settlements against time for bellows hoses. The settlements are shown only for the uppermost measurement point which is at somewhat different depths under the surface for the different measuring hoses. A comparison of the settlement size with the depth shows that the settlements in the upper layers take place more quickly while they are slower in the lower layers where the drainage paths are longer.

The settlements in the vicinity of the measurement station measured with horizontal measuring hoses shows that the settlements are somewhat larger than those which were measured with bellows hoses and magnetic screws, which may be due to their not lying in exactly the same section and that the horizontal measuring hoses are located at the ground surface while the other measurement hoses are installed somewhat deeper. The tendency however is that the settlements are still occurring.

4.3. *Comparison*

A comparison of the settlements in the columns and clay, which was shown in Figure 5 indicates that flat cross-sections remain flat and thereby meet the requirements imposed on the calculation model. Closest to the shortening of the columns there will be some irregularities in the cross-sections due to the load distribution between clay and column.

Calculated settlements have been compared with the measured ones in Figure 9 in September 1996. The excess pore water pressure increased from March to September, probably because of the seasonal variations. It is assessed that the excess pore water pressure which existed in March 1996 will cause further settlements. A

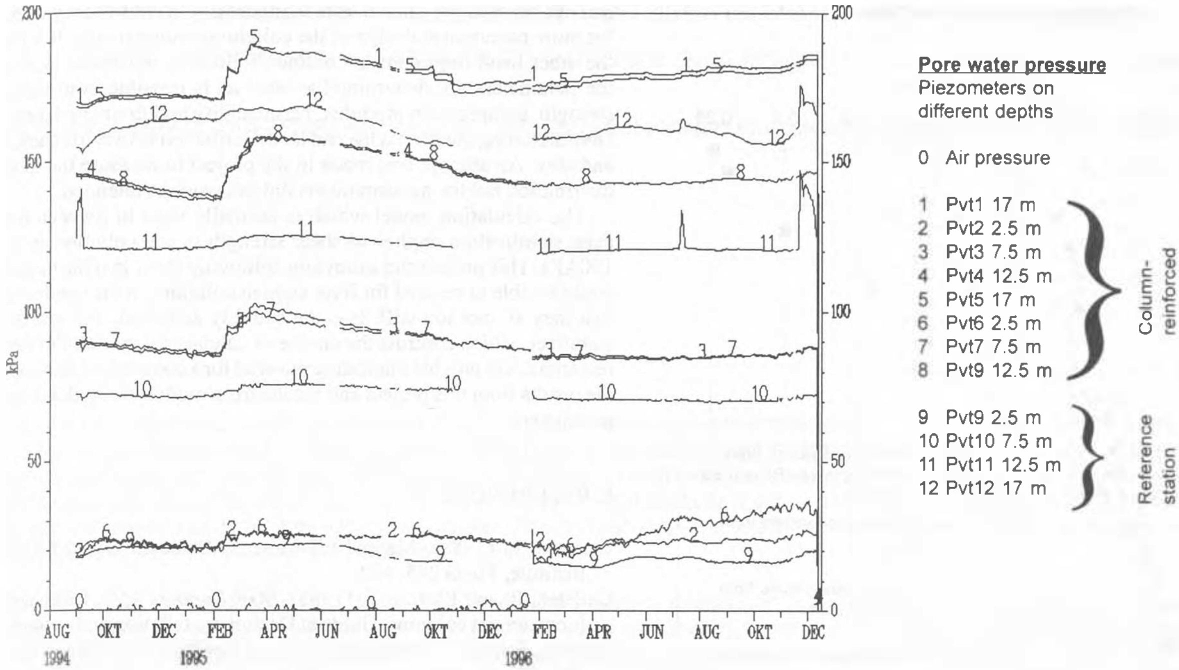

Figure 6. Pore pressure follow-up with time at different levels. Measurements carried out at a reference station and within the column-reinforced area.

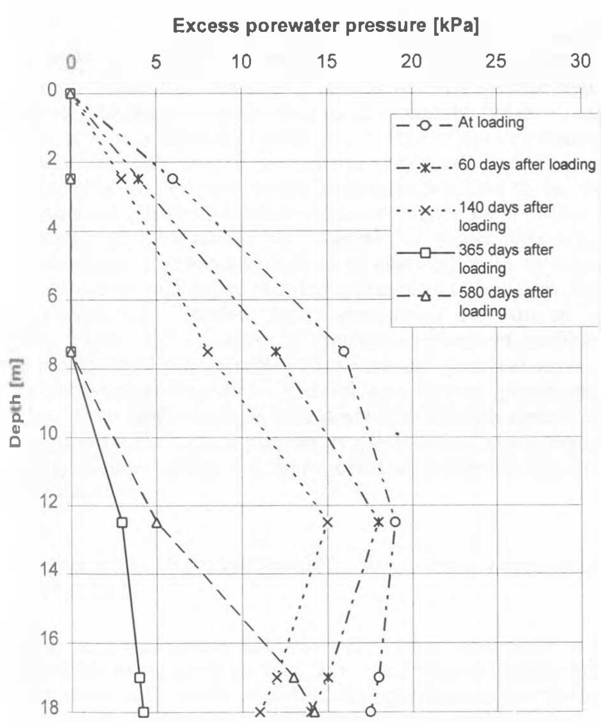

Figure 7. Distribution of excess pore water pressure in clay against depth at different points in time after loading of railway embankment.

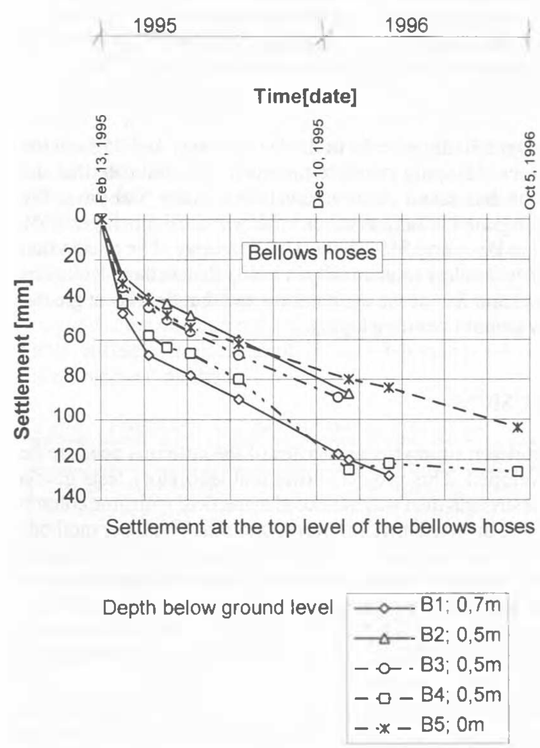

Figure 8. Follow-up of settlement with time for the upper level of bellows hoses.

prognosis has therefore been made in Figure 9 where the settlements at different levels have been assumed in relation to how much of the excess pore water pressure remained in March and that this is in addition to the settlements measured at that point in time. At the depth 0-2 m the measured settlements are much larger than those calculated whereas the settlements at the depth 10-12 m are much smaller. However, at the depth 2-10 m, the agreement between measured values and the values from the triaxial tests are good.

Regarding the use of the method, the picture of the settlements is satisfactory from a practical point of view, but there are differences in the absolute values.

The calculation model is partially empirical and uses the total stress in the calculations. It is possible, that a model using drainage parameters would give better agreement. Though, this an issue for future research.

In the calculations, 85 % consolidation was achieved in the 0-

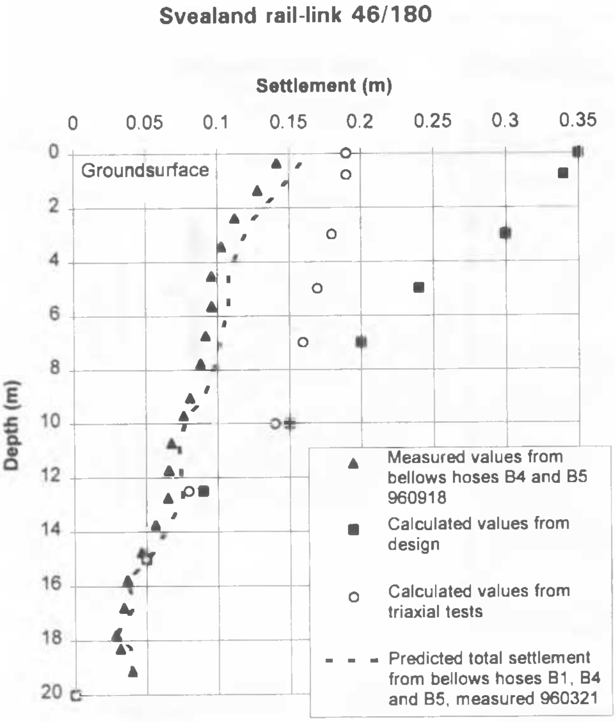

Figure 9. Measured compression in bellows hoses B4 and B5 in September 1996 and calculated compression on the basis of projected parameters or parameters chosen from triaxial tests. Prognosis of compression based on measured values in bellow hoses in March 1996.

5 m layer after 580 days, 65 % in the 5-10 m layer and 35 % in the 10-15 m layer. The pore pressure measurements indicate that the consolidation has taken place much faster. After 580 days, the corresponding consolidation is about 100% in the 0-5 m layer, 95% in the 5-10 m layer and 85% in the 10-15 m layer. The reason that the drainage takes place more rapidly in reality than in the calculations can be cracks and fissures in the columns and that the clay at greater depths may contain draining layers.

5. CONCLUSIONS

Methods of determining the properties of the columns needs to be further developed. This project shows that laboratory tests give a higher shear strength than was measured in the field with lime column soundings and inverted lime column soundings. The latter methods do not give a consistent result; control methods in the field should be developed further. This will take place within the framework of Svensk Djupstabilisering. The method of core drilling is suitable for stiffer columns but did not function so well in relatively soft and brittle columns. It is possible that the method can be modified to suit also this type of column.

Dimensioning with shortened columns according to depth gives large financial gains and this project has shown that the method works well.

Figure 9 shows that the calculated settlements from design are more than double the measured settlements. If the properties of the columns from the triaxial tests are used, then the difference is less than 0.05 m. The distribution of the relative deformation with depth, calculated and actual, shows the same pattern. There are great differences in result depending on the manner of choosing the entrance parameters. An explanasion to these differences may be an effect of the confining pressure. The parameters from the triaxial tests are judged to give a better picture of the actual properties of the columns. The tests show that the ratio of the shear strength and modulus can be greater than what is usually used but it is still not possible to draw any general conclusions from this. It is also too early to say whether triaxial tests are necessary in consultancy work for more economical design of the column reinforcements. It is on the other hand important to continue follow-up measures so that the parameters are determined as well as is possible both shear strength, compression modulus, permeability and drainage layers. One interesting question is the real load distribution between columns and clay. An attempt was made in the project to measure the load distribution but the measurements did not work as intended.

The calculation model which is normally used in Sweden for deep stabilization applies to shear strength of the columns up to 150 kPa. This project and a previous follow-up show that the model could be able to be used for lime-cement columns on the condition that they are not too stiff. It is the stability achieved, and not the stabilizer, which controls the choice of calculation model. Further research could provide interesting material for a comparison between the results from this project and results from models using drainage parameters.

6. REFERENCES

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