A CASE STUDY ON THE EFFECTS OF USING SURCHARGE FILL AS A COMPLEMENT TO GROUND IMPROVEMENT WITH DRY DEEP MIXING

Stefan Larsson, KTH Royal Institute of Technology, Stockholm, Sweden
Sebastian Adevik, ELU Consultant, Stockholm, Sweden
Razvan Ignat, Skanska Sverige AB, Stockholm, Sweden
Sadek Baker, Skanska Sverige AB, Stockholm, Sweden

Abstract Uncertain strength, deformation and flow properties in dry deep mixing columns often lead to conservative choices concerning design values where the design is regularly complemented with a surcharge fill. The surcharge not only leads to increased cost but also to logistical problems where the surcharge is a physical obstacle. The deformation properties of a ground improvement were studied by means of two small embankment load tests. The settlement measurements were also used in FEM analyses, where the effect of surcharge on creep settlements was investigated. Based on the results, the effects of the surcharge were considered negligible and could safely be ignored. The project is an example how the contractor can use a field test as an effective design approach.

Introduction

Ground improvement of soft soils with dry deep mixing columns, or lime-cement columns, is often combined with a surcharge fill. The main reason for using a surcharge is to force the primary settlement and some of the secondary settlement that would have occurred under the final embankment height alone, to take place under the surcharge loading. Another reason is to level out uneven settlements. Strength and deformation properties in improved soil cannot be sufficiently assessed in advance, before columns have been tested on-site, and this uncertainty often lead to conservative choices concerning design values. In order to compensate for a slow forecasted consolidation process, the design is regularly complemented with a surcharge. The surcharge not only leads to increased cost but also to logistical problems where the surcharge is a physical obstacle.

Extensive research undertaken by Åhnberg (2006, 2007) shows, by means of triaxial and oedometer tests, that cementation processes and a load acting on the columns during hardening, makes the improved soil to behave as a heavily over-consolidated soil due to an increase in quasi-preconsolidation pressure. The effects of using a surcharge on the time for consolidation and creep properties are thus probably small, since the stress level does not normally exceed this quasi-preconsolidation pressure. A study presented by Venda Oliveira et al. (2013) shows that the stress level within small ranges has limited effect on the creep properties.

This paper presents a case study where the deformation properties of a ground improvement using dry deep mixing columns was investigated by load tests on two small test embankments. The tests were performed at the *Interchange Värtan* project site, which is part of the *Norra länken* infrastructure project in Stockholm. The tests were not performed as a research project but simple load tests performed by the contractor with the purpose of rationalizing the design delivered by the client. One purpose of the load tests was to assess the deformations when the columns are loaded temporarily by overlaying concrete works. Another purpose was to provide some input data to a study of the effects of using surcharge on creep settlements. The analysis was conducted as finite element modeling using the commercial code PLAXIS, where creep settlements can be simulated using the built-in Soft Soil Creep model.

Description of the project

The load tests were conducted at the *Interchange Värtan* project site, which is part of the *Norra länken* infrastructure project in Stockholm, Sweden. *Norra länken* is one of northern Europe's largest road tunnel projects. The hub of the *Värtan* project is a new junction where the *Norra länken* highway is joined with the harbor, a local road network, and a major road. During construction of this complex interchange, some 40,000 cars and trams pass the site every day, making the project logistically complex.

The geological and geotechnical conditions vary at the site, but at the location for the two trial embankments the soil consists of fill material (1 m), a dry crust of gray clay (1 m), soft sulphide bearing clay (4-7 m), sand and silt (8-10 m), and moraine (1-2 m) on rock. The phreatic level is located just below the dry crust. The ground improvement is performed mainly for the local roads that are located at the original ground level. The embankment height is thus low. The ground improvement was designed by the client with dry deep mixing columns with a 1 m center-to-center distance. The length of the columns was about 7 m and the diameter 0.6 m. According to the design, the ground improvement was to be complemented by a surcharge of 1.0 m with 3 months of surcharge time. Since the area of the project is relatively small, a surcharge fill would constitute an obstacle.

The main new interchange is located above a large concrete construction. During construction of the concrete structures, the mold and stamps are founded on the underlying ground that is partly improved by means of dry deep mixing columns. One question that arose was the magnitude of the settlement due to the loading from the concrete works, which was estimated to be about 25 kPa. When the deformations can be sufficiently assessed, the mold can be pre-cambered with the corresponding magnitude of deformation. The columns are loaded for just under a week, after which the concrete hardens and the loads are redistributed to the adjacent pile-founded concrete pillars.

The trial embankments

In order to study the deformation properties in the improved soil, the contractor performed test loadings at two spots in an early stage of the project. The construction sequences are shown in Figure 1 and described in the following.

1. After the areas had been cleared of obstacles in the ground, 8*16, 0.6 m in diameter, columns were installed in a square pattern for each of the two trial embankments. The center-to-center distance was 1.0 m and the length about 7 m. The binder was a lime and cement (50% / 50%) mixture and the binder content was 90 kg/m3.

2. After 4 weeks, the areas were excavated down to the terrace level, corresponding final road level minus 1.0 m. It was also checked that the columns were continuous through the dry crust. Eight settlement gauges was placed above and between the columns. The gauges consisted of reinforcement bars welded on 300*300 mm steel plates. The bars were protected by plastic tubes. The settlements were measured by total station.

3. One half of the test area was filled to a level corresponding to the level of the final road. The other half was filled to a level corresponding to the level of the final road plus 1.0 m. The settlements were measured directly after loading and then twice a day the first week followed by twice a week for eight weeks.

4. After 8 weeks, the surcharge was removed. The whole test area then had the level corresponding to the final road level.

5. After the gauges were measured, the whole test area was filled to a level corresponding to the level of the final road plus 1.3 m (approximately 25 kN/m2). The settlements were then measured for a number of weeks.

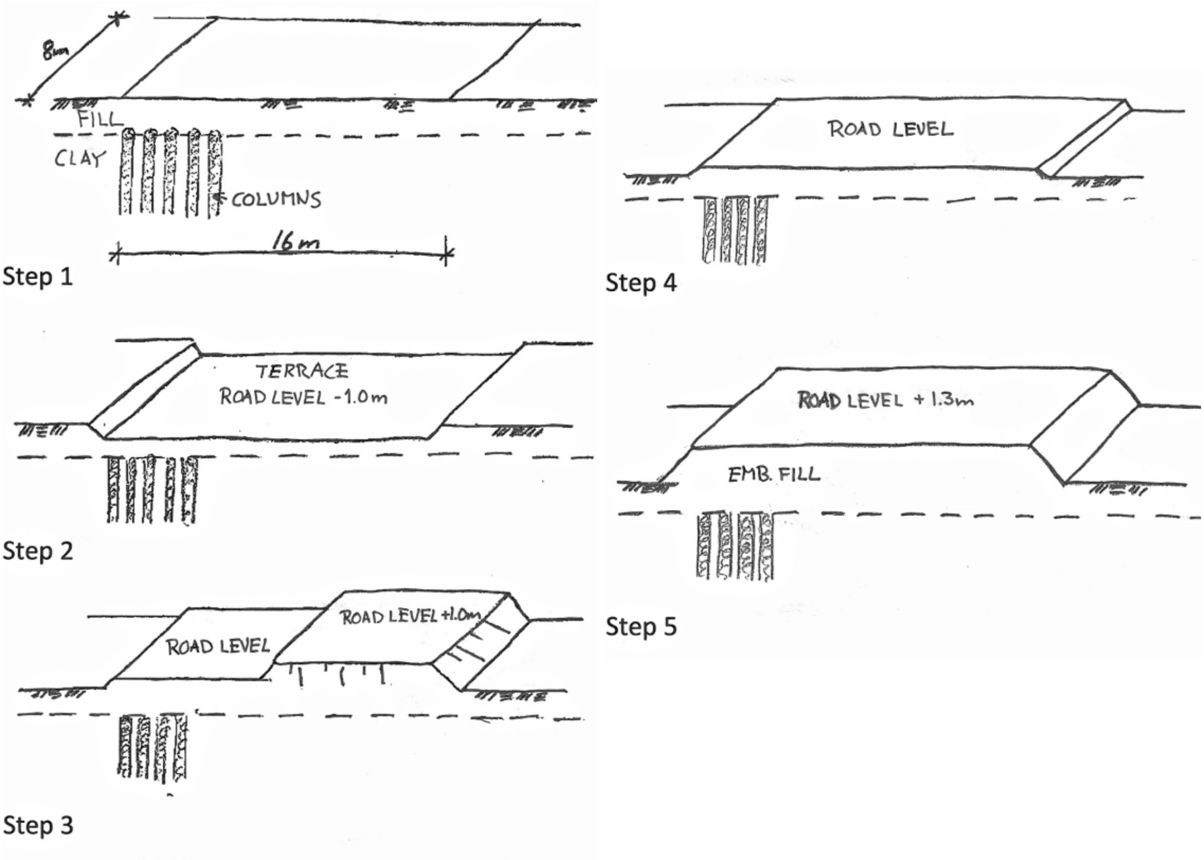

Figure 1 The construction sequences of the test embankments

Figure 2 Test embankment 1 at step 5, loaded to 1.3 m over the final road level

Results and discussion

Figure 3 shows the load sequence related to the level of the final road and the measured settlements. The settlements shown are the average of four gauges (two above columns and two in-between) for the respective surface. The settlements are shown as averages since there were no differences between the gauges with respect to the location. The settlements were thus developed roughly as plane strains.

The settlement rate was almost linear and did not decrease evenly, which was probably a result of the weather. It rained abundantly during the first two weeks of June, while the last two weeks were dry and warm and the settlement rate increased significantly. As a consequence, the settlements developed rather linearly during June. At the beginning of July it began to rain again, which resulted in a slower settlement rate. At the end of July, the weather turned dry again, which resulted in the remaining consolidation settlement. From the last week in July and for the next three weeks, the settlement rate stopped. After removing the surcharge, the ground was heaved approximately 2-4 mm. After the last on-loading, 1.3 m gravel, settlement on the preloaded surface was 4-6 mm and on the unloaded surface 6-9 mm. Over a period of two days, a heave of about 3-5 mm could be observed and after three days the embankments began to settle again. However, the settlement rate decreased rapidly. The reason for the second heave is probably the compaction of the fill material causing the pore pressure to increase.

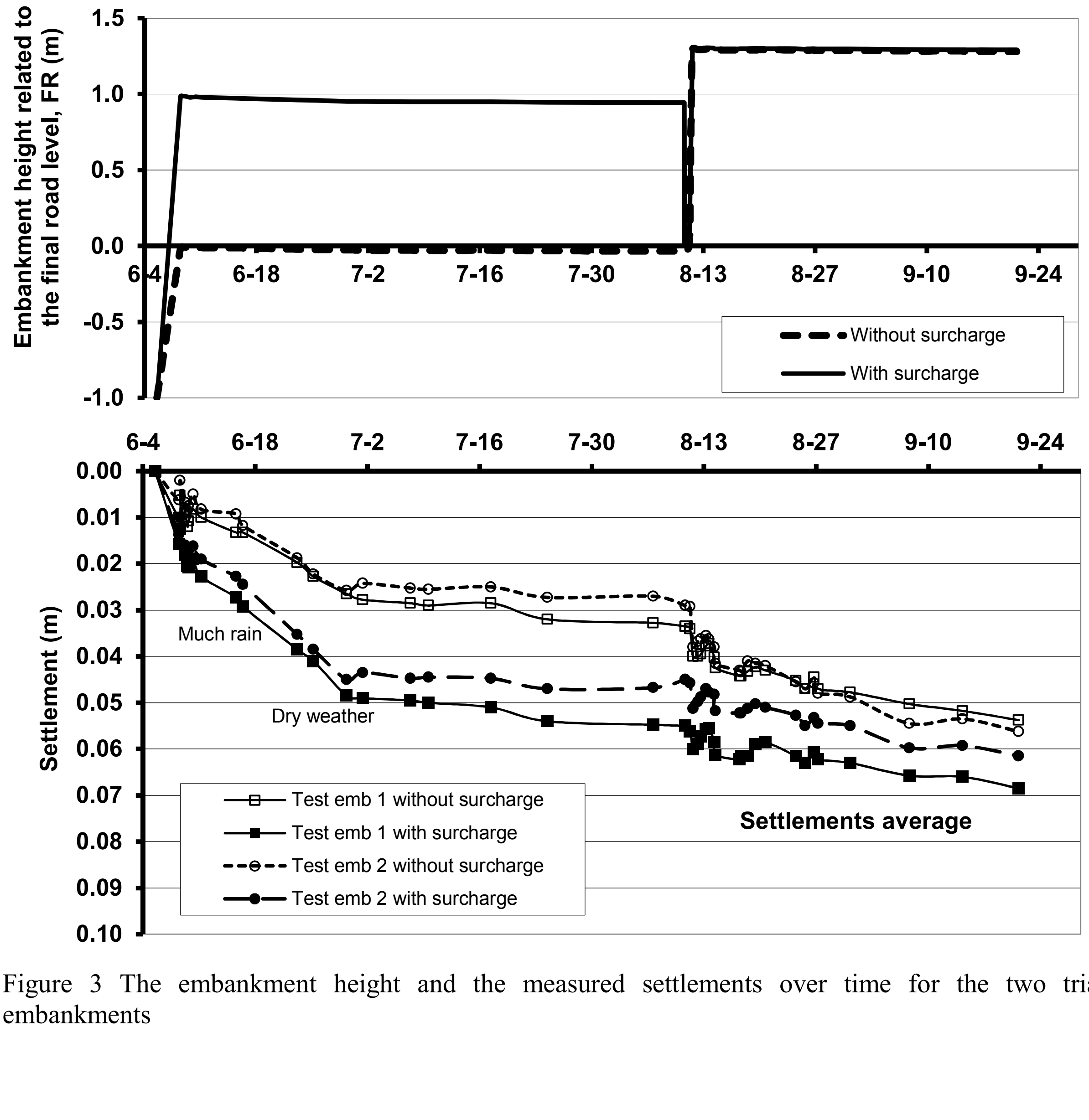

Figure 3 The embankment height and the measured settlements over time for the two trial embankments

An overall comparison between the two trial embankments showed that they behave similarly and the settlement difference was small, only 5-10 mm. A judgment concerning the influence of using a surcharge can be made by comparing the difference in total settlement with surcharge and without. It can be seen that the difference is about 20 mm and that the most of this difference is connected to the settlements that developed during the first week. If the settlements developed during the first week are excluded, the difference is only 6 mm.

The effect of the surcharge on the load of 1.3 m fill material, simulating the load of the concrete works, was relatively small. One week after loading, 6-7 mm settlements developed on the surface that had been preloaded and 10-14 mm settlements on the surface without preloading. The difference of 4-7 mm is considered insignificant in practice.

Based on the results of the load tests, the mold for the concrete decks should be pre-cambered with the corresponding magnitude of deformation plus part of the tolerance. Depending on the thickness of the clay deposit, the magnitude of the pre-cambering could easily be adjusted.

Numerical modeling

The trial tests did not render the possibility to study creep settlements since the time was too short. The influence of preloading on creep settlements was therefore studied by means of numerical modeling using the finite-element code PLAXIS (2D 2012). The dry deep mixing columns and the surrounding soft clay were modeled with the Soft Soil Creep (SSC) model (Stolle et al. 1999) and the other materials using Mohr Coulomb. The Soft Soil Creep model is considered suitable for the dry deep mixing columns. An extensive study performed by Åhnberg (2007), showed that yielding models for natural clays can also be used to describe the behavior of improved soil. The cementation process in the columns causes a quasi-preconsolidation pressure, which renders the possibility to model the columns as heavily overconsolidated soil.

The geometry model is shown in Figure 4. The engineering properties in the numerical model were adjusted to fit the load-deformation behavior according to load steps 1 to 3 in Figure 1. The adjustment was made by considering reasonable strength and deformation properties and the transformation of geometry and flow properties from the axisymmetric to a 2-dimensional case was performed according to Tan et al. (2008) and Castro & Sagaseta (2010), respectively. The parameters used for the different soil types are given in Table 1.

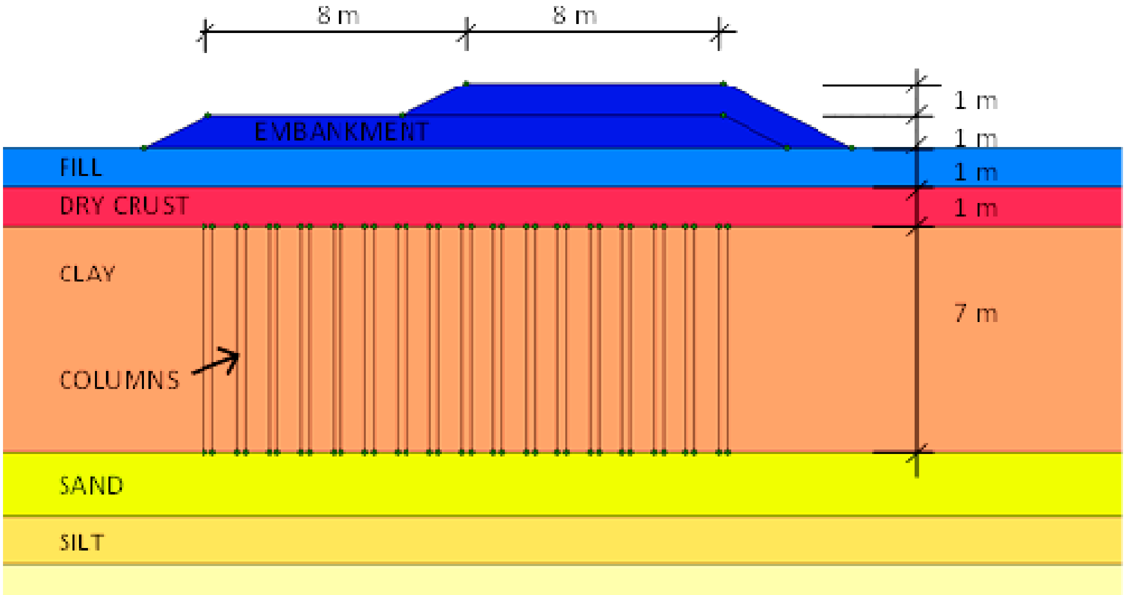

Figure 4 Geometry model of test embankment

Table 1 Input data for the FEM analyses

Material	e_{init}	γ_{unsat}	γ_{sat}	E'	M_0	M_L	ν'	c_u	c'_{ref}	ϕ'	ψ	λ^*	κ^*	μ^*	OCR	K_0	k_v	k_h
Embankment	0.5	20.0	22.0	50000	-	-	0.25	-	0.2	42	12	-	-	-	-	Def. in PLAXIS	8,64E+02	8,64E+02
Existing fill	0.5	18.0	21.0	7000	-	-	0.3	-	0.21	34	4	-	-	-	-	Def. in PLAXIS	8,64E+02	8,64E+02
Clay dry crust	0.5	19.3	20.0	-	5000	3000	0.2		20	30	-	-	-	-	-	0.80	1E- 0	1E-0
Columns	0.8		17.9	-	-	-	0.2		60	32	2	0.080	0.015	0.00024	3.0	1.6	3.5E- 03	1E- 02
Clay	1.0		17.9	-	-	-	0.3	20	2.0	30	-	0.080	0.029	0.0040	1.1	0.67	3,50E- 05	1,36E- 04
Silt	1.7		19.0	40000	-	-	0.3	-	0.1	35	5	-	-	-	-	Def. in PLAXIS	8,64E- 03	8,64E- 03
Moraine	0.5		21.0	60000	-	-	0.3	-	0.1	40	10	-	-	-	-	Def. in PLAXIS	8,64E- 02	8,64E- 02
Sand	0.5		20.0	60000	-	-	0.3	-	0.21	40	10	-	-	-	-	Def. in PLAXIS	8,64E+01	8,64E+01

e_{init} : initial void ratio (-)	γ_{unsat} : soil unit weight above phreatic level (kN/m3)	γ_{sat} : : soil unit weight below phreatic level (kN/m3)	E′: Young´s modulus (kPa)	M_0: initial oedometer modulus (kPa)	M_L: oedometer modulus (kPa)
ν′: poissons ratio (-)	c_u: undrained shear strength (kPa)	c'_{ref} : cohesion (kPa)	ϕ′: friction anle (°)	ψ: dilatancy angle (°)	λ*: modified compression index (-)
κ*: modified swelling index (-)	μ*: modified creep index (-)	OCR: over-consolidation ratio(-)	K_0: lateral earth pressure coefficient (-)	k_v: vertical hydraulic conductivity (m/day)	k_h : horizontal hydraulic conductivity (m/day)

Data on creep properties in lime and cement improved soil is limited. Åhnberg et al. (1995) conducted a number of tests but a more comprehensive study has recently been made by Vende Oliviera et al. (2013). We began with a creep index of $C_{\alpha,col} = 0.001$ and then varied it by a factor of 3 up and down. A $C_{\alpha,col}$ around 0.001 is considered to be a reasonable value for the present binder type and content according to both Åhnberg et al. (1995) and Vende Oliviera et al. (2013).

During the loading step, a plastic calculation was made where all materials were assigned as very stiff. With this procedure, the pore pressures increase without any deformations. This loading step only applies for one day, after which consolidation analysis is applied for the rest of the stages. For the unloading of the surcharge, the columns and the surrounding soft soil were modeled using Mohr Coulomb and determined as very stiff in order to avoid unrealistic swell. In the other steps, the influence of creep settlements was studied using the Soft Soil Creep model, where the creep parameters were varied.

The calculated creep settlements under the test embankment were modified with respect to the calculated creep settlements in the unloaded vertical boundaries of the model. The modification is performed since for normally consolidated clay soil modeled in Plaxis using the Soft Soil Creep, creep settlement will developed even in unloaded parts of the model.

Results and discussion

Figure 5 shows the results of the FEM analyses together with the measured settlements. According to the numerical analysis, the settlements have not stopped as indicated by the measurements. However, the adjustment was considered reasonably good relative to the purpose of the analyses. The total settlements are in the order of 7 to 8 cm over a period of 40 years.

Even though the columns have a significant effect on the deformation properties of the improved ground, the magnitude of the settlements after unloading according to the analyses is of the order of 3 cm. Part if this settlement is attributed to remaining consolidation settlement, but a significant part is attributed to creep. However, the results clearly show that the influence of the surcharge is minor and the effect is valid for all three levels of $C_{\alpha,col}$ investigated. This result is clear even though the absolute magnitude of the settlement is uncertain since the analyses are based on an adjustment of parameter and estimated creep properties.

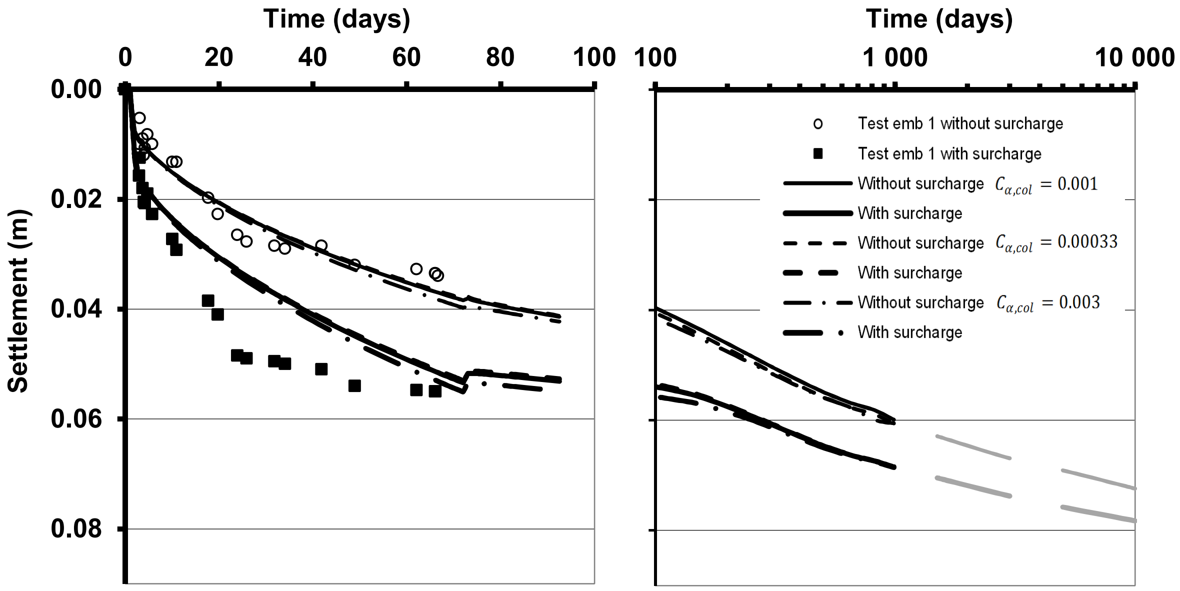

Figure 5 The embankment height and the measured settlements with time for the two trial embankments

The difference in creep settlements after a period of 100 days, as shown in the right graph in Figure 5, is less than one centimeter and therefore insignificant in practice. The creep rate is similar but one interesting observation is that the final settlements, with respect to whether a surcharge has been applied or not, approach each other somewhat over time. However, since the magnitude of the calculated settlements is not insignificant, it is of interest to investigate the interaction between creep properties in the columns and in the surrounding soft soil for different cases. Especially for low embankments these effects may have a significant influence on the requisite maximum center-to-center distance.

Excluding the surcharge, the upper part of the columns must be controlled. Under certain conditions, craters or holes may form in the dry crust (Larsson, 2003). These craters may cause redistribution of stresses that lead to large or uneven settlements. A surcharge may compensate for this phenomenon but the effect is highly uncertain. Today, the Swedish Transportation Administration normally requires that craters be filled with friction material or that the soil be excavated down to the upper part of the columns.

Conclusions

The two purposes for the contractor to perform load tests on the improved soil were fulfilled.

1. An additional load of 1.3 m fill material, corresponding to the load of the temporary concrete works, causes settlements of approximately 10 to 14 mm. The loading could be performed after one month after installation and loading of the columns corresponding to the load of the final road construction.
2. The numerical analyses indicated that a surcharge of 1 m is unnecessary provided that the improved soil is loaded with a corresponding service load for approximately 2 months before the road is completed.

Acknowledgements

The authors acknowledge Markus Lindén, production manager at Skanska Sverige AB, for his interest in simple field tests, and the Swedish Transport Administration

References

Åhnberg, H., Johansson, S-E., Retelius, A., Ljungkrantz, C., Holmqvist, L., and Holm, G., 1995. Cement and lime for deep stabilisation of soil. Swedish Geotechnical Institute, Report No. 48. (In Swedish)

Åhnberg, H., 2006. Consolidation stress effects on the strength of stabilised Swedish soils. Ground Improvement, Vol. 10, No. 1, pp. 1–13.

Åhnberg, H., 2007. On yield stresses and the influence of curing stresses on stress paths and strength measured in triaxial testing of stabilized soils. Canadian Geotechnical Journal, Vol. 44, No.1, pp. 54-66.

Castro, J., and Sagaseta, C., 2010. Discussion of "Simplified plane-strain modeling of stone-column reinforced ground" by S. A. Tan, S. Tjahyono, and K. K. Oo. American Society of Civil Engineers, ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 136, No. 6, pp. 892-894.

Larsson, S., 2003. Mixing processes for ground improvement by deep mixing. Ph.D. Thesis. KTH Royal Institute of Technology, Stockholm, Sweden, Stockholm.

Stolle, D. F. E., Vermeer, P. A., and Bonnier, P. G., 1999. Consolidation model for a creeping clay. Canadian Geotechnical Journal, Vol. 36, No.4, pp. 754–759.

Tan, S. A., Tjahyono, S., and Oo, K. K., 2008. Simplified plane-strain modeling of stone-column reinforced ground. American Society of Civil Engineers, ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 134, No.2, pp. 185-194.

Venda Oliveira, P., Correia, A., and Garcia, M., 2013. Effect of stress level and binder composition on secondary compression of an artificially stabilized soil. American Society of Civil Engineers, ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 139, No.5, pp. 810–820.