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INNOTRACK

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1. Executive summary

Many railway lines in the world are 60 to 100 years old, and are not designed in accordance with requirements for modern railway traffic. Due to the future demands for faster and heavier transports, railway structures can experience problems, such as reduced stability, increase of settlements, and possibility of extensive vibrations. These issues have an adverse effect on the safety, reliability, and economy of the railway operations. Therefore, many existing railways require upgrading of Subgrade before the opening for new traffic conditions. Engineers are faced with the task to assess the performance of structures and if necessary to design appropriate strengthening measures.

There are many methods to improve subgrade on the market. Contractors can offer various strengthening suggestions for all types of geotechnical conditions, track and subsoil geometries. In many cases decision about using a certain method depends on available machines and contract limits for each project. To find best solution and decide about feasible method for soil improvement of a new constructed railway line is generally not a problem. Much more complicated is to carry out some remedial work under existing track and under train operations. There are two possibilities. Either close the train operations and remove the track and embankment and perform strengthening, or to execute subsoil stabilization without traffic interruption.

There is a number of railway, installation and geotechnical related issues to be solved before feasible soil improvement is chosen. The first one is always identifying the cause of the problem for which a strengthening has to be performed. Basically there are three types of problems. Settlements, stability and track or environmental vibrations are the main causes that can require strengthening of subsoil. In all those cases increasing of subsoil stiffness is the main objective. It is not a method of soil improvement which is different for mitigation above mentioned problems, but placement and dimensions of strengthening that can be different from case to case. As regards an installation there are two possible ways. Installation can be performed from the track or from sides of railway embankment. Both methods have advantages and disadvantages.

In the frame of INNOTRACK project two methods for subgrade reinforcement have been tested.

Vertical columns – LCPC

The first part of this report presents the results of a feasibility study of subgrade reinforcement by vertical soil-cement columns. In-situ load tests were carried out with soil-cement columns on a test site of Northern France with four partners; KELLER FOUNDATIONS, LCPC, SNCF and SOLETANCHE BACHY. Two types of tests have been carried out:

- standard load tests have been performed on columns built in a zone located outside but close to an existing railway, near an water retaining basin. During such tests, the columns are subjected to a vertical load, increased progressively until failure (or a state near failure) ; the test results is the curve of the column head vertical displacement as the applied load increases, and the distribution of the normal force in depth along the column;

- besides, columns were built under the platform of an existing sidetrack and the vertical displacements of the column head and of the ballast as well as the distribution of the load along the column have been measured under the load brought by a train axle.

The soil-cement columns have been built by two distinct companies (KELLER FOUNDATIONS and SOLETANCHE-BACHY) on an SNCF (Société des Chemins de fer Français, French Railway administration) site, and monitored by LCPC. The main differences between the columns are their diameters (400 and 600 mm) and the tools used to achieve the soil mixing.

After the experiments, some of the columns were dug out to reveal their actual geometry, the quality of the soil mixing and to check whether or not the ballast has been polluted by the grout. Results appeared to be satisfactory. This report presents a synthesis on the field experiments; it also includes results of finite element simulations carried out to discuss the influence of various parameters.

Inclined columns – Banverket

Full scale in-situ test has been carried out using lime cement columns installed like inclined walls (panels) under existing railway embankment and without any restriction on current railway operations. The main objectives of this test were the following:

- verification of possibility to install inclined columns using machines and tools that are normally designed for vertical installations
- investigate impact of installation on operated track
- evaluate and control a quality of strengthening

The following conclusions can be drawn from the tests:

- The performed test installation of inclined columns panels has proved that this method of soil improvement is a possible alternative for increasing the stability of an existing railway embankment on originally soft subgrade.
- Installation of subgrade strengthening can be performed under an operational railway embankment.
- There is always need to take safety precautions and consider restriction for train speed or axle loads at the time of the strengthening work

2. Part 1: Vertical columns

2.1 Introduction

In order to reduce the Life Cycle Cost (LCC) of railway infrastructure, many research works deal with Track support structure. Development and implementation of several subgrade improvement methods allowing limited traffic interruption and privileged recourse to local material are one of the objectives the European research project INNOTRACK (INNOvative TRACK systems). The French National Railway company SNCF, KELLER-FOUNDATION, SOLETANCHE-BACHY and LCPC have investigated the potential benefits from the reinforcement of ground by vertical soil-cement columns. A research work based on field and laboratory tests was organized in order to:

- study the feasibility of soil-cement columns as a technique making it possible to reinforce the ground without disturbing or polluting the ballast. This technique is referred to as the lime cement column technique (or soil-mixing technique) in the report by CTU within SP2.2.1;
- evaluate the distribution of load along the column, together with shaft friction and tip resistance,
- give a basis for the validation of finite element simulations, that could in turn provide a way of analysing the influence of various factors on the efficiency of the technique.

The technique used enters the wet soil-mixing method: the soil is mixed with a cement grout (characterized by a Water/Cement (W/C) ratio). Many publications (generally linked to research works) are available on this technique as it is widely used in Sweden and Japan. We used the SGF Report 4:95 E as a guideline. Valuable information can also be found in the Innotrack report by CTU “State of the art report on soil improvement methods and experience. INNOTRACK project report, Deliverable 2.2.1”.

An experimental work was undertaken on a test site close to the High speed line station HAUTE PICARDIE on a side track. KELLER FOUNDATIONS and SOLETANCHE BACHY have designed independently two relatively similar tools to achieve soil-cement columns following the execution requirements by SNCF: the columns had to be built from the top of the railway structure within the space between sleepers. Steel tubes needed to be set up through the railway structure (ballast, subballast layer and sublayers until the subgrade layer) so as to prevent cement flooding that could pollute the railway structure. The tool used to build columns is deployed beneath the railway structure and then used to build the soil-cement columns in the subgrade layers.

The tool designed by SOLETANCHE had a minimal diameter of 168 mm (in the folded configuration used to drive the tool across the railway structure) and a maximal diameter of 400 mm in the soil mixing phase. The KELLER tool had a minimum diameter of 300 mm and a maximum diameter of 600 mm. In the railway environment, the technique is used as follows:

- a borehole is drilled out from the surface down to the top of the subgrade layers (at a depth of 1 to 1.50 m). Then, a steel tube is jacked vertically across the railway structure layers (ballast, sub-ballast, …) down to the subgrade layer; making it possible to drive the tool in the layers to be improved,
- the folded tool is then inserted through the steel tube. At the base of the tube, it begins to rotate and as it moves away from the tube, it opens up in order to perform the mixing phase of existing soil and cement grout.

The columns were then subjected to various load tests, and some of them were dug out in order to check their shape.

The soil-cement column technique is considered as an alternative to rigid inclusions and stone columns. It is generally thought that stone columns may fail to provide sufficient increase of stiffness and bearing capacity, whereas rigid inclusions tend to bring stiff elements that may be harmful to the railway structure. Soil-cement columns appear as an intermediate solution, that may provide a more homogeneously distributed load on the sub-grade layer as well as a homogeneous settlement profile. It must be recalled that the main objective was to study the feasibility of the technique in a railway environment; however, the ground of the experimental site chosen for this study had much better mechanical properties than the grounds in which soil-mixing reinforcement is usually used. This must be kept in mind for the interpretation of the results.

2.2 Test sites and geotechnical data – Soil investigation

2.2.1 Tests sites

Two test sites were chosen close to the village of Estrées-Deniécourt, near the TGV station HAUTE PICARDIE. The sites were located 500 m away from each other. The first one is close to a retaining water basin (figure 1) and the second one (figure 2) is located on a railway sidetrack at the west of the “LGV Nord” (High Speed Line of Northern France). In what follows, the sites are referred to as the “basin site” and the “sidetrack site”.

Figure 1 – View of the ”basin” site.

Figure 2 – View of the ”side track” site. The yellow dot shows the location of one of the columns.

On the basin site, load tests were undertaken to determine the bearing capacity of the soil-cement columns. On the sidetrack site, tests were carried out to study the response of the columns under the load brought by a train axle (figure 3).

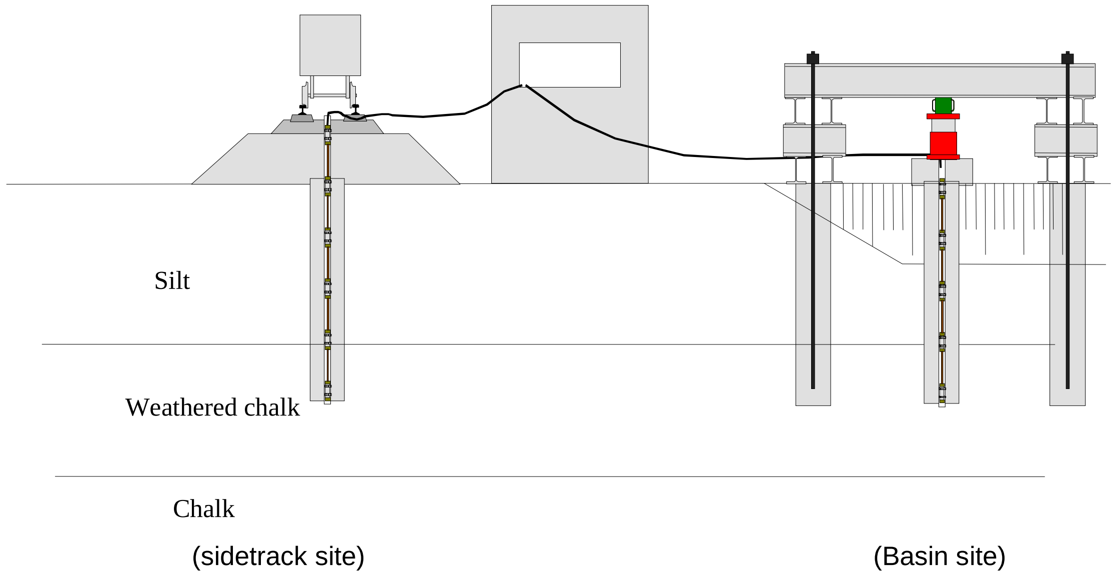


Figure 3 – Description of the experiments carried out on each site.

2.2.2 Geotechnical data

A soil investigation has been carried out by the Laboratoire Régional des Ponts et Chaussées of Saint Quentin in October 2006. Two pressure meter tests were carried out: one on the basin site, and the other in the sidetrack site. Boreholes were done with the help of a continuous flight auger, and the characterization of soils was made from the cuttings. Pressure meter tests were carried out according to ISOEN22475-4. Results are given by figure 4 and summed up hereafter.

<u>Basin site: pressure meter test PR1 – depth 12 m</u>

A 12 m long borehole was drilled. No water table was found. The site investigation showed the following layers:

- 0 - 1.20 m : dark silt layer characterised by a limit pressure p_l around 0.54 MPa (1 value)
- 1.20 m - 5 m : clayey sandy silt with chalk particles; with a limit pressure p_l encompassed between 0.27 MPa ≤ p_l ≤ 1.31 MPa ,
- 5 m - 6.5 m : a silty chalk layer with a limit pressure p_l ≥ 2.94 MPa (1 value),
- 6.5 m - 12 m : a chalk layer with a limit pressure p_l ≥ 2.44 MPa ,

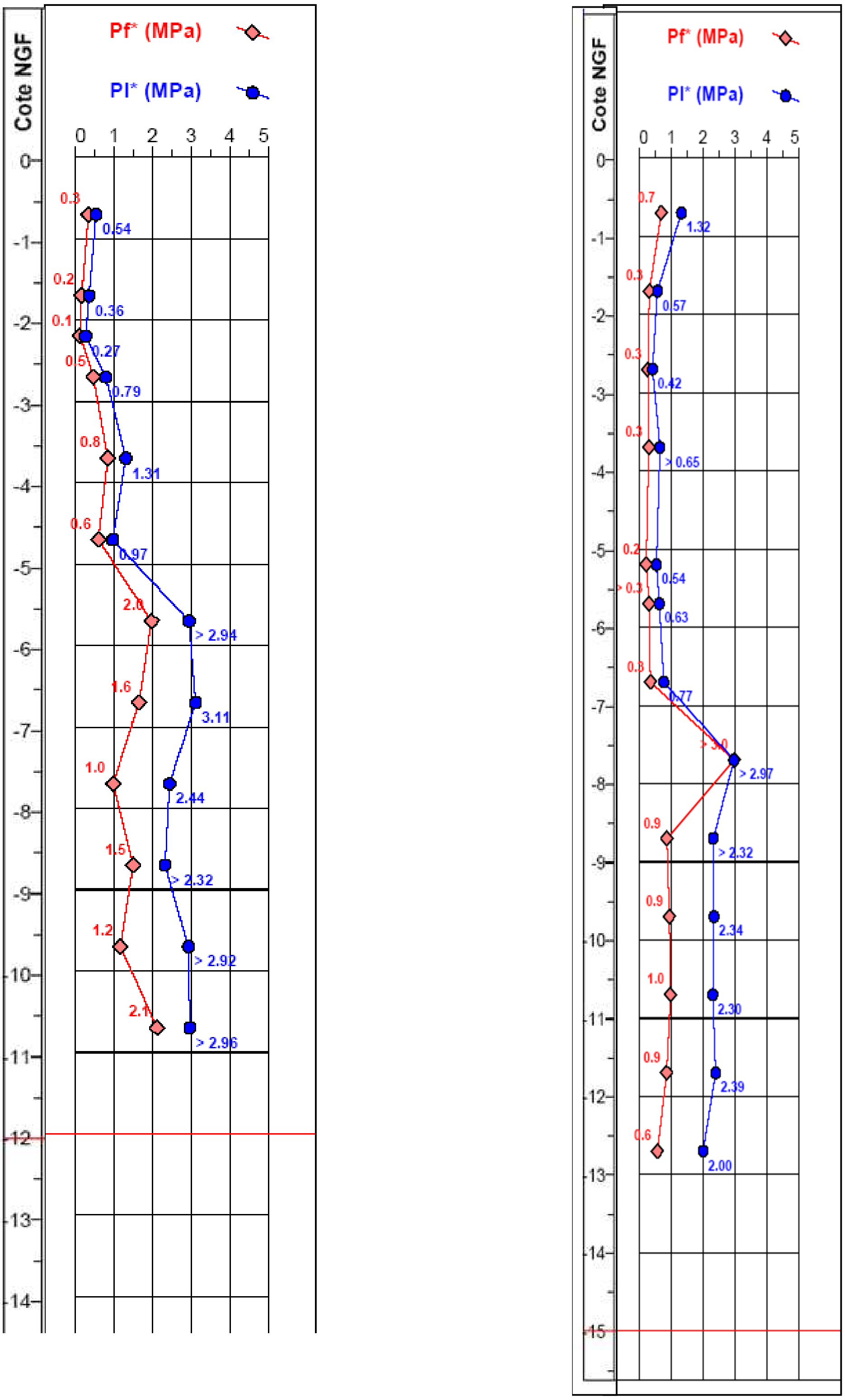
Water contents were about 20%.

<u>Side track site: pressumeter PR2 – depth 13 m:</u>

The following layers were found (from the lower side of the sleeper):

- 0 - 0.10 m: ballast;
- 0.10 m - 0.30 m: fill,
- 0.30 m - 5 m : sandy silt with 0.42 MPa ≤ p_l ≤ 0.65 MPa (1.32 MPa at the upper surface),
- 5 m - 7 m : silty chalk with 0.54 MPa ≤ p_l ≤ 0.77 MPa,
- From 7 m to 13 m: chalk with p_l ≥ 2 MPa.

Samplers have also been used to carry out laboratory tests. The identification from samplers showed similar layers.



(a) PR1 (b) PR2

Figure 4 – Pressure meter profiles (Pf* = creep pressure; Pl* = limit pressure)

2.3 Columns construction

The set up of columns was done through wet soil mixing method i.e. the soil was mixed with cement grout. A specific device has been used in order to achieve a homogeneous soil-cement mixing. The cement used was a CEM III/C 32.5 N PM-ES.

2.3.1 Basin site - KELLER columns

Two columns were built with the Keller tool (figures 5 and 6), outside the area of the track, in the basin site. There is no ballast or sub-layer, and the diameter of the column is constant (in its open configuration, the tool makes it possible to build 600 mm-diameter columns).

The construction procedure can be described as follows: the mixing tool, in open configuration, is jacked down and the soil is sheared and mixed with cement. From the top to the base of the column, both rotation and injection take place; it is the same in the second phase when the tool gets back to the surface.

Besides, soil cement columns were to be tested using the LCPC extensometer technique (Bustamante and Gianeselli, 1996 and 2001). A closed steel tube was installed in each column on the day of construction, before the cement set up. After a few days, a pile head was built on the column head, with steel reinforcement to ensure a good connection with the column, so that a vertical load could be applied.

Each column was built in about one hour. Columns P1 and P2 were built respectively on the 30th and 31st Oct. 2006 by KELLER. The anchors required for the reaction frame were built on 31st October 2006.

Samples 160 mm in diameter and 320 mm in length were collected at the head of the test column to perform unconfined compression tests.

The tip of the columns was located at an intermediate level between the silt and the good chalk layer (figure 12).

The physical properties of the columns are described in table I.

Table I – Characteristics of the columns

Column n°	Diameter (mm)	Length (m)	W/C	Density* kg/m^3
P1	600	5.30	1	338
P2	600	5.30	0.83	397

* estimated mass of cement in one m^3 of soil-cement column

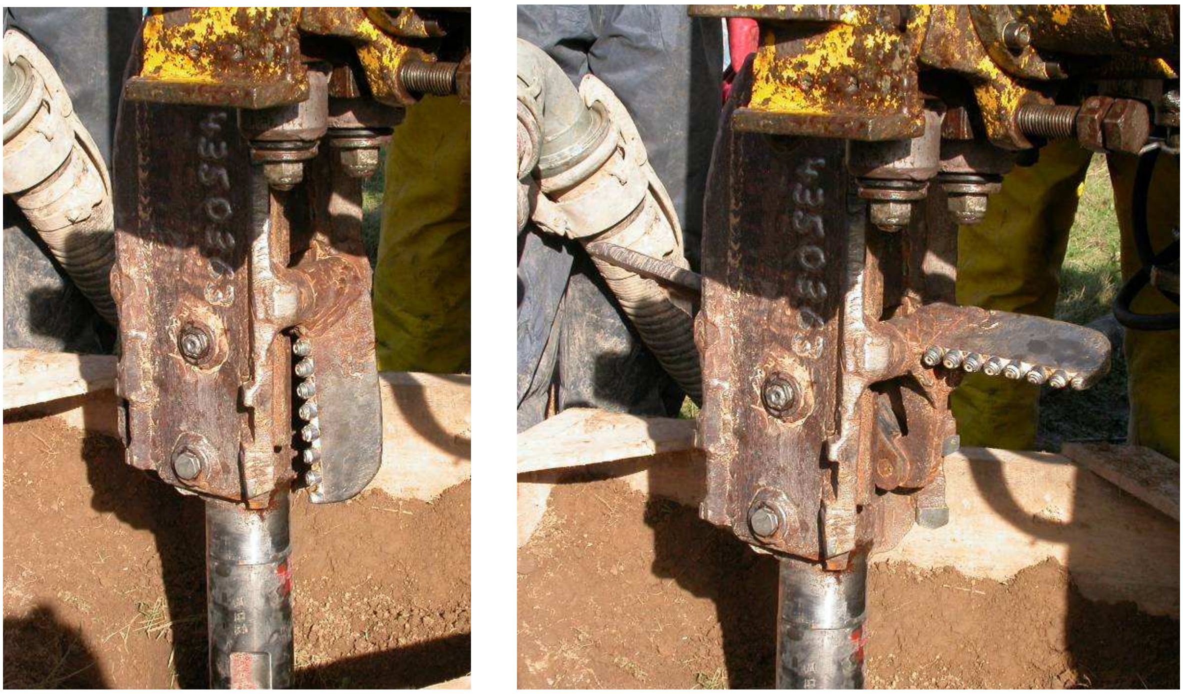


Figure 5 – Folded and open configurations of the Keller tool.

Figure 6 – Picture of the tool after the end of the mixing phase.

2.3.2 Basin site - SOLETANCHE BACHY columns

A test column was built on September 21st, 2006 using the new tool developed by SOLETANCHE BACHY, in the “basin site”. The tool mixes the existing soil with grout by means of two blades (mechanically retractable). The maximum diameter of the tool in the open configuration is 400 mm (figures 7 and 8). In its folded configuration, the diameter is 168 mm.

Figure 7 – Folded and open configurations of the Solétanche tool.

Figure 8 – Maximum wingspan of the open tool.

The column built in the basin site was to be used to perform a vertical static load test. The construction steps were the following:

- the tool was open, in order to get a 400 mm diameter soil-cement column. From top to the base of the column, the head was in rotation and injection was performed; during the return of the tool towards the surface, the head was in rotation in order to get a good homogenization of the soil-cement mix;
- 24 hours later, on Sept 22nd, the column was drilled with a 80 mm tricone, in order to place a closed steel tube, that was sealed to the column with grout; this tube is used to measure vertical strain along the column during the static load test, using the LCPC extensometer technique;
- after a few days (on Sept 26th and Sept 27th), a pile head was added to the column, with steel reinforcement to ensure a good connection between the head and the column. The head was about 420 mm in diameter and 390 mm high.

The loaded column was 6.24 m long. The chalk appeared at a depth of 4.80 m. The W/C ratio was about 1. The recorded data gave a grout quantity of about 50 l/m.

The tube length was 5.89 m with 5.30 m within the soil.

A loading frame with four anchors was necessary to measure the bearing capacity of the soil-cement column. The anchors were drilled on September 22nd down to a depth of 6 m. A 22 mm diameter steel bar was inserted in each anchor.

2.3.3 Sidetrack site – SOLETANCHE BACHY columns

Six columns were drilled through a side track between Sept 28th and Oct 2nd, 2006 (figure 9). One of the columns was placed between the rails, at some distance from the other five, that formed a pattern of 5 columns shown in figure 10.

Figure 9 – Drilling of the columns under the track

Two of the columns were instrumented, the single column and the column at the center of the group. To perform extensometer measurements, these columns (C12 and C15) were equipped with a steel tube, and the procedure was a little different for them.

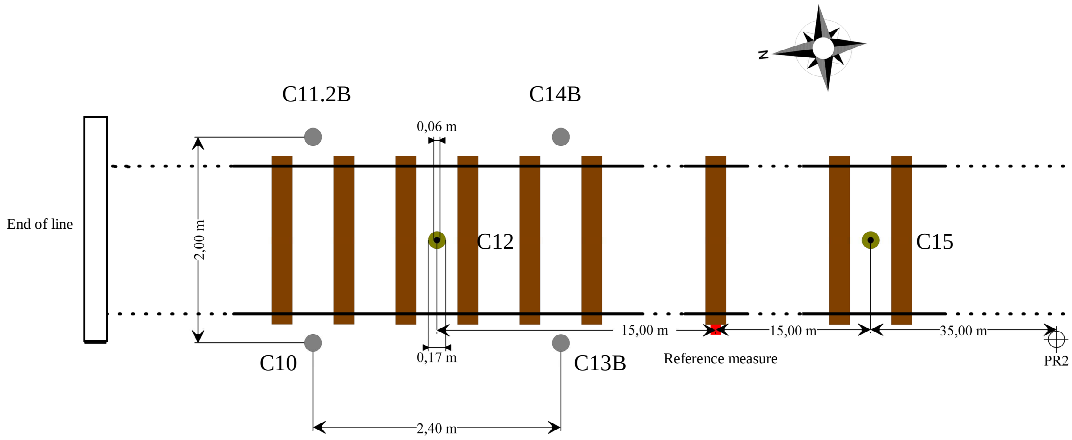


Figure 10 – General layout of the tests columns on the side track site

The non instrumented columns were built according to the following procedure:

- set up of a ∅ 170 mm steel tube crossing the railway platform down to the level where the blades of the drilling rotary head were to be open, at a depth of about 1.50-1.75 m.
- drilling of a ∅ 168 mm hole with a tricone down to a depth of 7 m,
- mixing of soil with cement to get a 400 mm column from 1.50-1.75 m to 7 m,
- withdrawal of the 170 mm steel tube, and filling up of the hole with grout (from the bottom of the sublayer to the bottom of the ballast) and then with ballast.

For the two instrumented columns, the procedure was the following:

- set up of a ∅ 170 mm steel tube which would cross the railway platform and allowing the opening up of the blades of the drilling rotary head at a depth of embedment of about 1.50-1.75 m.
- drilling of a ∅ 168 mm hole with a tricone down to a depth of 7 m,
- mixing of soil with cement to get a 400 mm column from 1.50-1.75 m to 7 m
- drilling with a 80 mm tricone and introduction in the column of a closed ended 52/60 steel tube for the extensometer, sealed with grout,
- withdrawal of the 170 mm steel tube, and filling up of the hole with grout (from the bottom of the sublayer to the bottom of the ballast) and then with ballast.

The columns characteristics were as described in table II.

Table II – Diameters and lengths of the columns

Column n°	Diameter (mm)	Length (m)
C10	400	6.98
C11.2B	400	6.56
C12	400	6.68
C13B	400	7.02
C14B	400	7.10
C15	400	6.16

2.4 Vertical static load tests (basin site)

2.4.1 Monitoring system

As mentioned before, some of the columns were prepared so that the vertical strain could be measured using the LCPC removable extensometer (Bustamante and Gianeselli, 1996 and 2001).

The technique consists in installing a ∅52/60 mm diameter closed ended steel tube in the middle of the column just after the soil-cement column is set up, then placing in the steel tube a set of ribbons separated by inflatable packers. Once the packers are inflated, they follow the displacement of the steel tube. Steel ribbons are equipped with strain gauges (figure 12), making it possible to measure the vertical strain for each level located between two packers.

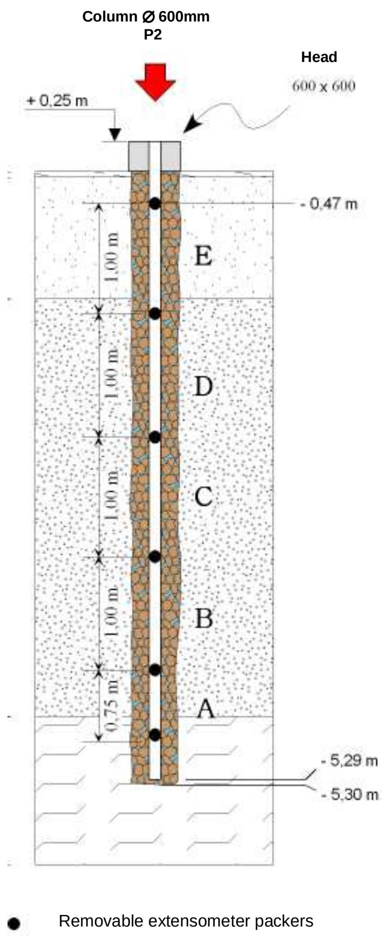
A reaction frame is necessary to apply the load on the columns. In the case of the KELLER columns, it consisted in a steel beam laying on wood beams and linked to four vertical anchor bars DYWIDAG ∅ 22 mm connected to four reaction piles ∅ 200 mm (figure 11), that work in tension when the column is tested in compression. The limit traction force of the DYWIDAG bars was 2 MN. A similar system was used for the tests on the Solétanche Bachy column.

Figure 11 – View of the load test system

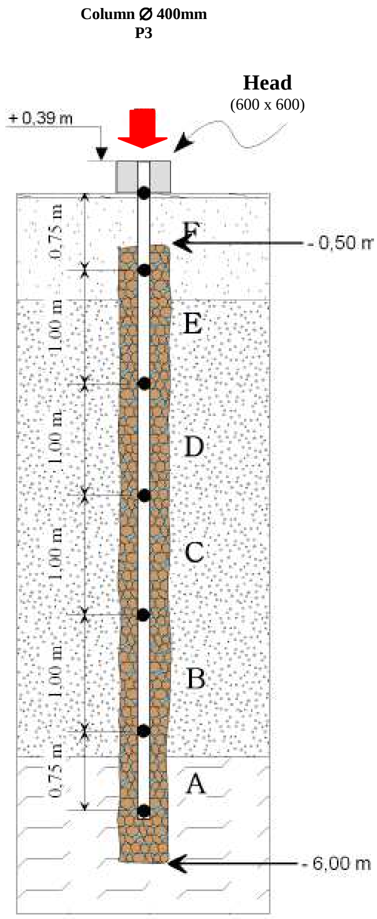
The three columns built in the basin site have been equipped with a concrete head in order to perform static load tests in good conditions.

For the 600 mm Keller columns, the head was a 600-mm square concrete block, with a thickness of 250 mm. The top of the soil-mix column reached the top of the column head.

For the 400 mm Solétanche Bachy column, the head was a 420 mm diameter circular concrete block, with a thickness of 390 mm. Because of a misunderstanding between teams involved, the top of the soil-mix column did not reach the column head (figure 12), which had some influence on the results of the vertical load test.



Keller columns P1/P2



Solétanche Bachy column P3

Figure 12 – Removable extensometer in the soil cement columns: positions of the packers.

During the static load tests, the vertical displacement of the column head is measured by means of four displacement transducers with a precision of 0.01 mm. Five strain gauges inside the column (figure 12), associated with appropriate data acquisition and storage devices, are used to measure the vertical strains at the different levels.

The columns were loaded following the standard method of the Laboratoires des Ponts et Chaussées. The load is applied in equal increments (each load was applied during one hour) until failure. The standard method leaves it to the staff in charge of the tests to choose the load increment, in view of the expected limit load and of the number of points needed on the curves.

Table III recalls the days of construction and test of columns, and gives the number of days between them. After the load test, column P2 was excavated.

Table III – Construction of the columns in the basin site

Columns	Set up date	Load test date	Days between construction and load test
P1 (south) – Keller	31 October 2006	6 December 2006	36
P2 (north) – Keller	30 October 2006	7 December 2006	38
P3 - Solétanche	22 September 2006	24 October 2006	32

2.4.2 Results of the static load tests

KELLER columns P1/P2

Two values of the W/C ratio were obtained: 0.83 for column P1 and 1 for column P2.

Figure 13 shows the vertical load Q_0 vs. the vertical displacement S_0 (mm) of the column head.

For column P1, only six load increments of 50 kN could be applied. The maximum load was 300 kN, and the corresponding settlement was equal to 5.8 mm.

For column P2, ten load increments of 50 kN were applied. The maximum applied load was equal to 450 kN and the settlement for that load was equal to 7.2 mm.

The bearing capacity of a pile is conventionally defined as the load for which the settlement is equal to 1/10th of the pile diameter. The bearing capacity of the soil-cement columns was higher than expected and the final settlements are far smaller the conventional limit. It was therefore necessary to perform an extrapolation using the Chin method (Tomlinson, 1995) to evaluate the bearing capacity as well as the distribution of the resistance along the column i.e. the maximum shaft friction and the maximum tip resistance.

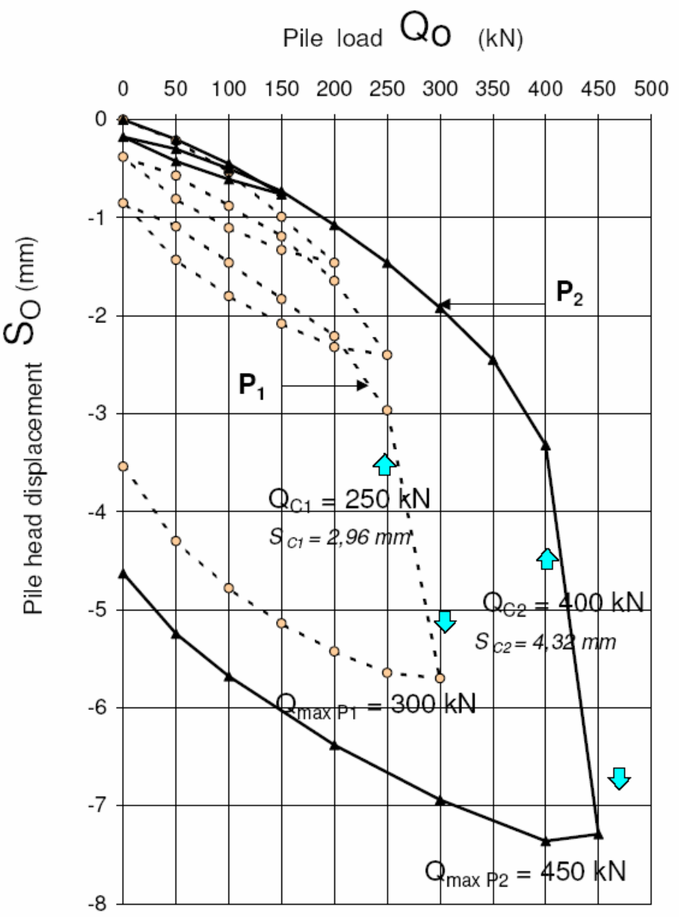


Figure 13 – Load-settlement curves for columns P1 and P2.

Column P1

The load test occurs after 36 setting days.

The maximum load applied was 300 kN for a settlement of 5.8 mm (about 1% of the pile diameter, whereas the bearing capacity corresponds to a displacement of 10% of the diameter). The creep load reached Q_c = 250 kN. (Note: it is recalled that the creep load is the load for which delayed settlement become significant. As shown by figure 15, each load increment is applied for a definite time interval (generally 30 or 60 minutes, but the same value must be kept for all increments), which makes it possible to check that delayed settlement takes place or not under a constant load).

The load distribution along the columns was determined from the measured vertical strains ε. Young's modulus was estimated from the results of R_c laboratory tests from excavated lumps of soil-mixed materials.

Figure 14 shows that a large part of the load is transferred to the ground through friction along the segment A. The contribution of shaft friction to the maximum load of 300 kN was equal to Q_s = 258 kN (or 86% of the total load) whereas the contribution of the tip was equal to Q_p = 42 kN (14% of the total load).

Using the Chin method (Tomlinson, 1995), it was estimated that the bearing capacity of the soil cement column was equal to 417 kN, with a shaft friction of 266 kN (64%) and a tip resistance of 151 kN (36 % of the total load). This shows that the shaft friction had been fully mobilized for a settlement of 5.8 mm whereas tip resistance required more settlement to be fully mobilized.

According to the Chin method, the tip resistance stress could reach about 0.5 MPa (151 kN/(πx0.3^2)) which is smaller than what could be expected on the basis of former pile analysis, given the pressure meter results (limit pressures of 0.97 MPa at 4.8 m and > 2.94 MPa at 5.8 m). This may reflect disturbance of the soil by the mixing process.

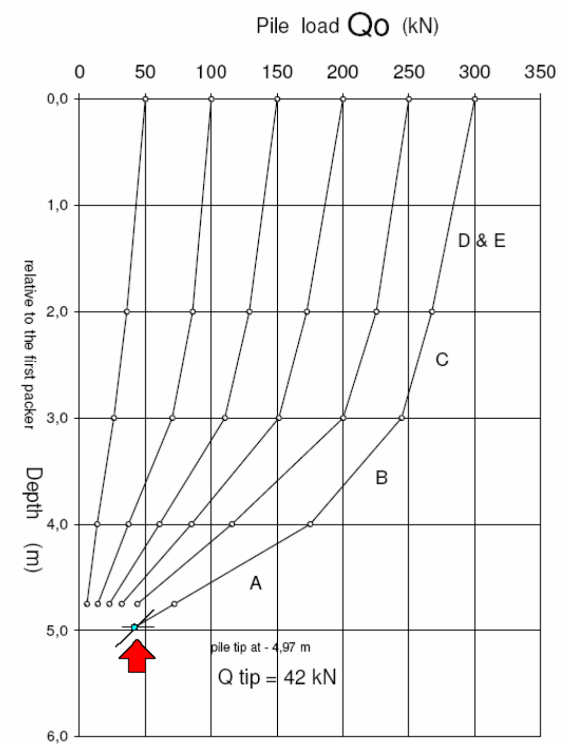


Figure 14 – Load distribution along column P1.

The load distribution along the column makes it possible to calculate the shaft friction for each of the levels (A, B, C, D and E) as the pile head displacement increases (figure 16). The load transferred to the ground through levels A and B was much larger than at other levels due to the better mechanical properties of the ground at that depth. Values of friction q_s are summed up in table III (levels D and E are grouped due to experimental reasons).

Table IV – Shaft friction calculated from vertical unit strains

Level	A	B	C	D-E
Friction q_s (kPa)	72	37	13	13

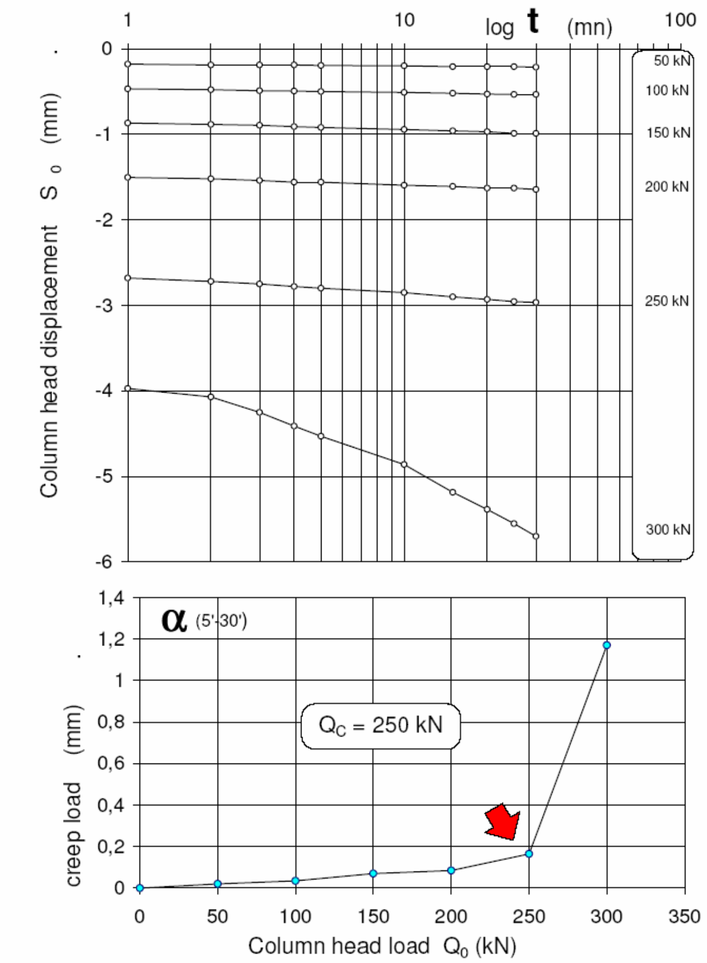


Figure 15 – Determination of the creep load for column P1.

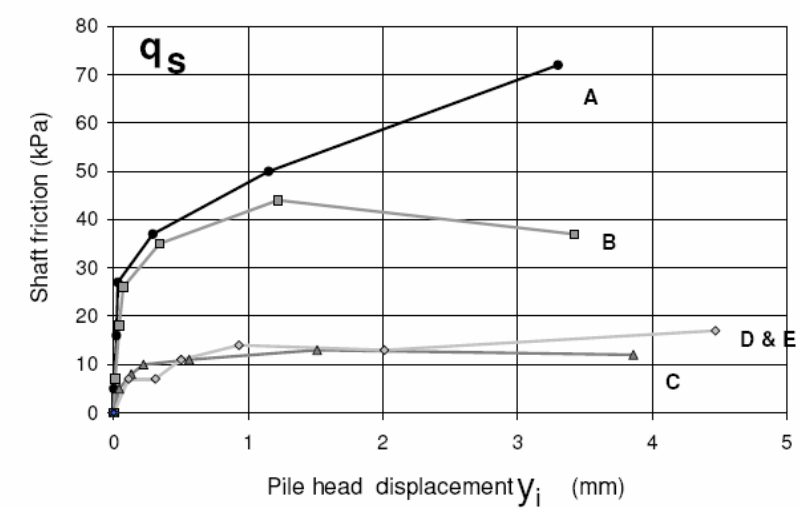


Figure 16 –Mobilization of shaft friction for the different load steps (column P1).

Column P2

The load test occurs after 38 setting days.

The maximum load applied was 450 kN for a settlement of 7.2 mm (about 1% of the pile diameter). The creep load reached Q_c = 400 kN.

The load distribution along the columns was determined from the measured strains ε. Young's modulus was estimated from the results of R_c laboratory tests from excavated lumps of soil-mixed materials.

Figure 17 shows that, also for column P2, a large part of the load is transferred to the ground through friction along the segment A. The contribution of shaft friction to the maximum load of 450 kN was equal to Q_s = 279 kN (62% of the total load) whereas the contribution of the tip was equal to Q_p = 171 kN (38% of the total load).

Extrapolation of the results using the Chin method (Tomlinson, 1995) gave a bearing capacity of 811 kN, with a shaft friction of 570 kN (70%) and a tip resistance of 241 kN (30 % of the total load). Contrary to what was observed for column P1, shaft friction had not been mobilized for the last load increment. Tip resistance also required more settlement to be fully mobilized.

Figure 17 also shows that, at load step 9 (400 kN), the steel tube slid on the column. Therefore, from this point, we were not able to measure the distribution of load along the column, and we were only able to analyse data from the extensometer for loads below 350 kN (figure 18).

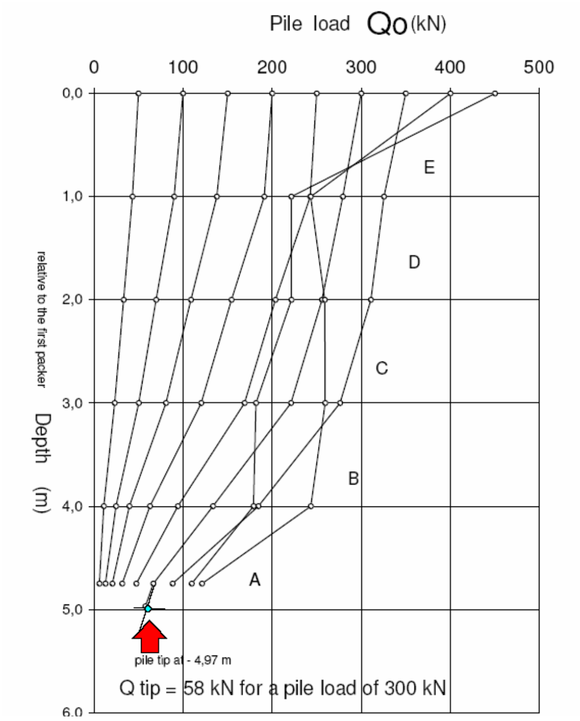


Figure 17 – Load distribution along column P1 (For pile loads larger than 350 kN, the conversion of vertical strains to loads in the column is no longer valid, because the tube slid on the column).

The load distribution along the column makes it possible to calculate the shaft friction for each of the levels (A, B, C, D and E) as the pile head displacement increases (figure 16). The load transferred to the ground through levels A and B was much larger than the other levels due to the better mechanical properties of the ground at that depth. Values of friction q_s, for a load of 300 kN on column P2, are summed up in table V.

Table V – Shaft friction calculated from vertical strains (column P2 / load of 300 kN)

Level	A	B	C	D	E
Friction q_s (kPa)	47	45	18	20	5

For both P1 and P2, the distribution of load looked similar, since about 70 % of the load was taken from the shaft and 30 % from the tip. However, the difference in terms of bearing capacity could not only be explained by the W/C ratio and the cement contents (338 and 397 kg/m^3 respectively). The actual distribution of cement along the column could be an explanation for this discrepancy.

In addition, for both columns, the shaft friction was fully mobilized at the levels E, D, C and B with measured values greater in the case of the column P2. At level A, the skin friction was not completely mobilized in both cases (results show that the friction had not reached a maximum value for the maximum applied load).

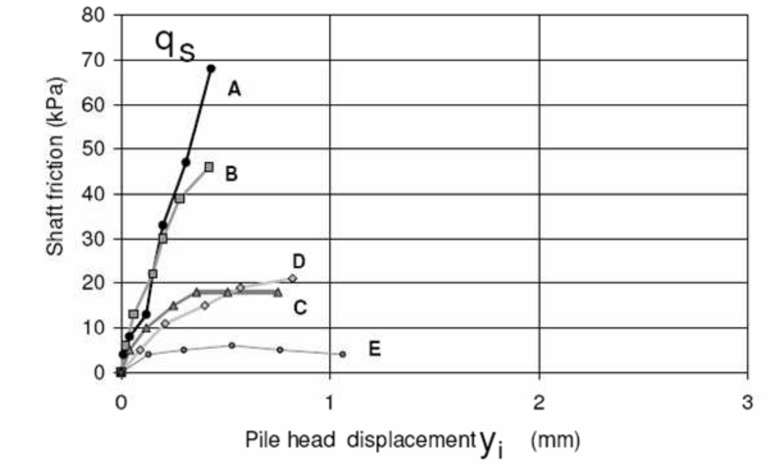


Figure 18 – Mobilization of shaft friction for the different load steps (column P2).

Column P3 (Solétanche Bachy)

In this paragraph, we present the results of the tests carried out on the soil-cement column built by SOLETANCHE-BACHY. Figure 20 shows the load / head settlement curve.

The bearing capacity could be reached, since the settlement head was equal to 90 mm (thus greater than 10% of the column diameter) for a load of 325 kN. A rapid increase of the settlement was observed.

The analysis of the load distribution was performed on the basis of the strains ε. Figure 20 shows that, for column P3, a large part of the load is transferred to the ground through friction along segments C and D. The contribution of shaft friction to the maximum load of 325 kN was equal to Q_s = 305 kN (94% of the total load) whereas the contribution of the tip was equal to Q_p = 20 kN (6% of the total load). Figure 21 shows the distribution of the load along the column and the mobilization curves of the shaft friction (q_s).

Values of friction q_s for a load of 325 kN are summed up in table VI.

Table VI – Shaft friction calculated from vertical unit strains (column P2 / load of 300 kN)

Level	A	B	C	D	E	F
Friction q_s (kPa)	30	30	75	75	not analyzed	not analyzed

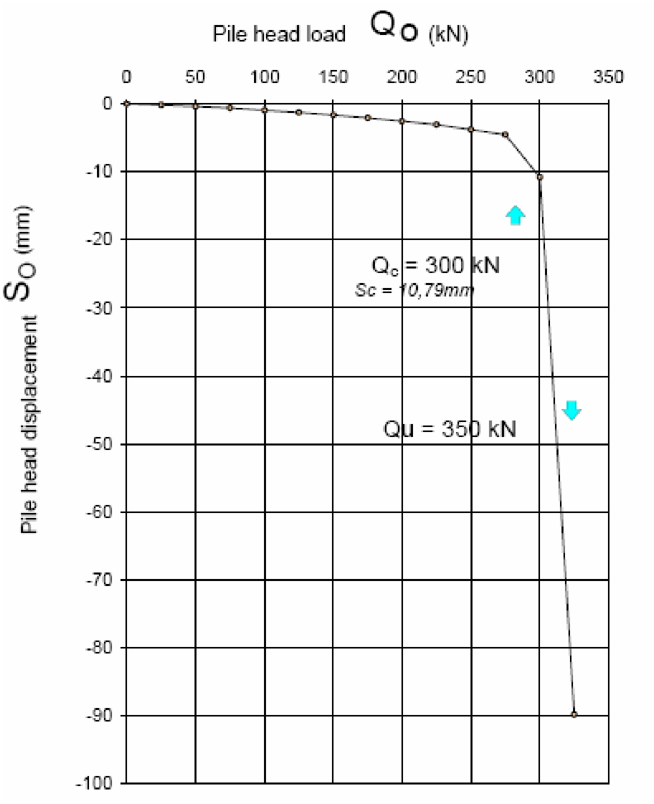


Figure 19 – Load-settlement curve (column P3).

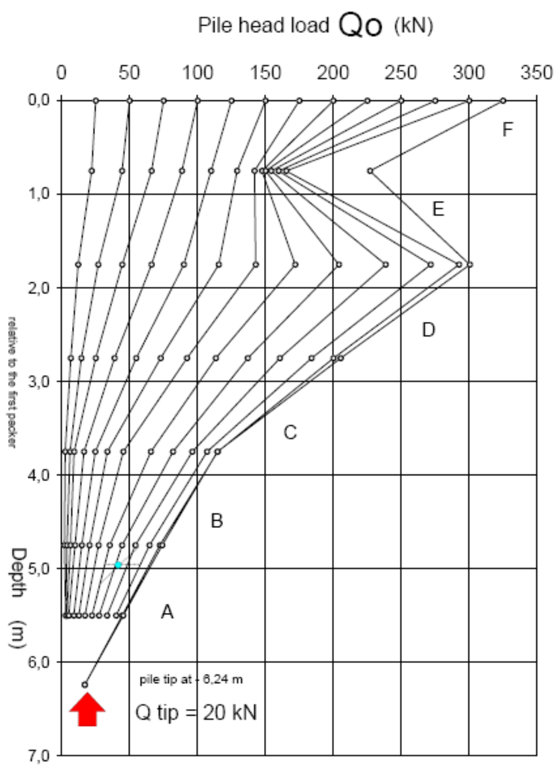


Figure 20 – Load distribution along column P3.

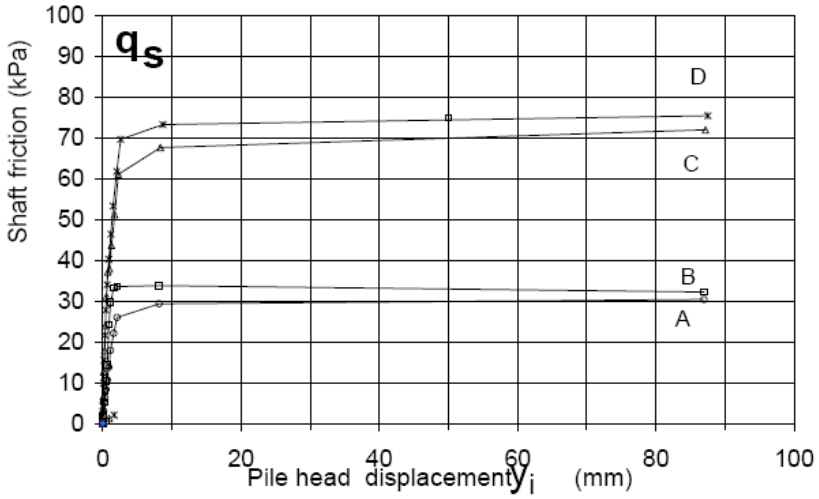


Figure 21 – Mobilization of shaft friction for the different load steps (column P3).

2.4.3 Excavation of columns

Keller columns

Column P2 and another similar column (not subjected to a load test) were excavated (figure 22). The shape was smooth and cylinder shaped. The measured diameter of the test column was about 640 mm (instead of the theoretical value of 600 mm). The section of the other column shows, in the centre, a zone with a higher density of cement, with a diameter of about 300 mm, apparently due to the soil-mixing method. A significant heterogeneity appears between the center of the column and the outer crown.

Figure 22 – Excavation of column P2 (left) and view of the section of a similar (not loaded) column (right)

Lumps were taken in the upper part (top 2-3 m) of the soil-cement columns to perform laboratory tests.

Solétanche Bachy columns

On the February 7-8, 2007, columns have been excavated (figure 23). On the basin site, a test column and the loaded column were dug out. The loaded column had a perimeter of about 1300 mm and the test column had a perimeter of about 1400 mm; the theoretical value was 1256 mm.


Loaded column (basin zone)


piece of the loaded column


test column (basin zone)

Figure 23 – Excavation of columns built with the Solétanche Bachy tool.

2.5 Behaviour under service load: side track site

2.5.1 Instrumentation of columns – static load

Two columns have been equipped with the extensometer system used for the static load tests described above. Measures of vertical strains of the columns have been carried out with the help of removable extensometers for three levels within the close ended steel tube (figure 24).

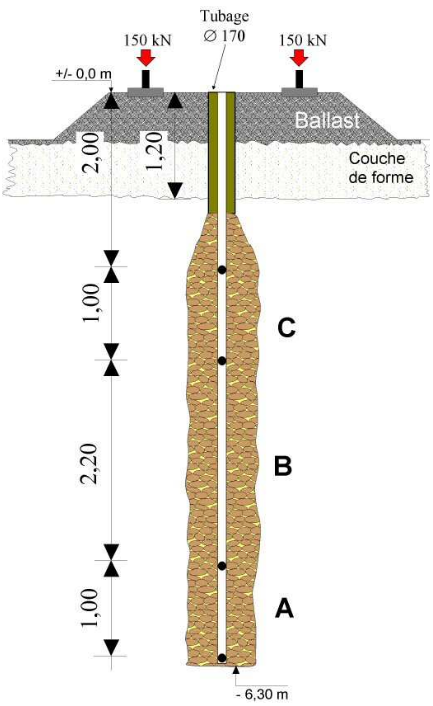


Figure 24 – Removable extensometer layout in the instrumented column.

The instrumented columns were the single column and the central column in the group of five columns (figure 10). The vertical displacement of the column head was measured by means of displacement transducers, with a precision of 0.01 mm, located on fixed reference points (figure 25).

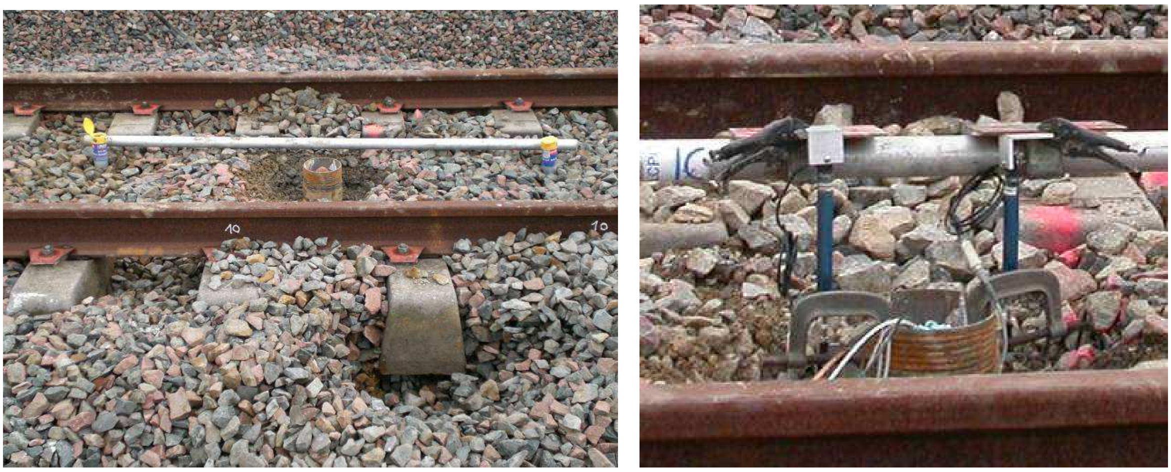


Figure 25 – Head of the column equipped with strain gauges (left); displacement transducers (right).

The load, is brought by one axle of a maintenance train, was about 300 kN (figure 26).

Figure 26 – View of the axle load applied by the maintenance train.

In the case of the single column, the vertical settlement of the head was measured by two sensors, and another sensor was placed on the closest sleeper. The same monitoring system was used for the central column of the 5-columns group. In addition, total stress sensors were positioned under the sleepers equipped with vertical strain gauges.

The loading program was divided into two phases: in the first phase, the axle load was applied on the single column during two hours. In the second phase, the axle was located above the central column of the 5-column group during 90 minutes.

2.5.2 Results of the column and track instrumentation

As far as stress/loads measurements, we can notice similar trends in the distribution of loads between the single column and the column within a group (figure 27).

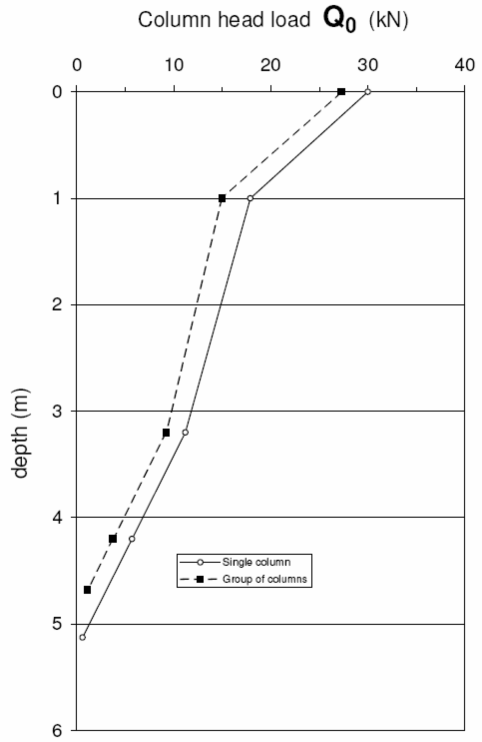


Figure 27 – Distribution of loads along the column. Comparison between the single column and the group of columns.

For the tip resistance, we did not observe a relevant evolution as it increases from 0.61 kN to 1.11 kN. Skin friction shows a small decrease probably due to the redistribution of loads along the five columns in the group case. Another explanation could be that the single column was 40 cm longer than the column in the centre of the group. The main results for each case are presented in the following paragraphs.

Single column

Measurement of the stress under the sleepers. These measurements did not give acceptable measures. We could not use these values in our analysis.

Measurement of the settlement at the level of the ballast: 0,89 mm (in 2 hours)

Measurement of the settlement at the level of the column: 0,09 mm

A ratio of ten between the sleeper and the column settlement is noticed.

Interpretation of the measures of shaft friction is given in table VII.

Table VII – Single column - experimental results

Single column	Measurements
Sleeper settlement (mm)	0,89
Column settlement (mm)	0,09
q_s (kPa) - Level C	10
q_s (kPa) – Level B	3,2
q_s (kPa) – Level A	4,3
Tip resistance (kN)	0,61

Column within a group

Measurements of the stress under the sleepers did not give acceptable measures. We could not use these values in our analysis.

Measurement of the settlement at the level of the ballast: 0,76mm,

Measurement of the settlement at the level of the column: 0,055mm

Again, a ratio of ten between the sleeper and the column settlement is noticed.

Interpretation of shaft friction measures is given in table VIII.

Table VIII – Group of columns- experimental results

Central column	Measurements
Sleeper settlement (mm)	0,76
Column settlement (mm)	0,06
q_s (kPa) - Level C	9.9
q_s (kPa) – Level B	2.3
q_s (kPa) – Level A	4,3
Tip resistance (kN)	1.11

The results obtained on the single column and the group of columns show a settlement of the column about ten times less than the ballast settlement.

The differences in terms of settlement in the two cases were quite small; less than 1mm. As mentioned in the literature (for example Al Shaer, 2005), the ballast settlement represented about 90% of the total settlement.

2.5.3 Excavation of columns

On the side track site, the single column was excavated (figure 28). Two sleepers were put aside so that the engine could dig around the column.

Figure 28 – Excavation of one column on the side track site.

An important result was that some grout was stuck to the steel tube driven through the ballast, but the grout did not spread within the ballast layer and the under layers.

We can see the visible part of the tube (ϕ168 mm) used to make the tool go down to the chosen depth. The thin grout layer around the tube shows a viscous grout which did not spread up. The different layers of the platform can be identified: the ballast layer, the treated silt, stone cement layer. The soil-cement column begins a few centimetres below the fill layer. Between 1350 mm and 1750 mm in depth, the diameter of the column increases as the tool needed a certain length to deploy before reaching the maximum 400 mm diameter. In other words, the column shows a conical shape in its upper part.

Figure 29 shows the observed variations of the column diameter with depth, below the steel tube driven through the ballast for the column construction phase.

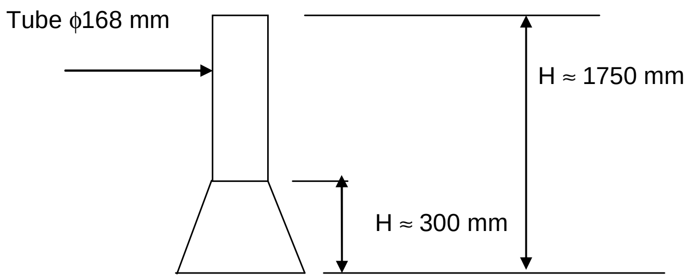


Figure 29 – Shape of the column in the upper part of the reinforced layer.

2.6 Finite element parametric study

In addition to the field tests in the basin site and in the side track site, numerical simulations have been undertaken in order to investigate the limits and the possible benefits of the reinforcement technique, and to discuss the possible optimization of the columns system (in terms of diameter, spacing, etc.).

In the first place, a simple numerical model has been used to simulate the static load tests carried out in the basin site. This has led to the conclusion that the columns under the track would be submitted to relatively low load levels when the track is in service, making it possible to use linear elastic analysis for the three-dimensional parametric study of the subgrade reinforcement technique. Only a short presentation of the main results is given here. More details can be found in previous intermediate reports (Bourgeois, 2007, 2008).

2.6.1 Back analysis of the static load tests

Determination of ground parameters

The static load tests performed in the basin site have been simulated using the finite element method. The model takes advantage of the axial symmetry of the column (figure 30). Results from the load tests, especially the load distribution along the column, have been used to fit the values of the mechanical characteristics of the various ground layers. The results of the fitting are shown by figures 31 and 32. Values of the ground parameters obtained by this back analysis are summed up in table IX.

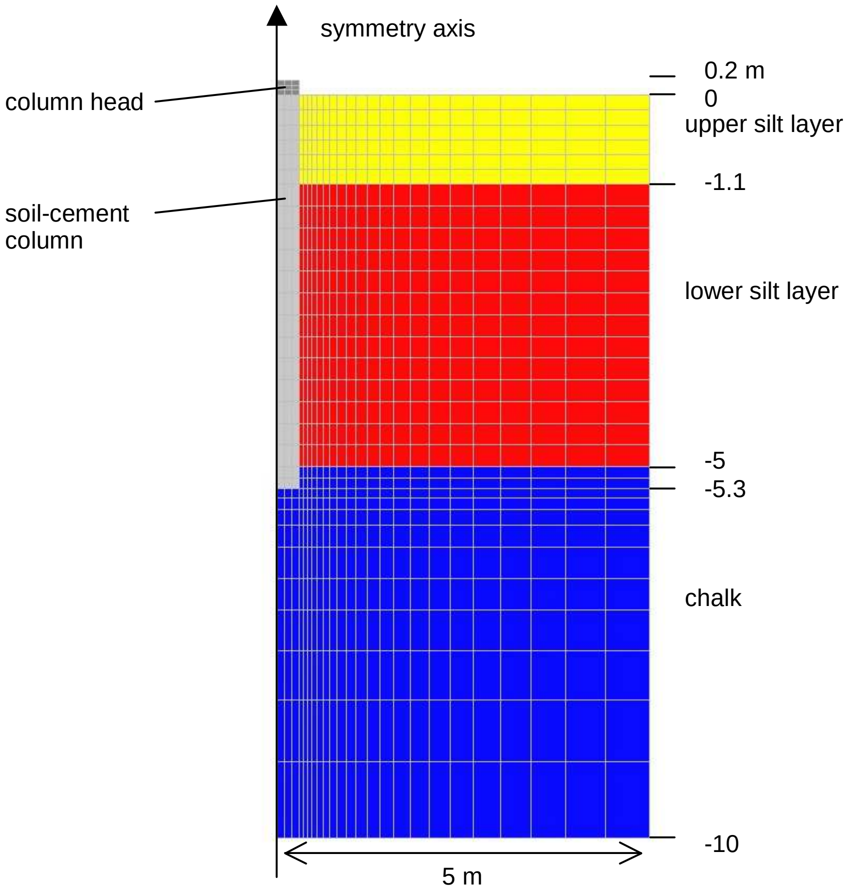


Figure 30 – Mesh used for the simulation of the static load tests on columns P1/P2 in the basin site (2400 nodes, 750 quadratic rectangular elements).

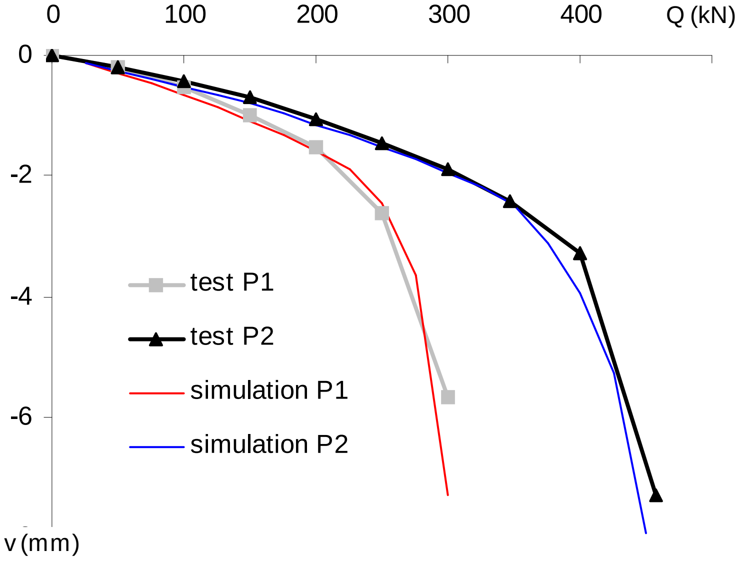


Figure 31 – Comparison of field tests results and results of simulations with fitted ground parameters (columns P1/P2): vertical settlement of the pile head v vs. applied load.

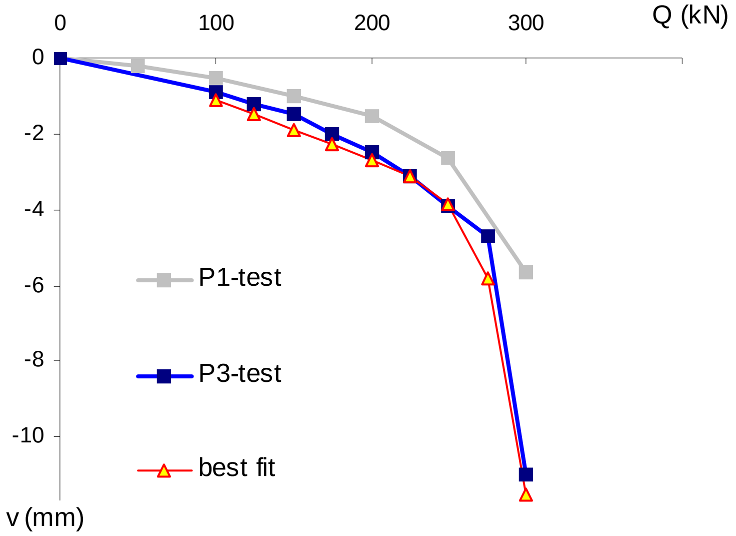


Figure 32 – Comparison of field tests results and results of simulations for various sets of ground parameters (column P3): vertical settlement of the pile head v vs. applied load.

Table IX – Sets of ground parameters giving the best fit between measured and calculated load curves

ground layer	P1	P2	P3
silt	E = 60 MPa ; ν = 0.3 c = 5 kPa ; φ = 32°; ψ = 0 °	E = 60 MPa ; ν = 0.3 c = 11 kPa ; φ = 32°; ψ = 0°	E = 60 MPa ; ν = 0.3 c = 5 kPa ; φ = 32°; ψ = 0°
chalk	E = 400 MPa ; ν = 0.2 c = 40 kPa ; φ = ψ = 0°	E = 400 MPa ; ν = 0.2 c = 70 kPa ; φ = ψ = 0°	E = 400 MPa ; ν = 0.2 c = 55 kPa ; φ = ψ = 0°
column	E = 1700 MPa ; ν = 0.2	E = 2000 MPa ; ν = 0.2	E = 3000 MPa ; ν = 0.2

In all cases, the column head was assumed to be linearly elastic, with E = 8000 MPa and ν = 0.2.

For this study, we have chosen very simple constitutive models (linear elasticity for the concrete, linear elasticity with Morh Coulomb criterion for the silt, linear elasticity with Tresca criterion for the chalk), and preferred not to introduce “interface elements” along the column shaft. Such elements have a direct influence on the results, but introduce additional parameters for which there is generally no clear determination procedure.

Even in the restricted framework used here, a lot of parameters have a more or less clear influence on the load-displacement curve: some of them modify the initial stiffness, others modify the shape of the curve, and others only modify the value of the maximum load.

The influence of various parameters has been investigated numerically. The discussion was focussed on the columns modulus, the dilatancy angle in the ground, and the strength parameters of the silt and chalk layers.

It is worth pointing out that the initial part of the load curves is mainly related to the elastic parameters, and that the back analysis gives consistent values for all columns. The difference in moduli between columns P1 and P2 can be explained by the difference in cement content in the completed column. We do not comment on the modulus of column P3 since the actual geometry of the column is not well known (it is recalled that the mechanical connection between the column and the column head was limited to the steel tube were the strain gauges were placed, as shown by figure 12).

For some parameters, test results give a relatively narrow range of variation: the friction angle of the silt seems to be precisely determined by laboratory tests; the silt cohesion is very likely to be close to 3-5 kPa. Examination of numerical results show that the plunging failure obtained experimentally cannot be reproduced unless the dilatancy angle is (or tends towards) zero. Cohesion c then remains the parameter for which there remains a significant uncertainty. One can also mention that the optimal value of the column modulus (given in table IX) seems to be rather in the lower part of the range of values given by lab tests (Le Kouby et al, 2008b).

Values of c are not the same for all columns. It is necessary to adopt higher values for column P2 in order to reproduce the increase in bearing capacity. It must be emphasized that numerical simulations by themselves cannot explain why the bearing capacities of columns P1 and P2 are different. The difference of mechanical behaviour between two columns built within a time interval of two days at a distance of 2 m is not satisfactorily explained. To get a good agreement between simulations and tests, we had to assume that chalk strength is very different at the toe of the columns. However, there is no support for this assumption, so one could also assume that one of the columns is shorter, or has for one reason or another a poor mechanical contact with the chalk layer. Differences in the building procedure could also account for this difference, which leads to the conclusions that this procedure should be described carefully, and that, should the reinforcement technique be used on a large scale, it would be worth submitting a few columns to static load tests in order to check their actual mechanical properties.

Besides this, the procedure of determination of the parameters relies mainly on curve fitting. In a general way, results (not presented here) show that, for large loads, simulations tend to over-estimate the load transmitted to the column toe (and therefore to underestimate shaft friction): this could be a consequence of the construction process, which might improve locally the strength of silt.

The parametric study could be carried on, for instance to discuss the relative influence of the modulus and the cohesion of the silt layer; one also could distinguish two sublayers in the silt. Such refinements may prove useful to discuss the parameters that can have an influence on the efficiency of the column reinforcement technique, but are otherwise pointless, unless a comparable effort is made to improve the precision of geotechnical data.

2.6.2 Three-dimensional analysis of soil-cement columns under an existing track

In the next step, finite element simulations have been undertaken to discuss the role of the soil-cement columns under an existing track subjected to the load of a train axle. The simulations had several objectives:

- evaluate the ratio of the load transmitted to the columns versus the total load,
- compare the computed vertical deflection with measures (for a single column below the rails, and for the group of five columns),
- discuss whether or not discontinuous rows of columns might induce variations of the vertical deflection when the load moves along the track, which could induce unwanted vibrations,
- discuss the choice of the number of columns and their spacing.

Given the ratio between the bearing capacity of the tested columns and the axle load, it was assumed that the behaviour of the column/track system was likely to remain within the elastic regime.

The main difficulty consisted in reproducing as well as possible the actual three-dimensional geometry of the columns system. Several simplifications were nevertheless necessary:

- the sleepers are not taken into account: a perfect bonding is assumed between the rail and the ballast layer;
- the shape of the cross-section of the rails is replaced by a rectangular area;
- it is assumed that there is a perfect bonding between the columns and the surrounding ground (which is realistic given the construction procedure and the load levels), and between the ground layers.

Mesh

Given the symmetries, only one half of the system needs to be included in the mesh. The same mesh was used (with different materials properties) for the simulations with one or five columns (2 ½ in the mesh). It includes 60000 nodes and 14000 quadratic volume elements. The length of the meshed area was equal to 14 m. The depth and width of the meshed area were set to 7.5 m. The memory needed to perform a linear computation with this mesh using a multi-frontal solving algorithm is roughly 3.5 Gbytes.

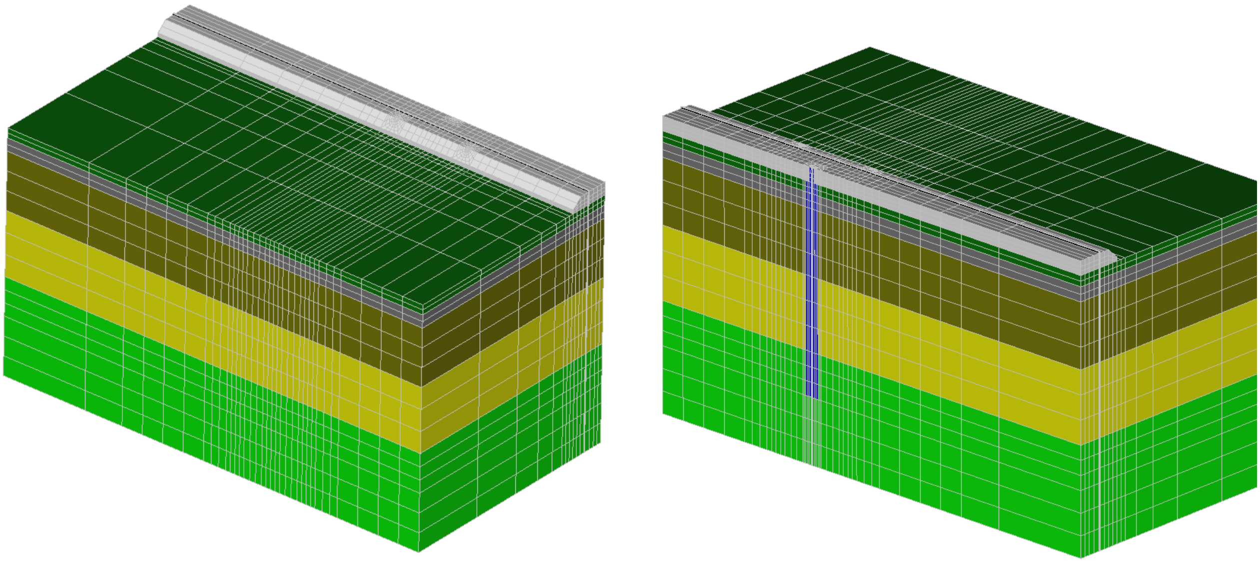


Figure 33 – Isometric views of the mesh.

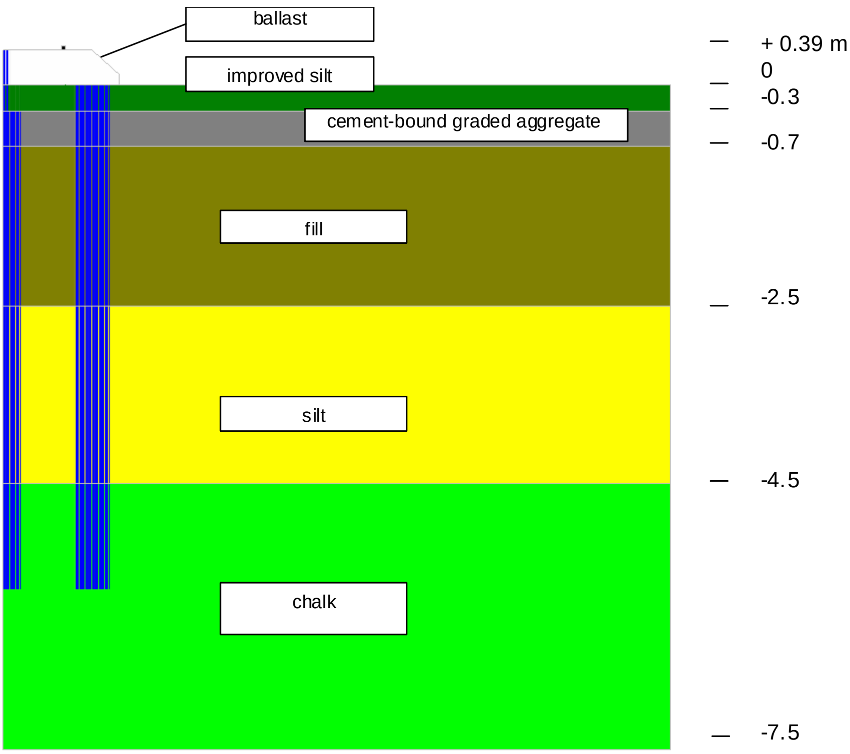


Figure 34 – Section of the mesh in a vertical plane.

Boundary conditions and loads

Displacements in the direction of the outer unit normal direction are set to zero on the lower boundary and on the vertical boundaries of the mesh. The load is a uniform pressure applied on the rail in such a way that the vertical resultant is equal to 150 kN, corresponding to one half of the load applied by one axle (30 t). The load is placed in several positions successively, starting at 1.8 m from the central column then moving towards it.

Constitutive laws and reference values of the parameters

Given the size of the mesh and the relatively low load magnitude, all elements are assumed to remain elastic and computations are made with a linear elastic behaviour: it is not necessary to define initial stress conditions. Properties of the rail have been chosen to reproduce the flexural inertia (3055 cm^4) of UIC 60 rails. Since the real section of the rail is not taken into account, but replaced by a rectangular cross-section, we have adopted an equivalent modulus E_{eq} of the rail given by:

$$E_{eq}\, b_{mesh}\, h_{mesh}^3/12 = E_{rail}\, I_{rail} \rightarrow E_{eq} = 14.7\ 10^6 \text{ MPa}$$

Computations have been carried out with a single value for all columns, E=1000 MPa. This value is based on simple compression tests on samples taken after the excavation of columns (Le Kouby et al., 2008a). Parameters of the silt and chalk layers are those obtained from the back analysis described above. All other parameters are given in table X.

Table X – Values of the parameters used in the reference simulations

layer/material	Young's modulus (MPa)	Poisson's ratio (-)
ballast	325	0.1
improved silt	700	0.2
cement-bounded graded aggregate	2000	0.2
fill	80	0.3
silt	60	0.3
chalk	400	0.3

Vertical deflections

The figure shows the maximum displacement (below the load) for different positions of the load, in the three situations: no reinforcement (NC), one single column (1C) and five columns (5C). In the group of five, the central column is placed at x=0; the others at x= ± 1.2 m.

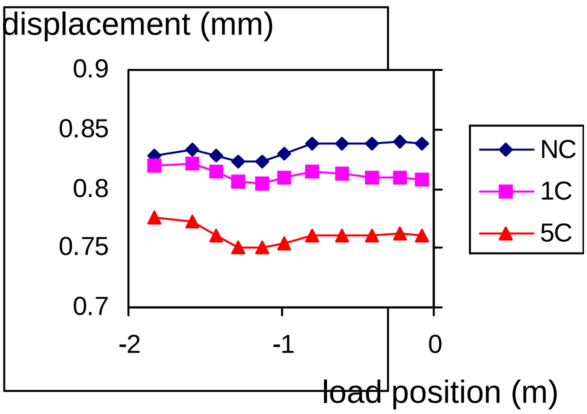


Figure 35 – Vertical deflection (mm) of the sleeper under the load vs. position of the load along the track (Columns are at x=-1.2 m and x=0 m). Diamonds: no columns; squares: one column; triangles: 5 columns.

The computed deflections are consistent with measured values near the sleepers (between 0.76 and 0.9 mm). In the simulation, the columns do reduce the deflections, but the reduction remains relatively small, in any case less than 10 % at the sleeper level: this reflects the fact that the experimental site was located in a zone where subgrade layers were actually rather stiff and did not really need reinforcement (the site was close to the high speed line of Northern France, and the main purpose of the experiment was to check the feasibility of the technique in a railway environment).

It is clear that the network of five columns is much more efficient than the single column. It is also worth noticing that the variations of the deflections in the longitudinal direction (along the track) are small when the load moves, which means that the columns do not induce local “stiff points” that could generate periodic oscillations of a vehicle moving along the track.

Other results show that, in the longitudinal direction, the influence of the single column is negligible at a distance of 2 m.

It can also be noted that the experimental setup included measures of the displacement of the column heads, or, more exactly, of the head of the steel tube used for the monitoring of the columns. Measures show that the displacement of the column head is approximately ten times smaller than the displacement of the sleeper. Simulations do not reproduce this result: they give for the column head displacement a value equal to approximately 70 % percent of the sleeper deflection (0,53 mm in the simulation with five columns). Here there is a qualitative difference between the measures and the simulations. It is not unlikely that references of the measurements were too close to the load to get a correct estimate of the column head displacement. If measures are interpreted as differences between the deflection of the column head and that of the reference points placed at a distance of roughly 60 cm from the load, simulations and measures are in reasonably good agreement.

In the last place, it should be noted also that the simulation gives a much larger deflection just below the rail. This means that the efficiency of columns built outside the interval of the rails, but close to them, is probably greater than that of columns built between the rails.

Load on column heads

The total load brought by the axle on two rails is 30 t (300 kN).

With one column, the load transmitted to the column head is equal to 28 kN, which seems to be rather small. With five columns, the fraction of the load transmitted to the group of columns as a whole is close to 20 % and remains remarkably constant as the load moves along the network of piles.

On the whole, the analysis of the results shows that:

- with no fitting of the model parameters, numerical results are in good agreement with measures;
- in the area where the experimental columns have been built, the deflection reduction is small, because of the good mechanical properties of the improved silt layer and of the cemented aggregate layer;
- the columns do not induce longitudinal variations of the deflection when the load moves, and the deflections in the reinforced zone are relatively homogeneous;
- the influence of a column is not noticeable at a distance greater than 2 m;
- a set of five columns is much more efficient than one single column to reduce deflections.

2.6.3 Comparison of the reference simulation with experiments: load distributions

Numerical simulations gave a load of 29 kN on the head of the single column under the axle load (300 kN), and 26 kN for the central column of the group of five. The measured load was equal to 30 kN for the single column. We made the assumption that the load on the head of the central column of the group of five was equal to 27.3 kN (= 30x26/29). This made it possible to compute the load distribution along the columns, and to compare the simulations with measurements.

The load distributions are shown on figure 36 for the single column and the central column of the group of five. One can see that numerical results show the same trend as the measures: there is a similar difference between the curves corresponding to the single column and the column within the group. Simulations tend to show that the load at the column tip is almost the same for both columns; measures do not make it possible to confirm it.

The shear stress along the column is relatively well reproduced by the simulations, but not in the upper layer, where it is underestimated (table XI).

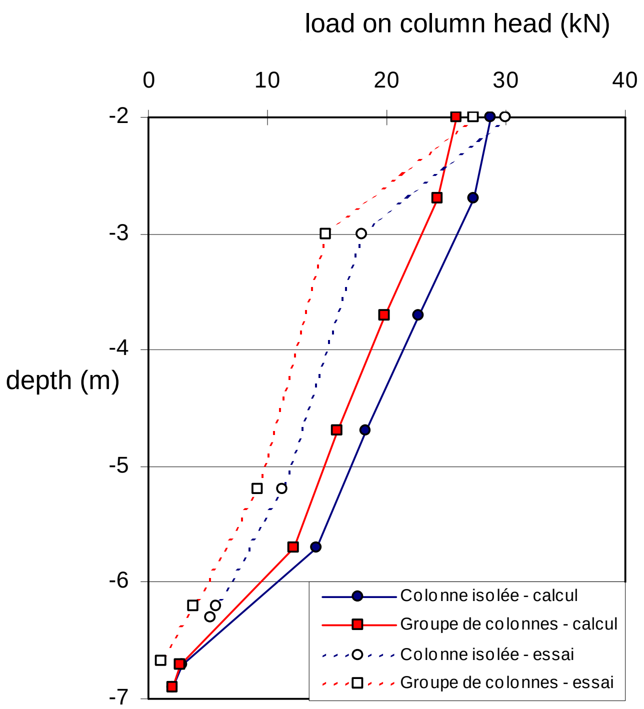


Figure 36 – Load distribution along the columns (comparison between the measures and the simulations for the single column and the central column of the group of five)

Table XI – Comparison between measures and the results of the reference simulation

	Measures	Simulations
Single column		
sleeper deflection	0.89 mm	0.80 mm
column head deflection	0.09 mm	0.64 mm
shear stress along shaft (z < 3 m)	10 kPa	1.8 kPa
shear stress along shaft (3 m < z < 5.25 m)	3.2 kPa	3.3 kPa
shear stress along shaft (z > 5.25 m)	4.3 kPa	9.3 kPa

	Measures	Simulations
load at the column tip	5 kN	2 kN
Central column of the group of five		
sleeper deflection	0.76 mm	0.75 mm
column head deflection	0.06 mm	0.54 mm
shear stress along shaft (z < 3 m)	9.9 kPa	4.1 kPa
shear stress along shaft (3 m < z < 5.25 m)	2.3 kPa	2.8 kPa
shear stress along shaft (z > 5.25 m)	4.3 kPa	7.9 kPa
load at the column tip	2 kN	2 kN

On the whole, numerical results are in good agreement with measurements: the deflections and the shaft friction are similar; the order of magnitude of the load at the column top is correct. However, variations of the friction with depth along the columns are not so well reproduced, and the simulations do not give a good value of the ratio between the vertical displacement of the sleeper and of the column head.

2.6.4 Parametric studies

Silt and fill stiffness

In the simulations, the reduction of deflection provided by the columns is not very significant, and absolute values of the deflections induced by the load remain smaller than 1 mm. The site on which the experimentations were carried out is in fact that of a recent high-speed track, where a rather good subsoil is combined with two stiff layers that transmit the load from the platform to the subsoil. A second set of simulations was performed with lower values of the mechanical properties of the silt and fill layers, given in table XII (reference values are given above, in table X).

Table XII – Second set of simulations: reduced mechanical properties for the fill and silt layers

layer/material	Young's modulus (MPa)	Poisson's ratio (-)
improved silt	325	0.2
fill	50	0.3
silt	50	0.3

The values of the loads transferred to the column heads are given in table XIII. The load increase remains moderate, so that the column remains far from the values for which failure occurred in the case of the columns subjected to static load tests.

Table XIII – Loads transferred to the columns (simulations with weaker ground properties).

position of the load	Simulation with one single column	Simulation with five columns			
	Central column (x= 0 m)	Central column (x= 0 m)	Front column (x=-1.2 m)	Rear Column (x=+1.2 m)	Total
x=-1,825 m	11.98 kN	9.72 kN	19.7 kN	3.49 kN	56.1 kN
x=-1,275 m	19.14 kN	16.35 kN	22.7 kN	6.11 kN	74.0 kN

position of the load	Simulation with one single column	Simulation with five columns			
	Central column (x= 0 m)	Central column (x= 0 m)	Front column (x=-1.2 m)	Rear Column (x=+1.2 m)	Total
x=-0,075 m	28.2 kN	25.4 kN	14.5 kN	13.1 kN	80.6 kN

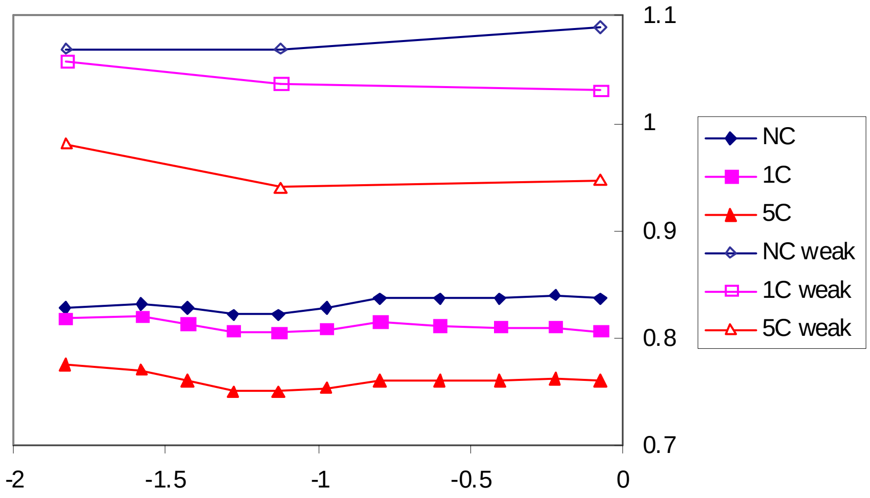


Figure 37 – Deflection under the load (mm) vs. position of the load (the columns are at x=-1.2 m and x=0 m). Diamonds: no columns; squares: one column; triangles: 5 columns.

The response of the ground is qualitatively the same, and the deflection reduction remains of the same order as before (see figure 37).

Number of columns

An additional simulation has been carried out, for the reference values of the parameters, with four columns instead of five: the central column is not taken into account in the computation. Results are plotted in figure 38. They show that the deflection reduction is less homogeneous than in the simulation with five columns, but larger than in the simulation with one single column. From a practical point of view, it might not be necessary to build columns between the rails in the interval between two consecutive sleepers, which could make the reinforcement technique much easier to use, and more cost-effective.

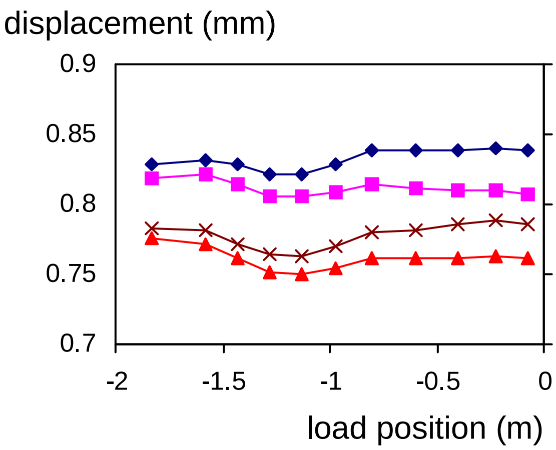


Figure 38 – Deflection of the ballast under the load (mm) vs. position of the load (the columns are at x=-1.2 m and x=0 m). Diamonds: no columns; squares: one column; triangles: 5 columns; crosses: 4 columns.

Column stiffness

A simulation has been carried out with a higher value of the column modulus, remaining in the range of values that could be measured on samples taken from the actual columns: the simulation was carried out with 5000 MPa instead of 1000 MPa. This leads to a relatively small reduction of the deflections, and the general deformation pattern is the same. However, contour lines of the vertical displacement in the vertical plane of symmetry show that gradients of the vertical displacement tend to be larger near the column: increasing the column modulus adds up to reinforcing the ground with rigid inclusions, which may generate oscillations of a vehicle on the track. To investigate further this question, we have performed another simulation, with the reference value of the column modulus, but with a lowered value of the modulus of the aggregate layer that distributes the load to the column heads.

Degradation of the cemented well-graded aggregate layer

On the site where the tests were performed, there is a layer of cemented well-graded aggregate layer, which is much stiffer than the other materials, and can be seen as a slab that tends to minimize the variations of the deflections. A new simulation was carried out with a significantly smaller value of the modulus of that layer. For other layers, we have kept the “weak” values given in table XII.

Figure 39 shows the vertical displacements computed with this smaller value, on the plane of symmetry, below the ballast layer (z=0) and at the depth of the column heads (z=-0.7 m). It can be seen that the local punching of the aggregate layer by the column head is not transmitted upwards, which means that there is little risk of creating undesirable oscillations of a vehicle moving on the track.

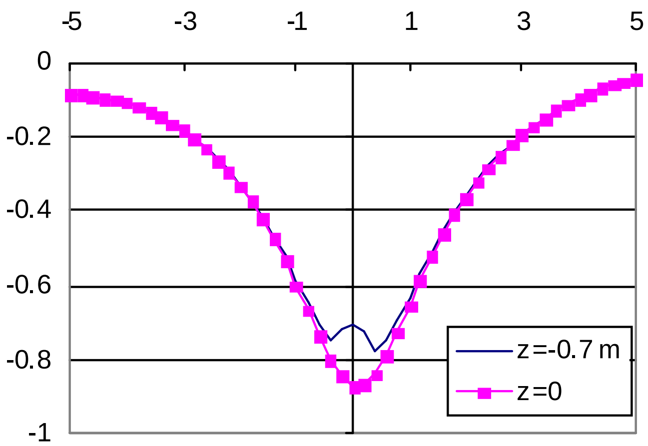


Figure 39 – Vertical displacement (mm) below the track axis: punching of the soil layer by the column (located at x=0).

Conclusions of the three-dimensional parametric study

In a general way, the three-dimensional simulations reproduce the experimental results. They constitute a framework in which the influence of the various parameters can be discussed. The main conclusions are the following:

- for the site of the experiments, the columns provide little deflection reduction;

- the deflection reduction is small but homogeneous;

- it may be simpler and almost as efficient to build two rows of columns on each side of the track instead of three rows (the third being along the track axis between the rails).

The simulations have been made with a mesh that reproduce the spatial positions of the columns, their depth and radius, the geometry of the platform and the thicknesses of the layers. Computation times are not excessive (40 minutes for 11 positions of the load); the disk space needed to perform the analysis and to store the results is large (3.5 Gbytes swap memory, 100 Mbytes to store the results). This is why, in the following section, we revert to the simple axisymmetric configuration.

2.6.5 Influence of the column geometry

The original idea of subgrade reinforcement by means of soil-cement columns consisted in improving an existing track without removing the rails: columns were to be built by a tool driven through the existing platform and between sleepers. The top of the column reaches the bottom of the structure placed under the track. The intermediate layers between the track and the columns are supposed, on the one hand, to transmit the load to the columns, and, on the other hand, to reduce the variations of the deflections at the rail level (so as to avoid the generation of vibrations in the train).

We now discuss the influence of the contact conditions between the columns and the substructure of the railway. It is recalled that the tool used to build the column tends to generate columns with a conical head (because the tool has to unfold two wings once driven to the desired depth, see the very simplified sketch in figure 40). The question is to know if the conical shape of the column head has any influence at all on the overall behaviour. Note that it is necessary to drive a steel tube across the ballast in order to let the tool reach the layers that are to be improved: the tool itself is meant to perform the soil-cement mixing operation, and cannot be used to drill.

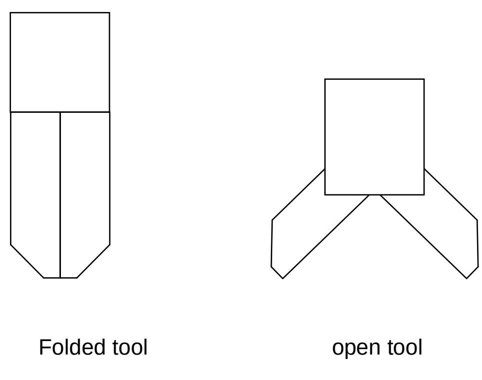
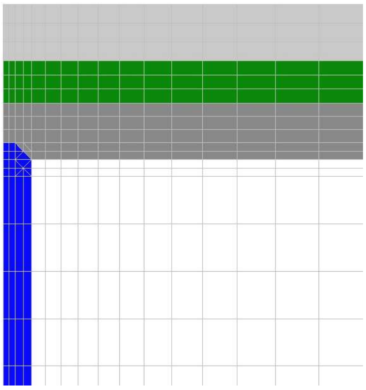
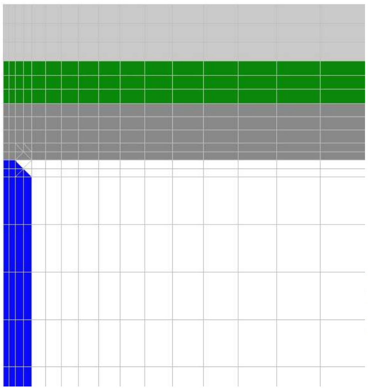


Figure 40 – Principle of the tool opening

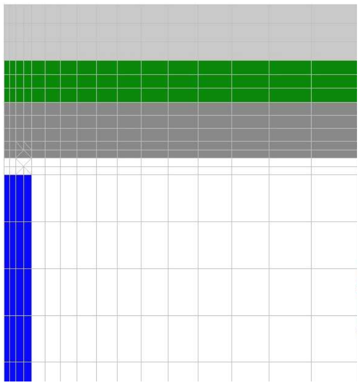
In this section, we investigate three geometries for the column head, shown by figure 41. The layers are the same (ballast, reinforced limestone, cemented well-graded aggregate, fill, silt, chalk from the surface downwards), with the same thicknesses as in the reference three-dimensional simulation. Note however that the ballast layer is supposed to be infinite (given the fact that the computation is carried out in axisymmetric conditions).



a/ column with a conical head intersecting the platform



b/ column with a conical head lower than the platform bottom



c/ column with a flat head

Figure 41 – Influence of the column head geometry

The simulation is carried out in axisymmetric conditions (figure 42); the load is a uniform pressure applied on the upper surface on a radius of 1 m. The total load applied is equal to 50 kN (5 tons), which corresponds to an applied pressure of 16 kPa.

All mechanical properties are identical to the values taken in the reference configuration, except the modulus of the aggregate layer (dark gray): we have adopted here a value of 325 MPa instead of 2000 MPa.

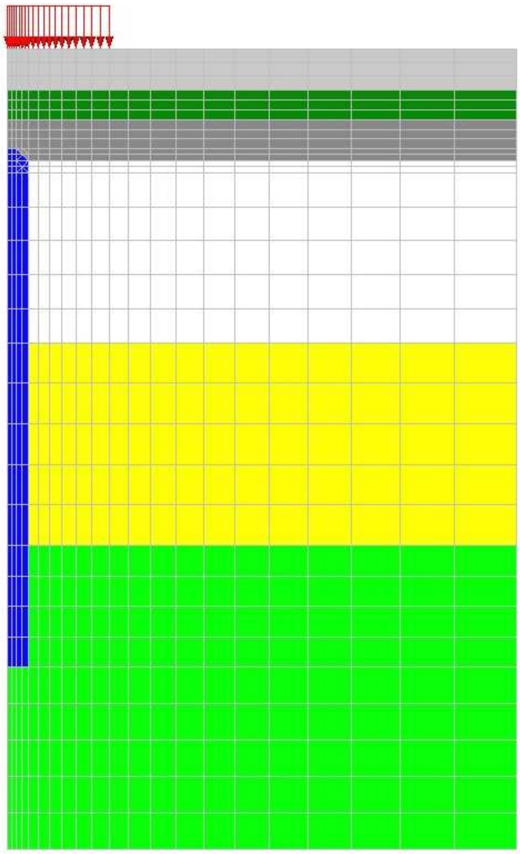


Figure 42 – Applied load

The influence of the column head shape on the computations results is not significant: the difference between maximum vertical displacements is less than 1 %.

In terms of maximum shear stress, the difference is larger, but it is clearly due to numerical stress concentrations that have little physical meaning.

In other words, numerical simulations tend to indicate that there is little influence of the shape of the column head on the global stiffness of the reinforced subgrade, which means that this parameter can be chosen freely on the basis of other constraints of the project, like for instance difficulty of opening the soil-mixing tool in a stiff layer.

2.6.6 Conclusions of the parametric studies

Numerical simulations provide an easy way of investigating the influence of the various parameters of the problem, and make it possible to draw several conclusions regarding the use of soil-cement columns to the reinforcement of the subgrade:

- in the first place, the fact that the column heads do not reach the bottom of the ballast layer seems to make the deflections at the ballast level rather homogeneous, which is an interesting feature;
- on the other hand, the deflection reduction achieved by the technique seems to be rather small (on the order of a 10%) : this is due to the good soil conditions encountered on the site, which did not actually needed to be reinforced;
- from a practical point of view, it seems that two rows of columns placed outside the interval of the rails are almost as efficient as three rows: in other words, the central row of columns, more difficult to build between the rails and between the sleepers, does not provide a significant improvement of the subgrade.

Finally, one can mention that several questions have not been discussed here:

- dynamic effects associated with the columns: all computations have been carried out in static conditions, and given the load levels, most of them have been carried out in a linear framework. However, no attention has been paid to the possible interactions between the columns and the propagation of the waves induced by a train;
- ageing effects have been neglected, since there is little data available on the possible evolution of the mechanical properties of the soil-cement mixed material, which are probably strongly dependent on the initial soil properties. It is worth recalling that the technique discussed here is seen as a way of reinforcing an existing railway without having to remove the track: such a process is a temporary reinforcement, that makes it possible to maintain the exploitation of a line for a limited amount of time, before heavy works are undertaken if necessary.

2.7 Conclusions on the feasibility of subgrade strengthening with vertical soil-cement columns

The main conclusion of this study is that the experimental programme confirmed that it was possible to build soil-cement columns below an existing track without having to remove it.

The technique allowed a quick execution on site and the excavated columns showed strong mechanical resistance.

In addition, the ballast and subballast layers were not polluted by the grout; although some grout was found (very locally) along the steel tube. This is an important advantage of the technique over jet grouting that pollutes the ballast and subballast layers.

It must be noted that the densities of cement used in the presented research work were not common. Soil reinforcement is often carried out with to 50-150 kg of cement per cubic meter in Sweden and Japan. In contrast, within this experimental programme, columns were built with over 300 kg/m^3. It can be expected that their behaviour is somewhat closer to that of rigid inclusions than that of regular soil-cement columns.

From a practical point of view, the proposed technique showed good results in terms of building rate and field load test.

The bearing capacities of the two columns KELLER were respectively 417 kN and 811 kN (extrapolated from the Chin Method on the basis of the maximum measured values of 300 kN and 450 kN, since we were not able to bring the columns to failure). The difference between the two columns, beyond the influence of the W/C ratio and the density of cement, is probably due to the better distribution of cement in the column P2 in comparison to the column P1 (built earlier).

Using the extensometer technique, it was possible to measure the vertical strain along the column, and then to derive the distribution of load. In a general way, 70% of the load was taken by the shaft and 30% by the tip. Examination of lumps of excavated columns showed that the distribution of cement and soil in the soil-cement column needs to be checked to avoid heterogeneity issues. Experimental tools are required to check the homogeneity, in order to ensure a better distribution of grout and a better mechanical resistance along the columns. For the columns built by SOLETANCHE BACHY, the bearing capacity was in the same order as the KELLER columns, both in terms of shaft friction and tip resistance.

As far as the Life Cycle Cost (LCC) is concerned, the soil-mixing technique saves materials (compared to concrete rigid inclusions). From a mechanical point of view, it is less likely to create stiff zones at the column location and damage the ballast which would involve maintenance additional works.

The experimental field tests carried out within the Innotrack project confirmed the feasibility of the technique in grounds less soft than the ones in which lime-cement columns are commonly used, and has yielded preliminary elements on the economic aspects (execution times, quantities of cement, …). However, the actual efficiency of the technique in a full-scale site remains to be investigated, as well as the mechanical behaviour of the columns under dynamic and repeated loadings.

3. Part 2: Inclined columns

3.1 Introduction

3.1.1 General

This part of the report presents the field test of subgrade reinforcement with inclined lime cement columns at Torp, Norrköping. The site has been selected by Banverket. The main funding like design, construction and partly evaluation of this full scale test has been covered by Banverket. Monitoring and special measurement have been performed in the frame of INNOTRACK project SP2/WP2.2. Alexander Smekal Banverket has had responsibility for geotechnical issues and coordination. Construction work and contacts with contractors has been performed by Banverket's Investments Unit. Special geotechnical monitoring and measurements have been carried out and evaluated in cooperation with Swedish Geotechnical Institute (SGI) represented by Per-Evert Bengtsson who mainly contributed with the text and pictures presented in this report. GeoVista AB under leadership of Håkan Matsson performed geophysical measurements.

The railway embankment 3-4 m high at Torp is a typical embankment founded on soft soil where an upgrading to higher axle load is required. Since the stability for new operation-conditions is not sufficient there is a need for measures to be taken so the demands in Banverket regulations are fulfilled.

The embankment has after the construction at the end of the 19th century experienced settlements causing deterioration of track geometry that demanded repeated track alignment. The line opened for traffic on the original first track at 16th October 1873. The second track on this part of the railway line (from Eksund to Okna) was opened in 1920. This means that the embankment was constructed more than 80 years ago for the existing two tracks and more than 130 years for the first original single track (eastern track).

The technical issue at Torp is a too low factor of safety that gives a demand for improvement of the stability. Several methods to improve stability of subgrade have been studied. Banverket has decided to test a new method using inclined lime cement column walls (panels) installed under the existing embankment without any interference with present railway operations.

Full scale in-situ test on a number of inclined walls to verify the possibility to install strengthening under operated railway tracks and impact on track geometry has been performed in the frame of INNOTRACK project.

3.1.2 Selection of test site

The chosen test site is one of the critical sections found when the design for the upgrading of the line for higher axle load (from 22,5 to 25 t axle load) was performed. It has been found that the factor of safety for stability for this part of the line was too low and this railway line can not be opened for new traffic conditions before strengthening measures are carried out.

The strengthening project itself had a monitoring programme for quality control of the execution of the work to secure that it could be performed without causing problems for the existing train traffic. No restrictions on the ongoing train traffic (restrictions on load or speed) were allowed.

Since the strengthening with inclined lime cement column walls has not been satisfactorily proved in Sweden it has been decided to carry out first a full scale test before a decision about strengthening methodology for the whole problematic part of the embankment is taken. There have been two main objectives for the presented full scale test:

- To study the possibility of installation of inclined lime cement column walls (panels) under the railway embankment – installation issues
- To study the impact of installation of strengthening of the track, need for operation restriction and safety issues

3.1.3 Execution of full scale in-situ test, subgrade strengthening with inclined lime cement walls (panels), track levelling

In this report the directions transverse (perpendicular to) the railway line is given as either 'northwest' or 'southeast' or 'left' and 'right'. The direction of the line is from northeast to southwest which means that 'northwest' is equal to 'right' and 'southwest' is equal to 'left'.

The design of the work was performed in 2007 and the execution of the test installation was first planned to be performed in September 2007. After problem with access to the construction site (refused by the land owner) the work was one year delayed and started at the end of May 2008.

- The work started with cutting down all the trees in the area for installation and construction of a new transport road parallel to the track on the northwest side. The location of the transport road was close to the railway and beside the existing pressure berm at the toe of the embankment.
- Excavation of existing pressure berm and building of a horizontal installation platform were performed in June 2008. Excavations were performed under Banverket supervision with certain restrictions on open sections at the toe of the embankment. The excavation was to be performed without causing any measurable impact on the existing track.
- The installation equipments for monitoring of the behaviour of embankment and soil were performed in the middle of June 2008.
- **The execution of full scale tests of the inclined columns started 7th July 2008 and ended 14th July 2008.**
- After installation monitoring continued according to predetermined dates.
- Tests of columns by soundings were carried out according to the previously decided time schedule.
- Geophysical measurements were performed in September 2008 and October 2008.
- **Track levelling of the western track was performed the 23rd of October 2008.**
- Adjustment of the area with installed panels was finished at the end of December 2008.
- The last monitoring was performed between the end of January and March 2009.

3.2 Background and geotechnical conditions

3.2.1 Background

The site of interest, Torp, is located on the railway line between Norrköping and Linköping about 7 km southwest of Norrköping. The nearest village is Eksund about 2 km northeast of Torp. Figure 43 shows an overview of double track at Torp. Subsoil strengthening was carried out on the right side of the embankment. In Figures 44 and 45 two maps show the location of the actual installation site at Torp.

Figure 43 – View of double track and embankment at Torp

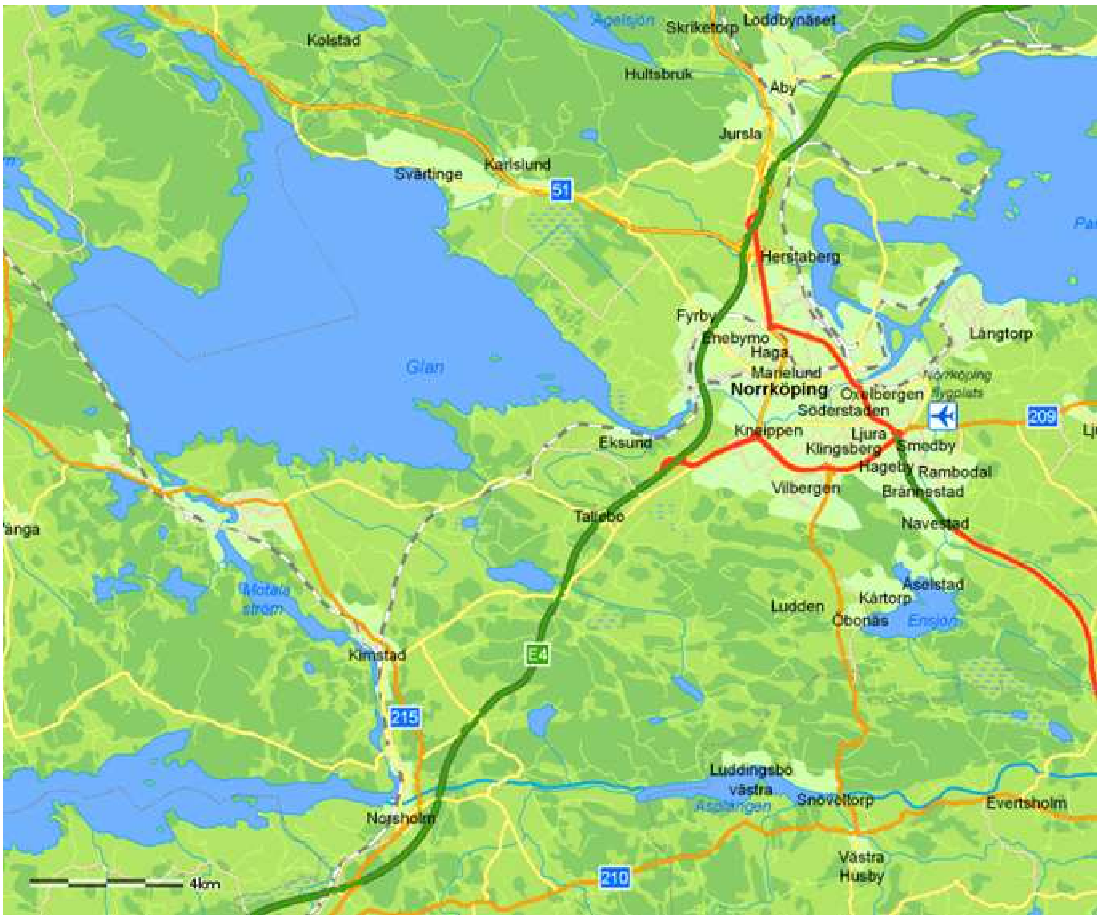


Figure 44 – Location of the test site Torp in Sweden.

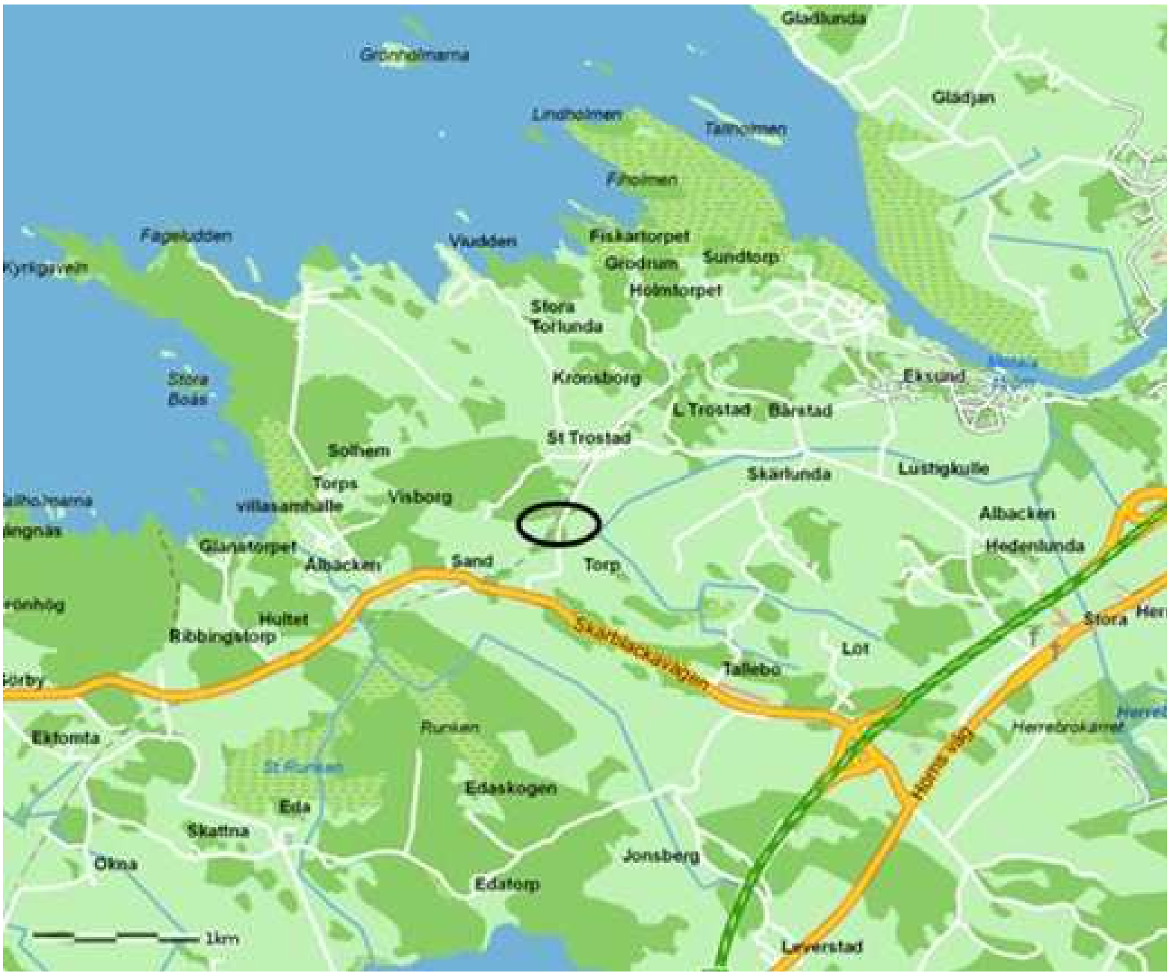


Figure 45 – Location of the test site Torp on the railway line between Norrköping and Linköping.

At Torp the line has two railway tracks and the traffic consist of freight trains, ordinary passenger trains and high speed Swedish trains (X2000).

The part at Torp that requires stabilisation is about 200 m long and the soil improvement is needed from km 193+800 to km 194+000 on both sides of embankment. The full scale test installations presented in this report were performed from km 193+893 to 193+907 on the northwest side (right side) of the embankment. This gives a length for the test installation of about 14 m.

At Torp the embankment rests on natural soil that originally was very soft.

The embankment has a height of 3-4 m above the surrounding ground level. The width of the embankment at 'ground level' is about 20 m.

The part of the line studied at Torp is well known to Banverket maintenance unit because of regular need of track alignment caused by subsoil settlements. It seems however as this has been a minor problem during the last ten years.

To secure stability of the embankment pressure berms have been placed at both sides of the embankment. No detailed information regarding when the pressure berms were placed exists but it is surely many years ago. The pressure berms are made of stones and larger boulders and have a width of about 10 m and a thickness between 0.5 to 1.6 m with the largest thickness at the toe of the embankment.

3.2.2 Geotechnical conditions

The design of the strengthening work was performed by WSP Sverige AB based on results from earlier geotechnical investigations and some complementary geotechnical investigations.

The railway embankment runs through a relatively flat area. The upper part of the soil consists of a thin organic topsoil followed by very soft organic clay on relatively thick deposits of very soft clay followed by frictional soil on rock. The relative density of the frictional soil is high (probably moraine).

The very soft clay has a thickness of about 15 m.

At both sides of the embankment there are pressure berms with a width of about 10 m and a thickness of about 0.5 m to 1.6 m. The pressure berms contain dry crust clay, stones and boulders.

The ground water level is variable with time and located at the ground surface during the wet seasons (winter) and about 2 m below ground level during the dry season (summer).

Stability analyses performed by WSP Sverige for the area of the test installation give a factor of safety for the existing railway at the section km 193+900 in the order of 1.0 (1.02) for the case of undrained analyses and with train load.

Figures 46 to 48 present unit weight, natural water content and liquid limit from the performed laboratory tests.

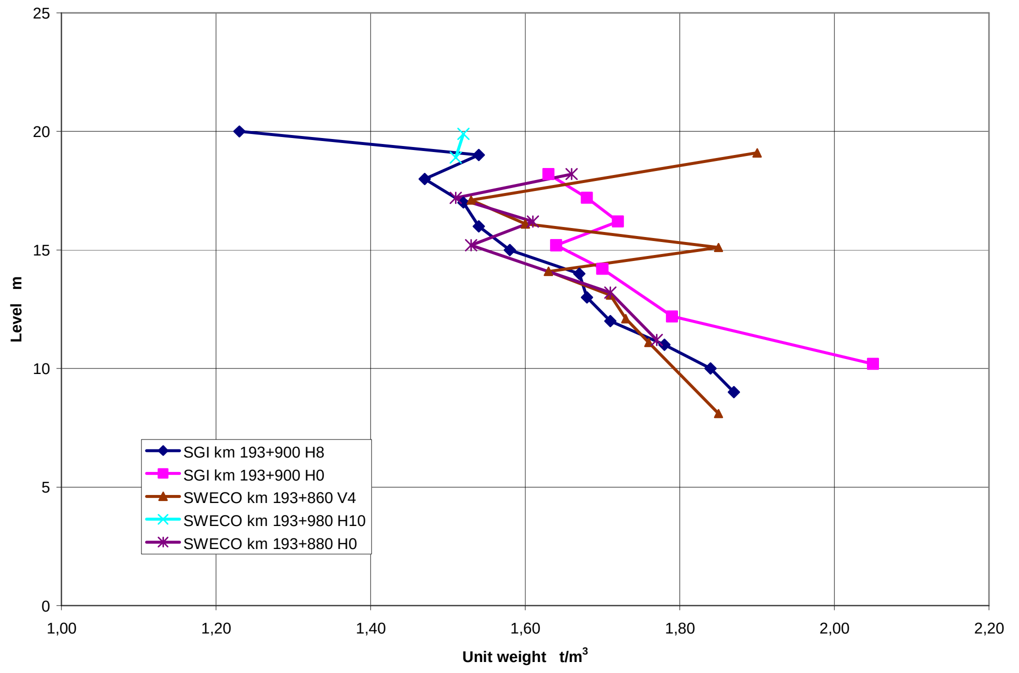


Figure 46 – Unit weight versus level. (H = Right and V = Left) - original ground level +21.5 m.

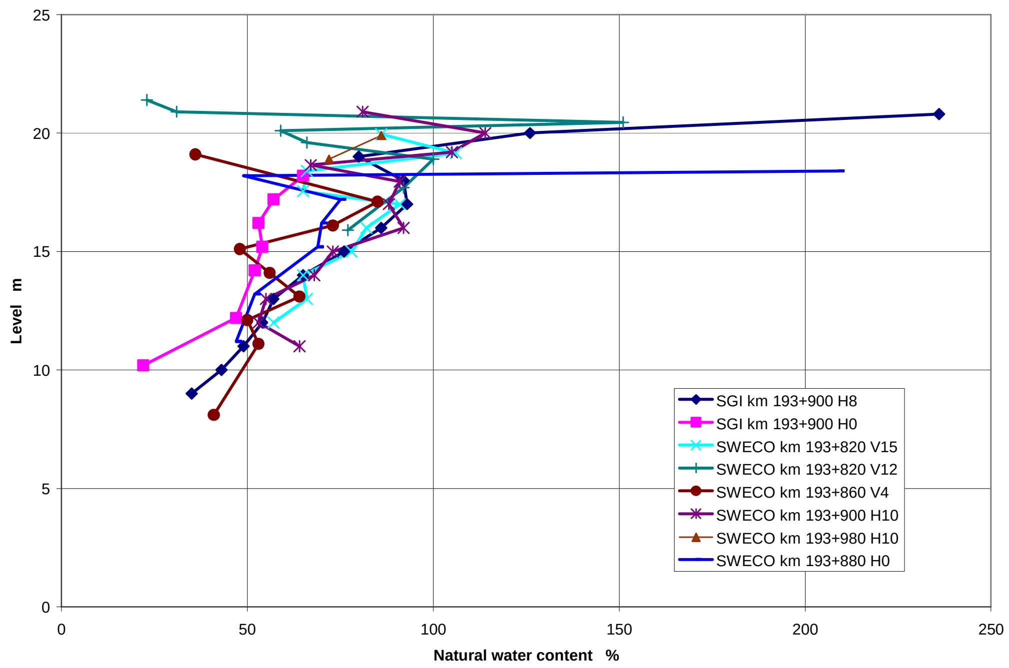


Figure 47 – Natural water content versus level. (H = Right; V = Left) - original ground level +21.5 m.

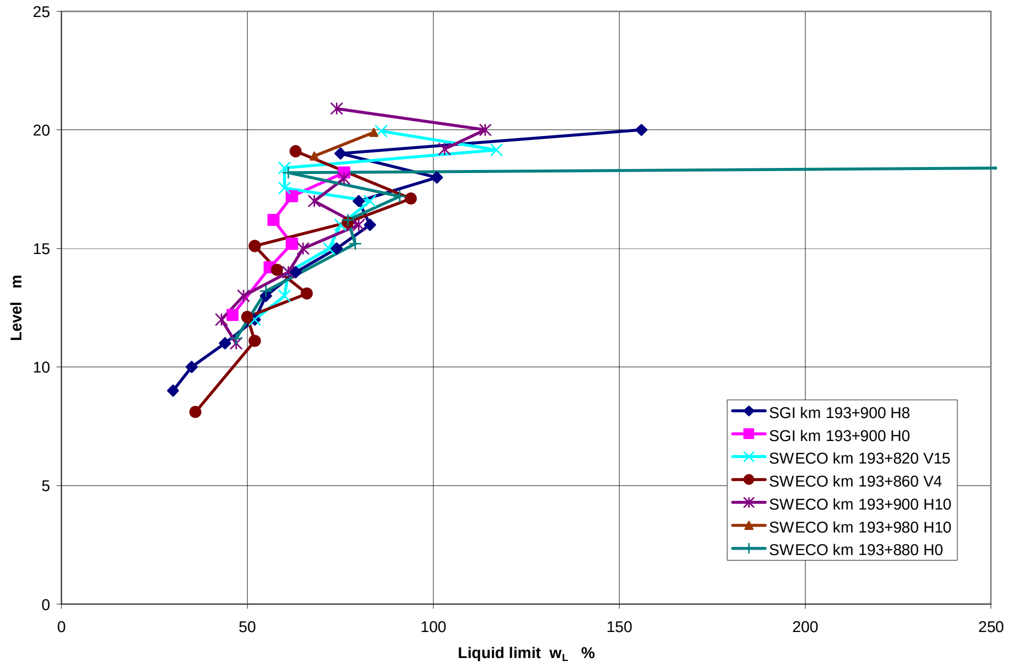


Figure 48 – Liquid limit w_L versus level. (H = Right and V = Left) - original ground level +21.5 m.

In Figures 49 to 53, sensitivity from fall cone test and undrained shear strength from fall cone tests and field vane tests are presented.

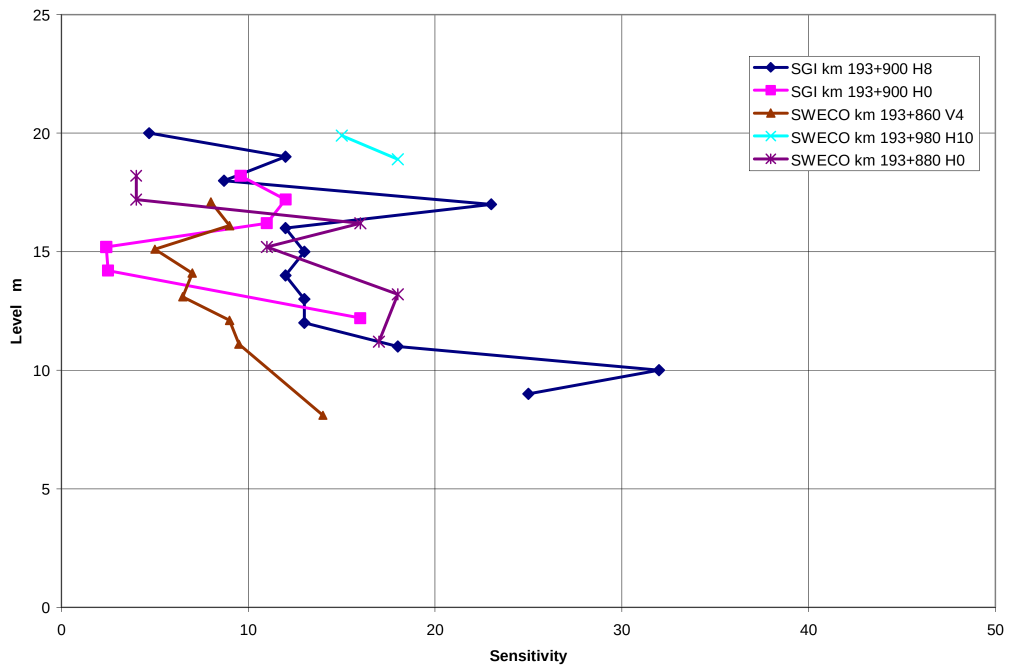


Figure 49 – Sensitivity (fall cone) versus level. (H = Right and V = Left) - original ground level +21.5 m.

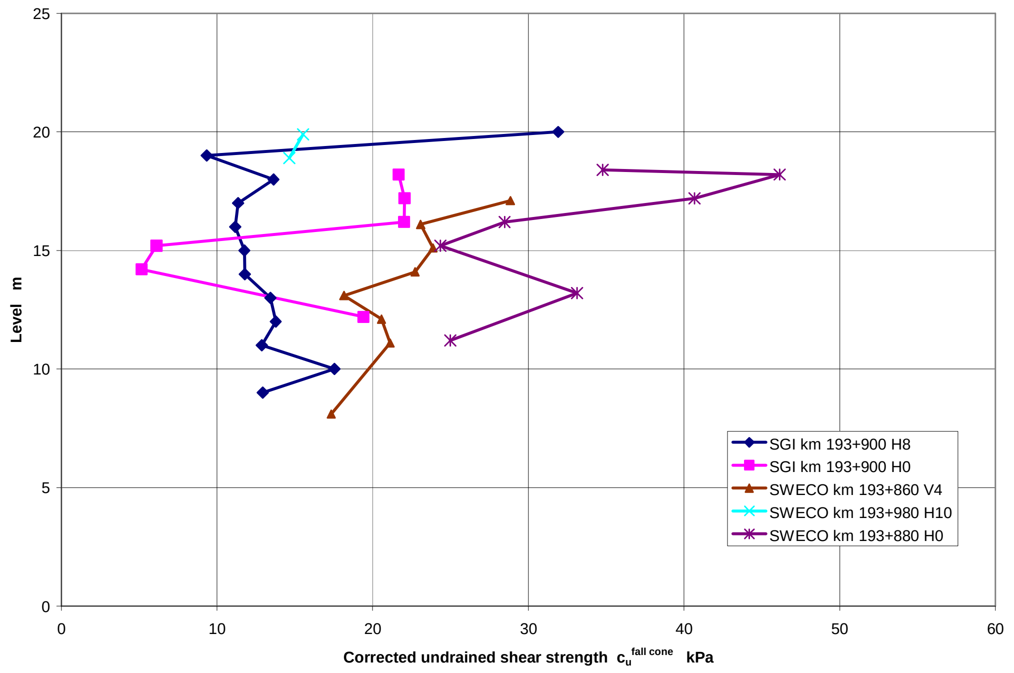


Figure 50 – Undrained shear strength from fall cone test versus level. (H = Right and V = Left). Original ground level +21.5 m.

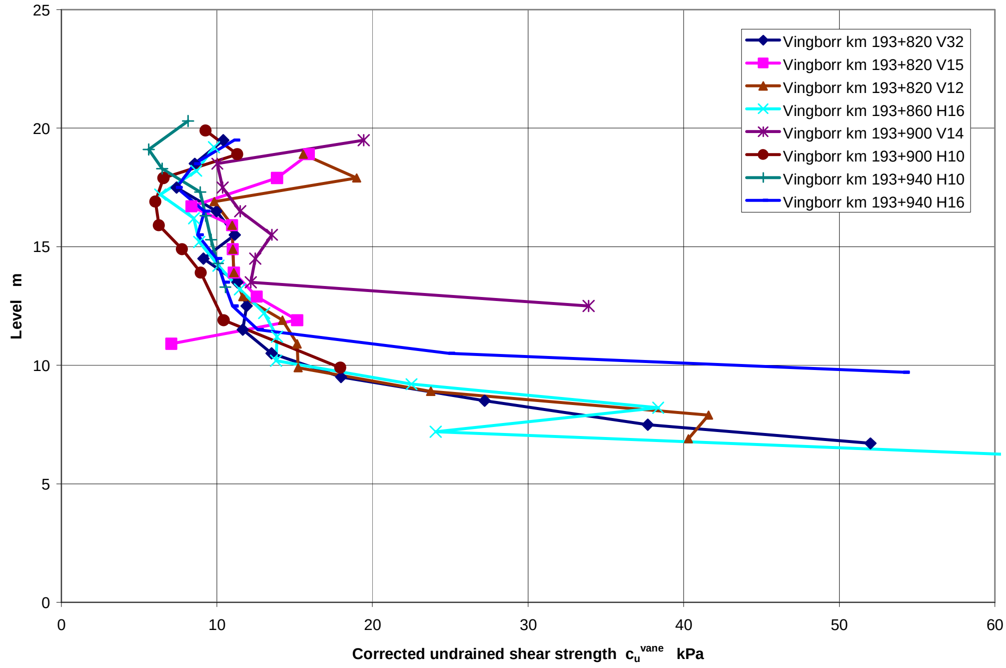


Figure 51 – Undrained shear strength from vane test versus level. (H = Right and V = Left). Original ground level +21.5 m.

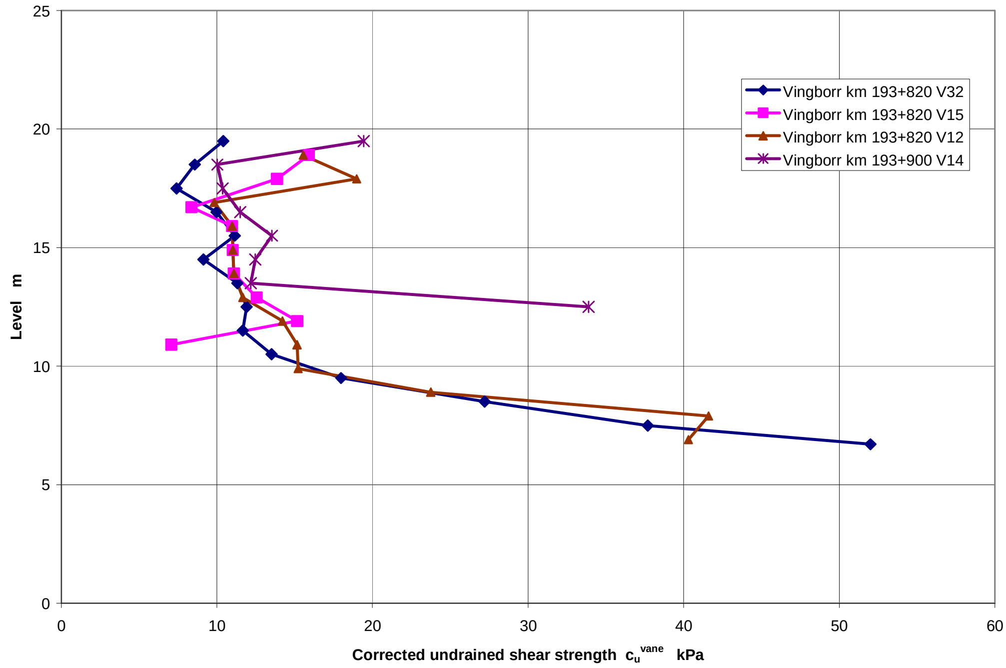


Figure 52 – Undrained shear strength from vane test versus level. (H = Right and V = Left). Original ground level +21.5 m. Only tests on left side.

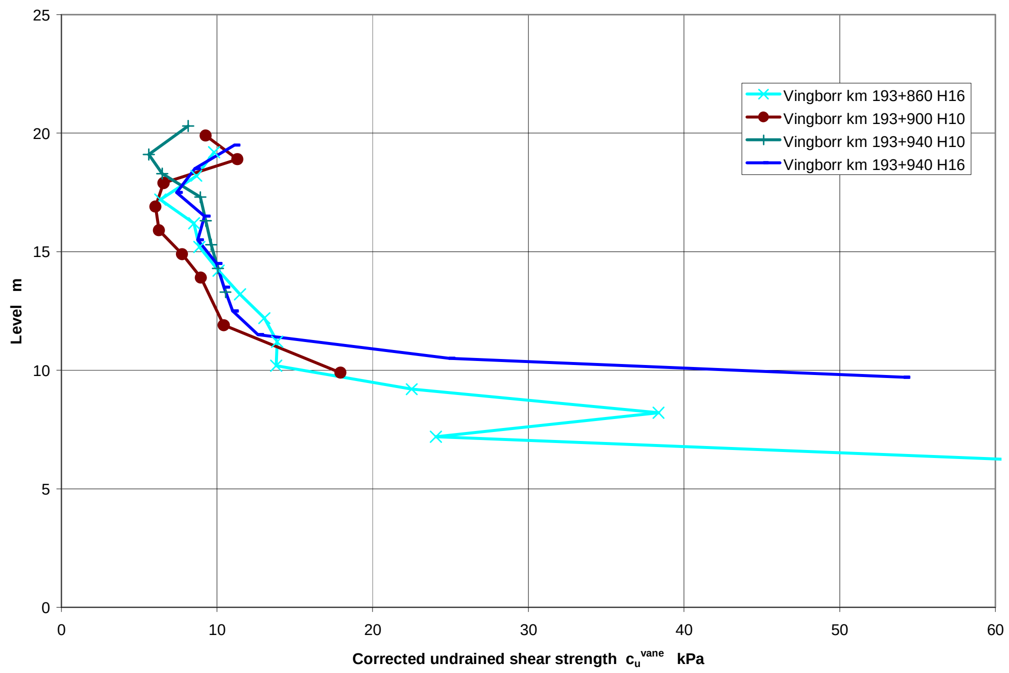


Figure 53 – Undrained shear strength from fall cone test versus level. (H = Right and V = Left). Original ground level +21.5.

3.2.3 Stabilized soil

When investigating the possibility to stabilize a soil it is vital to perform test with soil stabilization in laboratory and tests on the stabilized samples. In this case there exists a general knowledge of this type of soil from other earlier performed works (SGI experience bank). The experience is that it is possible to stabilize the soil and get an appropriate strength for the application in situ.

In this case sampling was performed in the soft clay outside the pressure berm. Stabilization of the samples was performed in the laboratory according to normally used procedures for laboratory stabilization in Sweden.

Samples from the organic clay from depth 2.5 and 3.5 m and from the clay at depth 6.5, 7.5 and 8.5 m have been stabilized in the laboratory with the binder lime/cement in proportion by weight (50%/50%) and a binder content of 100 kg per m^3 of soil. A total of four samples of stabilized organic clay and four samples of stabilized clay at depth 6.5 to 8.5 m were manufactured.

The stabilized samples were stored in plastic tubes in a climate room at 7 degree C.

The samples were tested by unconfined compression test after a curing time of 7 and 28 days. At each curing time two samples from each depth have been tested.

The normal procedure at SGI at the time of testing was that the compression test was preceded by a measurement of shear wave velocity of the sample by the bender element method.

The results from the tests have been used by the designer WSP Sverige AB in the design of the geometry of the panels. The assumption in design is an undrained shear strength of 50 kPa in the organic clay and 100 kPa in the clay below the organic clay.

In field extra columns were installed for in-situ verification of what strength could actually be achieved in-situ. The laboratory results only give an indication if it is possible to stabilize the soil and what type of binder composition that is favourable for the actual soil type. For the types of soils at the site it is the experience that the binder lime/cement is favourable for the clay and that in the organic soil it is favourable to have a higher content of cement in the binder.

Results from tests performed on stabilized samples in the laboratory:

Torp

km 193+800 - 194+000

Stabilized samples at SGI

Sampling performed by WSP with Swedish sampler

km 193+900 H10 **WSP Clay sampler StII at 2006-06-29**

Mixture No 1 with material from depth 2.5 och 3.5 m

Brown organic clay with remains from plants

Binder: Lime/Cement 50%/50%
Binder content: 100 kg/m^3 of soil
Curing temperature: +7° C

w_N = 85 % Natural water content on 'mixed' sample ν = 0,30

Sample no	Age days	ρ t/m^3	w_N %	Comp vel %/min	c_u^{UC} kPa	E_{50} MPa	V_s m/s	G_0 MPa	E_0 MPa	E_0/E_{50}	E_0/c_u^{UC}	E_{50}/c_u^{CU}
1	7	1,55	70	1,6	55	8,3	127,1	25,0	65,1	7,84	1184	151
2	7	1,55	69	1,6	54	5,1	113,9	20,1	52,3	10,25	968	94
3	28	1,56	67	1,3	76	28,1	190,9	56,9	147,8	5,26	1945	370
4	28	1,55	68		74		146,0	33,0	85,9		1161	

Mixture No 2 with material from depth 6.5, 7.5 and 8.5 m

Browngray varved clay with thin layers of silt

Binder: Lime/Cement 50%/50%
Binder content: 100 kg/m^3 of soil
Curing temperature: +7° C

w_N = 71 % Natural water content on 'mixed' sample ν = 0,30

Sample no	Age days	ρ t/m^3	w_N %	Comp vel %/min	c_u^{UC} kPa	E_{50} MPa	V_s m/s	G_0 MPa	E_0 MPa	E_0/E_{50}	E_0/c_u^{UC}	E_{50}/c_u^{CU}
5	7	1,61	59	1,6	108	19,4	210,0	71,0	184,6	9,52	1709	180
6	7	1,59	59	1,6	100	18,1	211,0	70,8	184,0	10,17	1840	181
7	28	1,60	58	1,5	206	87,0	293,0	137,4	357,1	4,10	1734	422
8	28	1,58	58	1,5	159	88,0	295,0	137,5	357,5	4,06	2248	553

3.3 Strengthening method

3.3.1 Background

The strengthening method used at Torp with inclined lime cement walls of stabilized soil has earlier been used in a few projects in Sweden but it has not been possible to retract information from these cases. The dry deep mixing method has been used in Sweden for more than 30 years but normally the columns are vertical and panels have been created by overlapping vertical columns.

3.3.2 Description of strengthening installation at Torp

The stabilized soil is produced by mechanical mixing of a binder and soil with a mixing tool having a nozzle for feeding the binder into the soil. The mixing tool was connected to a rotating Kelly deep stabilisation machine. In Figure 54 is a photo of the machine used at Torp. The binder is normally stored in special containers connected to the installation machine, see Figure 55.

Figure 54 – Lime cement stabilisation machine producing inclined columns at Torp.

Different types of mixing tools exist and usually they are 0.5 to 0.8 m in diameter. At Torp the mixing tool with a diameter of 0.6 m was used, see Figure 56.

Figure 55 – Lime cement containers for the binder connected to the stabilisation machine used at Torp.

Figure 56 – Lime cement mixing tool used at Torp.

The production of a column starts with penetration of the rotating shaft and the mixing tool down to the designed depth. The mixing tool is slowly rotated down to this depth. After this the mixing tool is reversely rotated and lifted while simultaneously the binder is mixed with natural soil. The result is a column of stabilised soil with a circular cross section. The stabilizing process starts and the strength of the stabilized soil increases with time after the installation.

At Torp the specified rotating speed of the mixing tool was 150 rotations per meter and the lift of the mixing tool was 20 mm per revolution of the mixing tool.

The binder was lime/cement with a proportion by weight of 50%/50% and a binder quantity of 23 to 28 kilograms per meter of column which corresponds to about 81 to 99 kilograms per cubic meter of soil for the column of diameter 0.60 m.

The quantity of binder of 28 kilograms per meter of column gives for a column length 14 m and a panel with 16 columns a quantity of binder of about 6300 kilograms for each panel. This binder quantity is pushed into the soil using pressured air. With an assumption that the soil is fully saturated this will result in considerable increase of the soil volume and in the end as a heave of the surface. With a grain density of the binder of 3.2 t/m^3 this gives a volume of about 2.0 m^3 per panel. With a centre to centre distance between panels of 1.3 m this gives a volume of about 1.5 m^3 per meter of the track. This volume has to be seen as a heave of the ground surface. With the assumption that it only occurs within the width of the panel (about 8 m) it corresponds to a heave of 0.19 m of the ground surface. With a binder quantity of 23 kg per m column and with 14 columns in each panel the calculated heave is 0.12 m.

The installation process was monitored directly in mixing machine At Torp the results for each column was presented in a graph, see example in Figure 57. Observe that the intention in this case was a binder quantity of 28 kg per m of column, a revolution of the mixing tool of 150 revolutions per minute and a lifting speed of 20 mm per revolution. The red lines show the limits of the binder quantity given in a tender document, in this case ±10% of the specified binder quantity (28 kg per m of column).

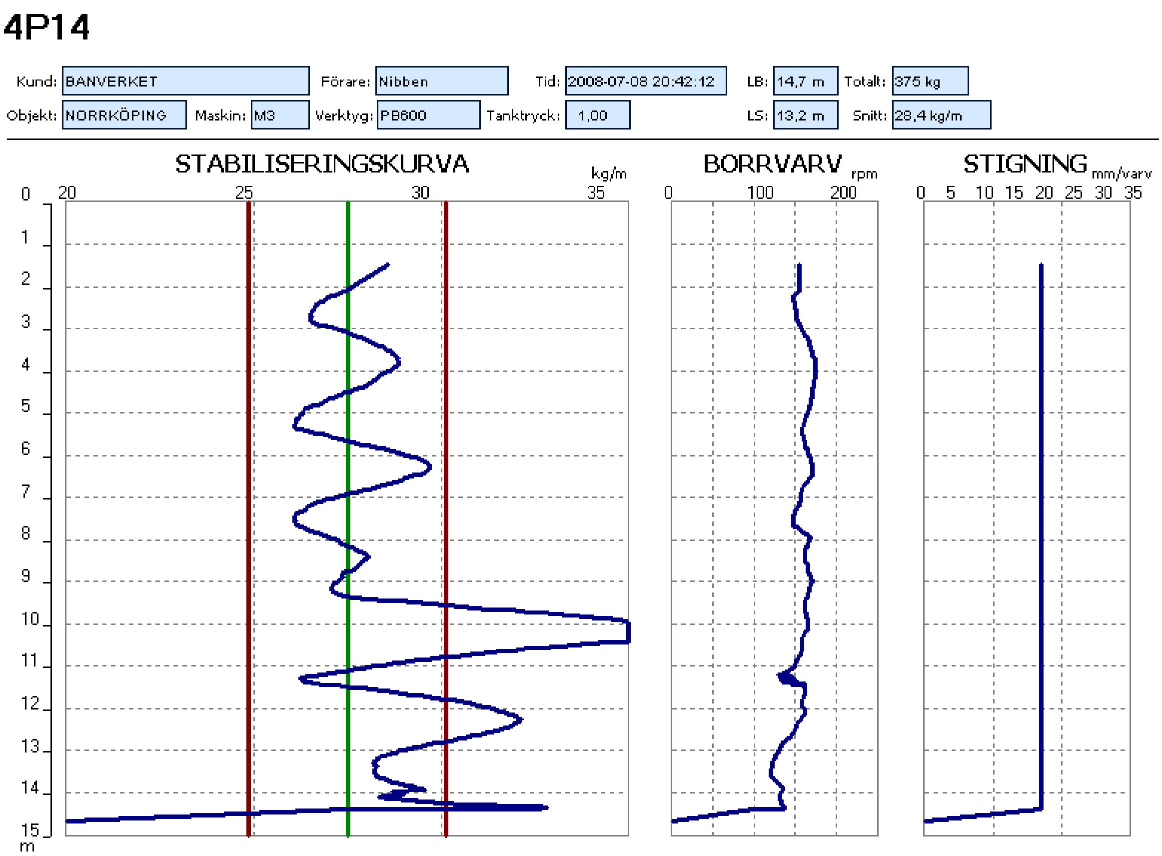


Figure 57 – Results from the monitoring of the installation of column P4:14 in Torp. (Stabiliseringskurva = Binder quantity, Borrvarv = revolution of the mixing tool, Stigning = lift in each revolution of the mixing tool).

Figure 58 shows a plan over the area where the test inclined lime cement walls were installed.

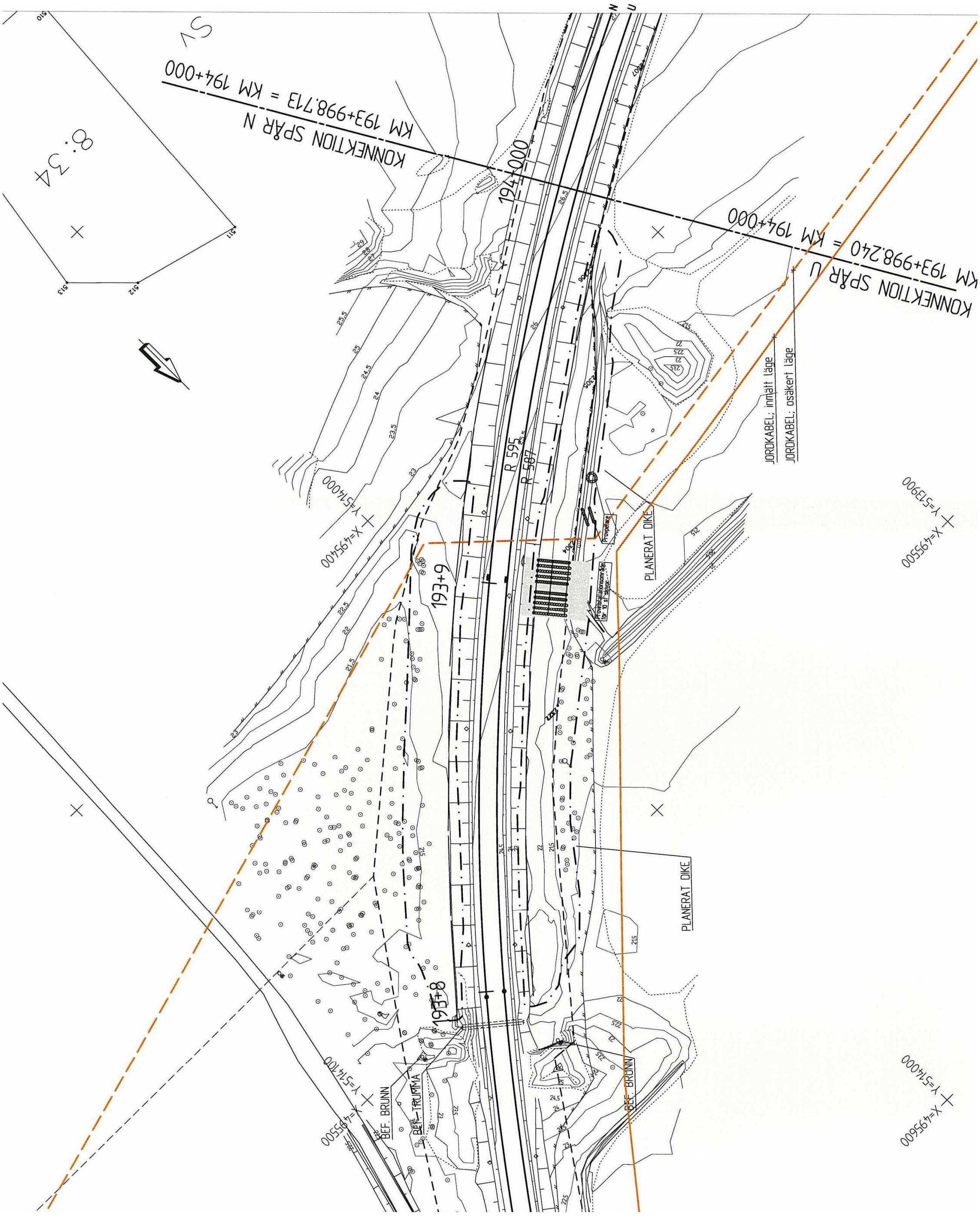


Figure 58 – Plan over the installation area at Torp.

In Figure 59 a plan over the installed columns in Torp is presented. It includes also columns installed in a ring with twelve vertical overlapping columns with length 10 meters, two short test panels with inclined columns and the ten full-size panels with inclined columns installed under the railway embankment.

In Figure 60 a cross section at the middle of the installation of the test panels at the km 193+900 is presented.

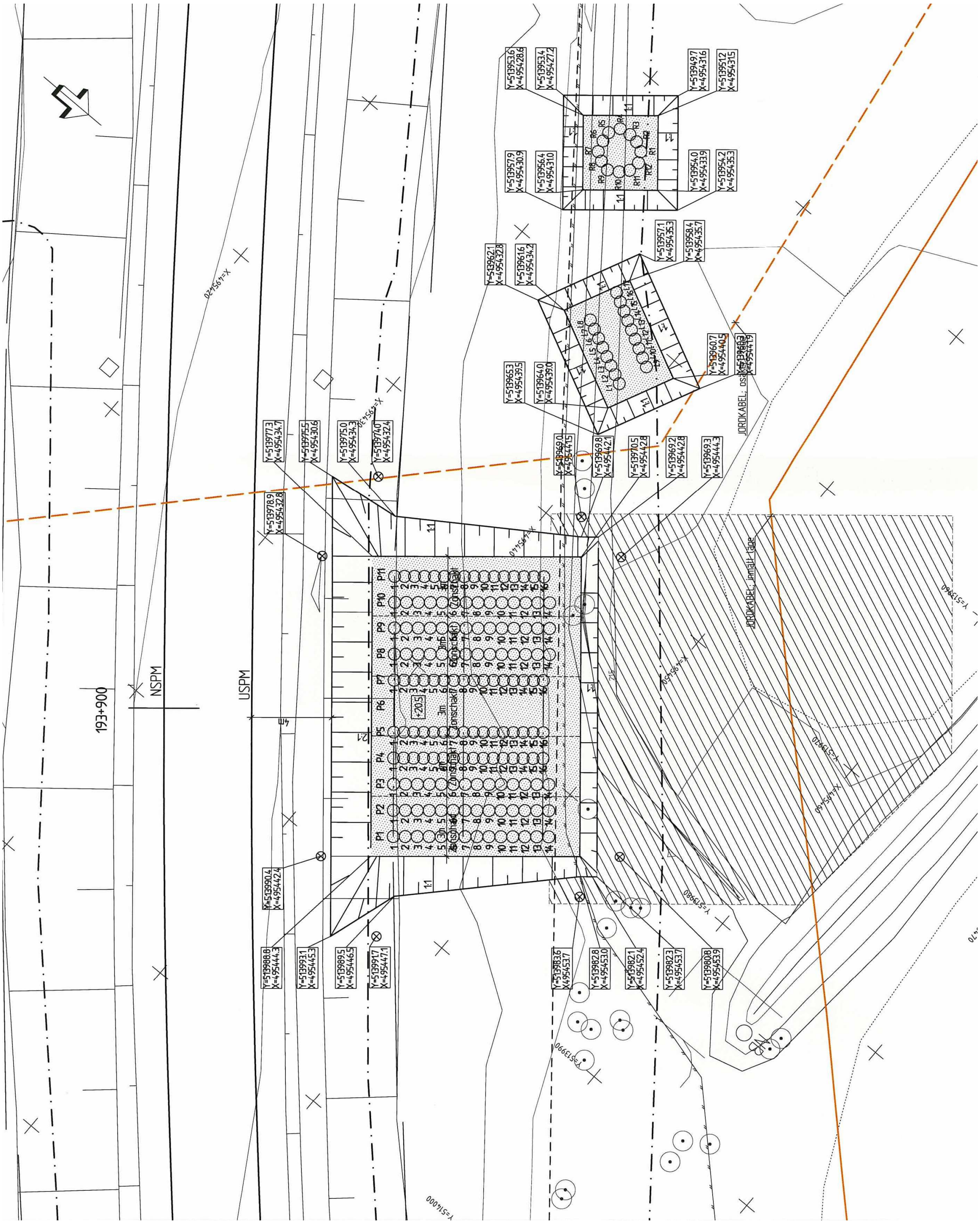


Figure 59 – Plan over the all installed columns at Torp.

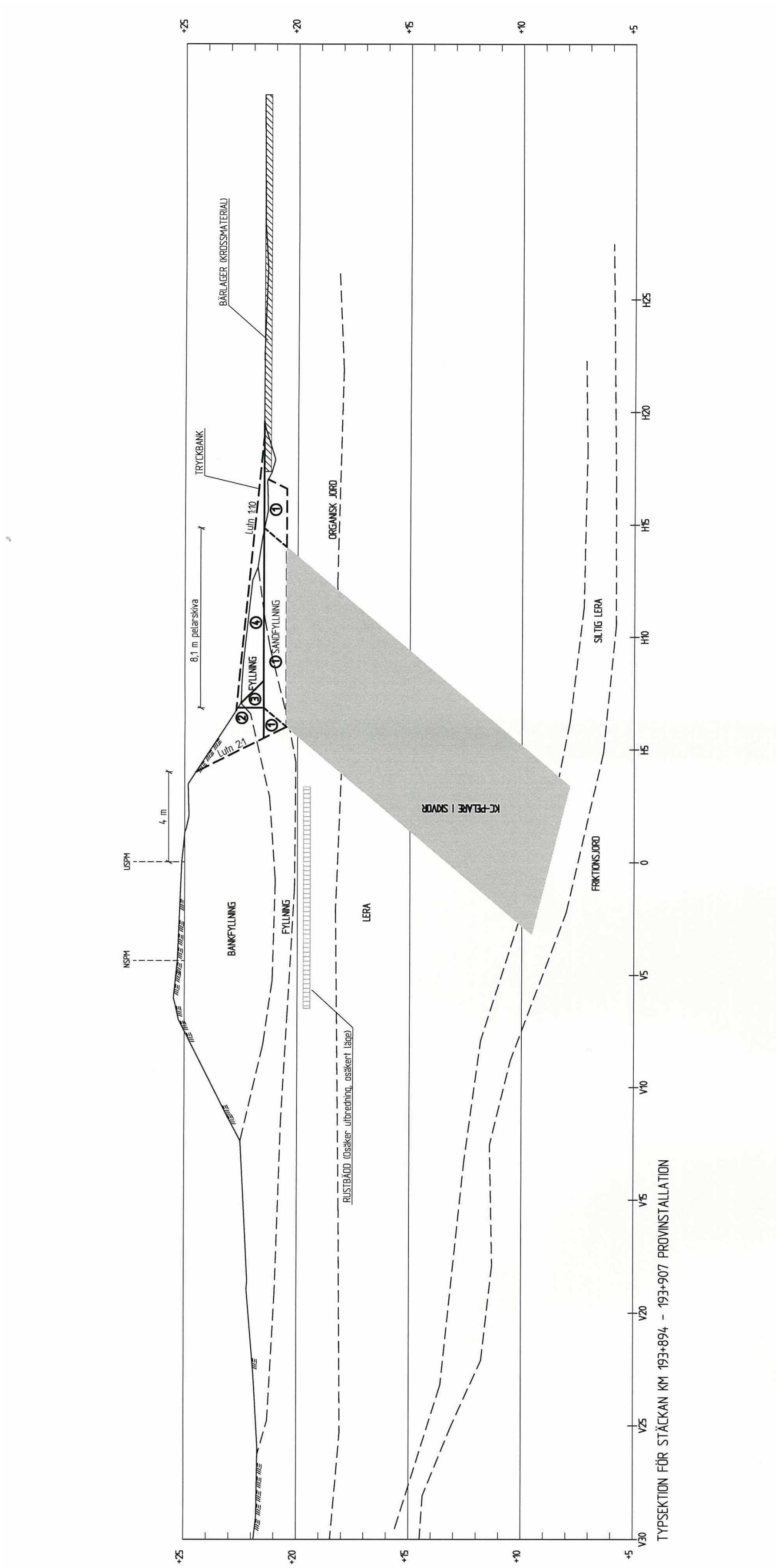


Figure 60 – Cross section at km 193+900 at Torp.

3.4 Monitoring

3.4.1 General

The aim of the monitoring was primarily to give information about the status of the track in order to secure operation on the existing track and secondly to give information about the behaviour of the ground when all the works necessary for the stabilization of the soft soil were performed.

The deep dry mixing method with inserting a dry binder in the soil leads to questions not only about the behaviour at installation but also about what happens after the installation and impact on above placed track.

In this case it was decided to perform the monitoring just before the start of installation to about 6 months after the installation of the inclined columns.

3.4.2 Monitoring system

The monitoring system was designed for monitoring of rail position and for observation of behaviour of the soil in the actual section where the installation of the stabilized inclined column walls was performed.

3.4.3 Track

The monitoring system for the track involved measurement

- twist of track was measured on both tracks
- track position by total station positioning of steel nails on sleepers

Two steel nails were placed on each end of the sleeper. Totally 72 nails were fixed. The measurements of twist were performed with an internal distance of 5 m in the area of installation and 20 m outside the installation area. In Figure 61 the tool used for measurement of twist of the track is shown.

Figure 61 – Measurement of track twist.

The measurement of twist of track were performed at section km 193+790, 193+800, 193+820, 193+840, 193+860, 193+880, 193+885, 193+890, 193+895, 193+900, 193+905, 193+910, 193+915, 193+920, 193+940, 193+960, 193+980 and 194+000.

This gives in total 18 measurement points for twist of track on each track.

Figure 62 shows a photo of one of the steel nails at the end of a sleeper.

Figure 62 – Steel nails for positioning of track.

Figure 63 shows a photo of the positioning system used at Torp. The measurement of track position were performed at section km 193+800 to 193+880 with a distance of about 20 m, at section km 193+880 to 193+920 with a distance of about 5 m and at section 193+920 to 194+100 with a distance of about 20 m. This means that the rail positions were measured a distance of about 100 m at both sides from the measurement section km 193+900.

Figure 63 – Positioning system used at Torp.

3.4.4 In soil

The objective of the measurements performed in the soil was to detect events under the necessary work for the installation of the columns.

It was decided to use one measurement section with detailed monitoring, taking into account the problem with orientation of the instrumentation as well as the risk for the mixing tool to hit and damage the installed equipment. After some discussion it was decided to skip one of the planned walls (P6) to get more space allowing safe installation of the monitoring equipment. The test section was chosen as the mid section (P6) of the panel P1 to P11 at section km 193+900.

The monitoring in the test section included

- nine settlement markers at the soil surface or just below (0.5 -1.5 m) this level to monitor the displacement of the surface
- one bellow hose to monitor the vertical displacement (distribution) with depth
- two inclinometers to monitor the horizontal displacement with depth. The inclinometers were installed at the toe of the embankment on the western and eastern side.
- pore pressure units (BAT) at four levels in the soil
- one stand pipe (ground water) in the lower frictional material considered as permeable

In a reference station the monitoring included

- pore pressure units (BAT) at four levels in the soil
- one stand pipe (ground water) in the lower frictional material considered as permeable

The monitoring in the test section in km 193+900 is presented in Figure 64 and 65.

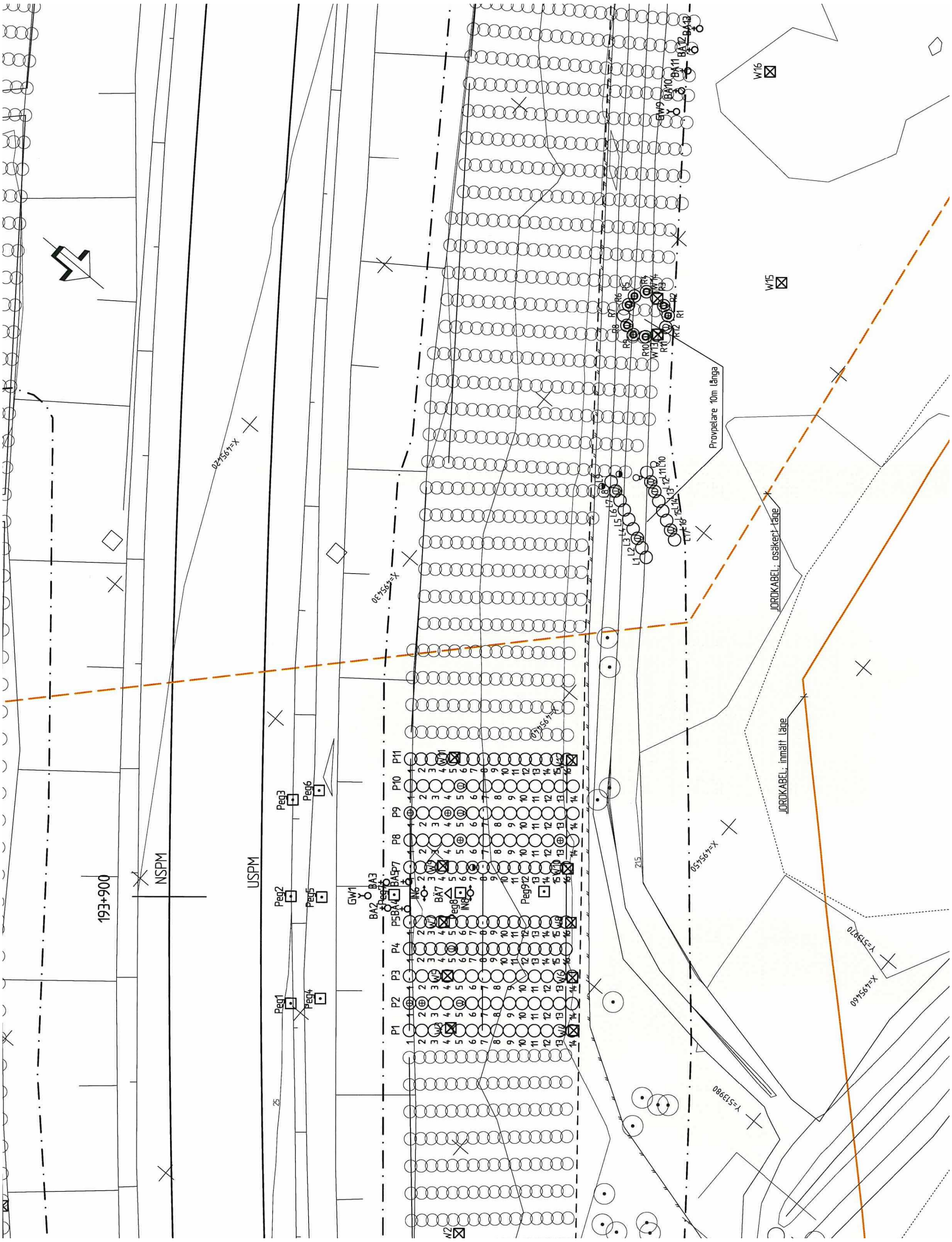


Figure 64 – Plan of the monitoring in section km 193+900 at Torp.

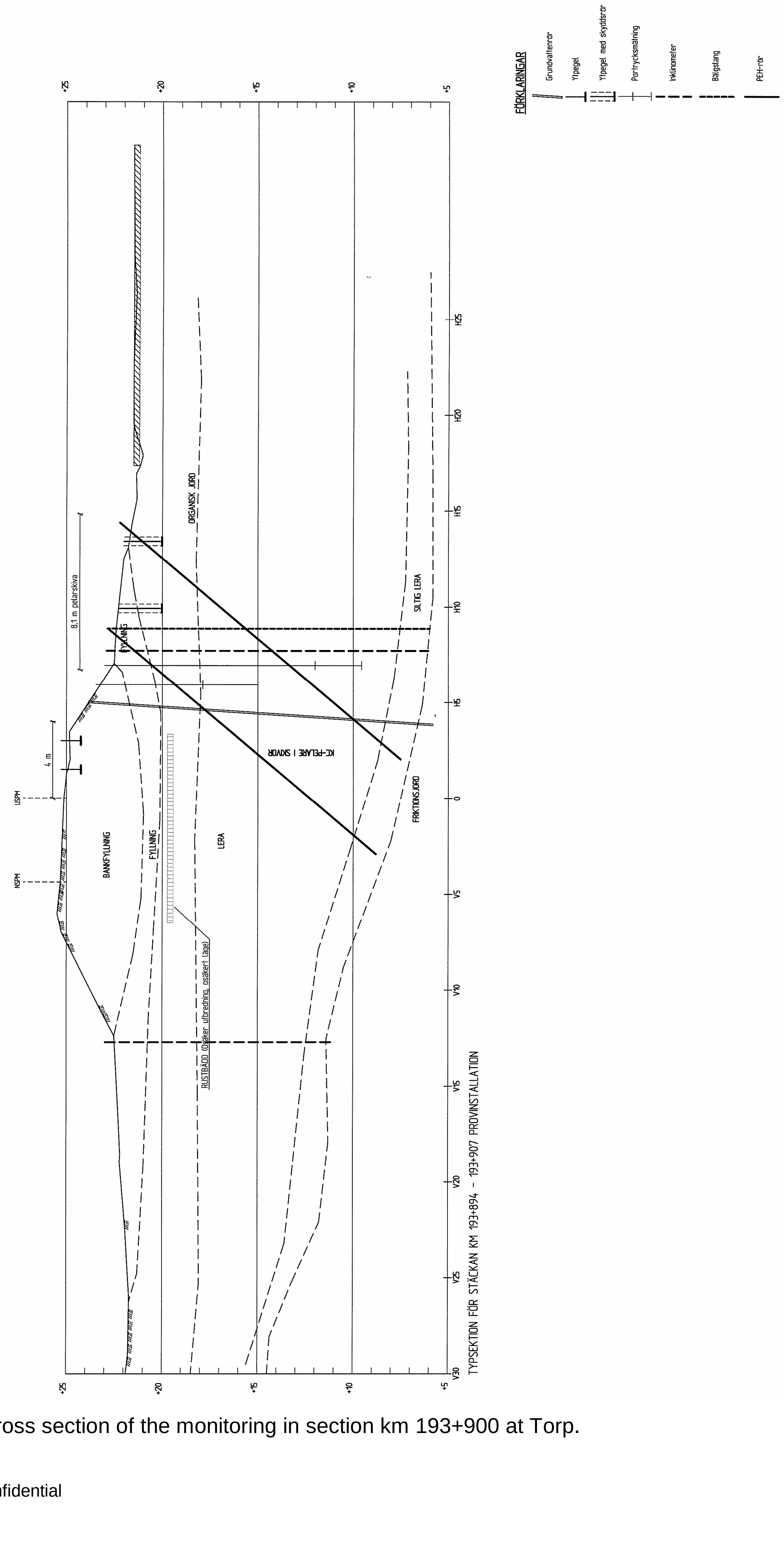


Figure 65 – Cross section of the monitoring in section km 193+900 at Torp.

Settlement markers P1 to P9

The settlement marker used at Torp was a screw mounted on a steel pipe. The installation was made by screwing down the steel pipe to fix measurement point.

The settlement markers P1 to P6 were installed in June 2007 before the planned column installation in September 2007. The settlement markers P7 to P9 were installed in the middle of June 2008.

The settlement markers P1 to P3 are located just outside the end of the sleepers, about 1.4 m from the centre of the western track. The settlement markers P4 to P6 are located about 3.0 m from the centre of the western track. The settlement markers P7 to P8 are located about 6.5, 9.5 and 13.5 m respectively from the centre of the western track.

The measurements of the settlement markers together with the measurements on the track give the possibility to detect displacements in an area from the eastern track to almost the western edge of the installed panels.

Bellow hose

The intention with the bellow hose was to measure the vertical displacement (distribution) with depth and in this case down to the frictional material. The vertical distance between the measurement points was about 1 m. The bellow hose was installed in the middle of June 2008 about 9.0 m from the centre of the western track.

Inclinometers

The objective with installation of the inclinometers was to measure the horizontal displacement versus depth at two points. One of the inclinometer was installed at the toe of the western part of the embankment and one at the toe of the eastern part of the embankment. The western inclinometer was installed about 8.0 m from the centre of the western track and the eastern inclinometer was installed about 12.0 from the centre of the western track. The inclinometer was installed down into the friction material.

Pore pressure (BAT)

Closed pore pressure systems type BAT were installed at the test section km 193+900 and at a reference station outside the pressure berm. The pore pressure units were installed at four different depths and at the distance of 6 m and 7 m from the centre of western track. At the measurement section 193+900 the units were installed at the level (depth) of +17.73 (4 m), + 14.84 (7 m), +11.79 (10 m) and +8.81 (13 m). At the reference station the units were installed at level +17.51 (4m), +15.56 (6 m), +13.57 (8 m) and +11.50 (10 m).

Ground water stand pipe

The ground water stand pipe was installed in the frictional material below the clay. The level was +5.07 in the measurement section km 193+900 and +7.99 at the reference station located outside of the in-situ test site. At the measurement section the stand pipe was installed 5.5 m from the centre of the western track.

3.5 Execution of installation work

3.5.1 General

The execution of the installation work demanded good access to the working area. A new transport road was constructed and connected to an existing road northeast of the test section at about section km 193+900. The transport road was built northwest of the embankment at the toe of the embankment and connecting to the test area at the toe of the existing pressure berm.

To install the columns it was necessary to remove the existing pressure berm because it contained large boulders and stones that were not possible to penetrate by the installation tool. The execution of the installation needed a relatively even horizontal working platform of gravel/sand to allow for good positioning of the columns.

The installation of the columns was performed from 8th July 2008 to 14th July 2008.

In the end of December 2008 an adjustment of the area was performed. The adjustment was made with an about 0.5 m thick fill in the area with the 10 panels. An adjustment of the ground surface was also made for the test area with the 17 inclined columns, where the ground surface was clearly lower than in the surrounding area. A more detailed description of the execution of the installation work is given below.

3.5.2 Excavation of pressure berm and placement of bearing layer

The existing fill beside the embankment was considered as a part of a pressure berm. It was not clarified when these pressure berms were constructed but it was probably in connection with the building of the second track, opened in 1920, or after that time. In Figure 66 there is a photo of the site shortly before the excavation started.

The excavation was performed in segments with a width of 3 m along the track line and with a slope V:H = 2:1 that starts 4 m from the middle of the western track. All boulders and stones were taken away. A check with a steel bar with a 10 mm diameter was performed further into the soil to detect if there were any boulders in the upper layer. It is important that there are no large stiff objects in the soil since this would cause problems when installing the columns. After the inspection the excavated area was directly filled with sandy gravel before the next segment of excavation started. The excavation and refilling was performed within an area extending well outside the free area needed for the installation of the 10 test panels. Figure 67 shows a photo from the excavation and replacement of fill.

Figure 66 – Construction site (embankment) before the start of the excavation at Torp

Figure 67 – Excavation and placement of new fill to substitute the existing pressure berm.

The excavation and refilling was successfully performed 14th June 2008, in a shorter time than was expected. The replacement fills were carefully levelled horizontally to create a good platform for the installation of the columns in the 10 panels. Excavation and replacement with a sandy gravel fill was also performed in the areas where the ring of columns and the two short test panel columns were to be installed. The excavated areas are indicated in Figure 68.

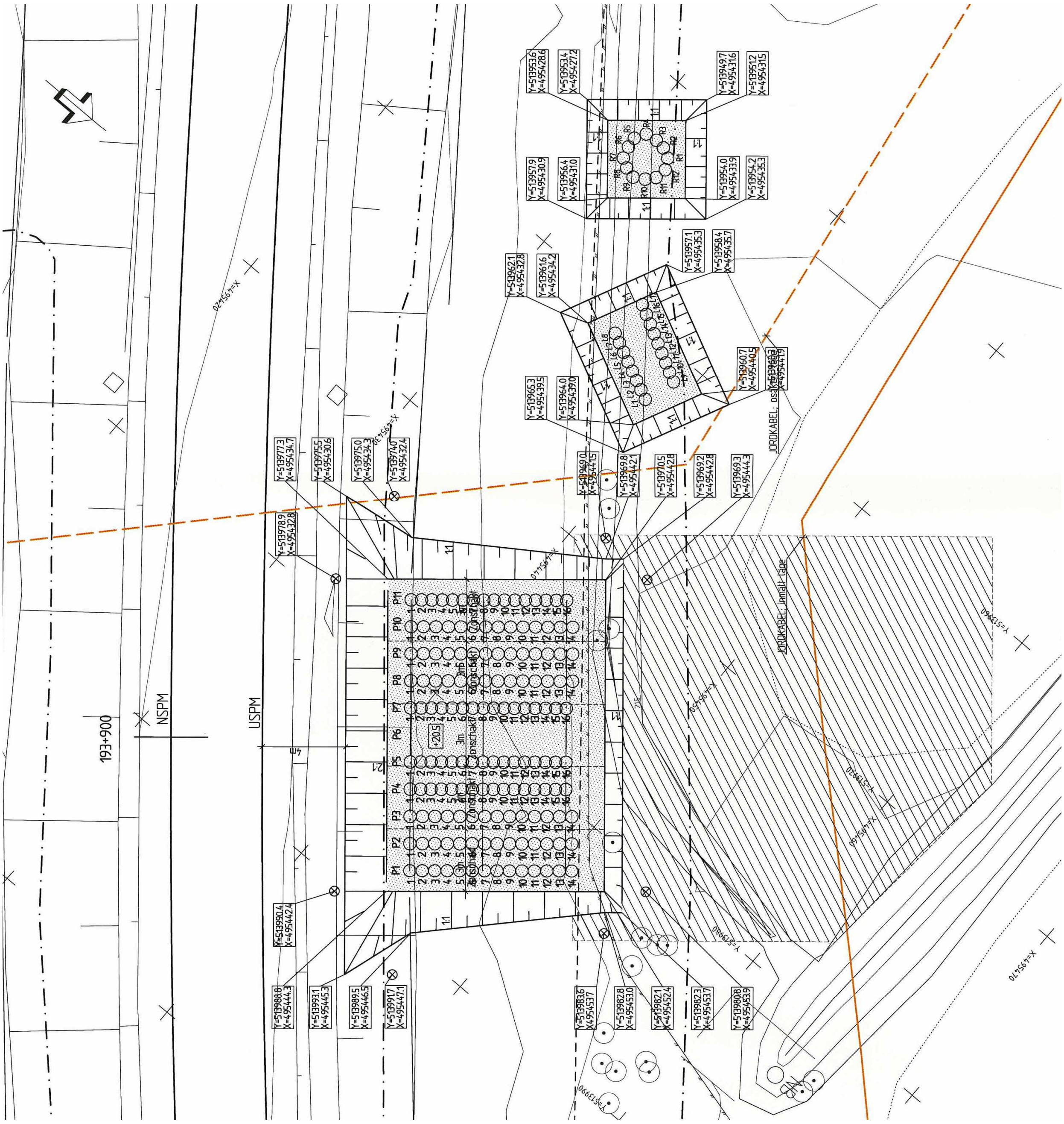


Figure 68 – Plan of the different test areas where installation of columns was performed. The excavated areas are indicated in the drawing.

3.5.3 Installation of columns

3.5.4 General

All columns were manufactured with a special mixing tool, (see Figure 56), with the aim to produce columns with a diameter of 0.60 m. The speed of rotation of the mixing tool was specified to be 150 rotations per minute and the intended lifting speed 20 mm per revolution of the mixing tool.

The horizontal centre to centre distance of the overlapping columns was to be 0.5 m according to the tender document. With an inclination of the columns of 40 degrees from the vertical axis this gives an overlap between two columns of 0.22 m. The horizontal projection of the individual inclined columns is a width of 0.60 m and a length of 0.78 m.

Because of high demands on mixing energy in the soil below the embankment and problems to distribute the binder evenly along the length of the column, the horizontal centre to centre distance below columns was after installation of column P11 increased to 0.6 m. This corresponds to an overlap between two columns of 0.14 m.

3.5.5 Ring and shorter walls – test lime cement columns

A ring with twelve vertical overlapping columns was installed with an overlap of 0.10 m. The twelve vertical columns in the ring were installed 8th July 2008.

In this area about 0.5 m of the natural soil had been excavated and replaced by a fill of sandy gravel prior to the installation of columns. The excavated area is indicated in Figure 68. In Figure 69 is a photo taken just before start of the installation of the columns in the ring.

Figure 69 – Placement of lime cement columns before start of installation of columns in the ring.

The ring was divided in three groups of different binder composition and binder content with four columns of similar type in each group. Group 1 with columns named R1 to R4 were installed with a binder of lime/cement in proportion 50%/50% by weight and a binder content of 28 kg/m column. For a column diameter of 0.60 m this corresponds to a binder content of about 100 kg per m^3 of soil.

Group 2 with columns named R5 to R8 were installed using lime/cement in proportion 50%/50% by weight and in a quantity of 23 kg/m column, which for a column diameter of 0.60 m corresponds to a binder content of about 80 kg per m^3 of soil.

Group 3 with columns named R9 to R12 were installed using lime/cement in proportion 25%/75% by weight and in a quantity of 28 kg/m column, which for a column diameter of 0.60 m corresponds to a binder content of about 100 kg per m^3 of soil.

The three different compositions of binder were chosen based on earlier experience of the actual soil type at the site Torp. The length of all the columns in the ring was suggested 10 m from the ground level. The mixing was stopped at the depth of about 0.5 from the friction soil level.

A plan of the position of the installed columns is presented in Figure 68.

Testing tools for performing reversed column penetration tests were installed in column R1, R5 and R9. In columns R3 and R11 plastic tubes to be used for down-hole measurement of shear wave velocity in the stabilized soil was installed. The plastic tubes had a length of 12 m and were installed with the head of the tube located about 1.0 m above the actual ground surface level.

The other columns were to be used for traditional column penetration test (with direction top-down) about 7 days (columns R2, R6 and R10) and 28 days (columns R1, R5 and R9) after installation.

3.5.6 Inclined test walls (panels)

To obtain information about the importance of the direction of installation of the columns for the inclined walls, with inclined columns were installed. The inclination of these test panels was the same as the one planned for the production panels in the ten test panels P1-P5 and P7-P11. The length of the columns was 10 m.

In this area about 0.5 m of the original soil was excavated and replaced by a fill of sandy gravel. The excavated area is indicated in Figure 68. Figure 70 shows a photo taken just before the installation of the column starts for the two shorter panels.

The inclined columns were possible to install with a direction of the installation tool inclined forward with respect to the position of the machine, see Figure 54.

The first test column panels were installed with the machine moving in a direction forward to the next column to be installed. This column panel consists of nine columns L1-L9 with the same binder as in Group 3 above.

The second test column panel was installed with the machine in a direction backwards between each column. This column panel consists of eight columns L10-L17 with the same binder as in Group 1 above.

A total of 17 inclined columns were installed 8th July 2008.

The experience from the installation of the 17 inclined columns was that there was no significant difference between the two methods (forward or backwards). For the production columns it was decided to use the installation method backwards.

When installing the inclined columns a new method was tested to measure the inclination of the mixing rod by a measurement unit in the top of the rod. However it was found that this method did not work properly and it was not used in later tests. The inclined columns in the shorter test panels were the first inclined columns produced in the project and the general conclusion was that to succeed with a good geometry of the panels a more precise horizontal orientation of the working platform was needed.

Figure 70 – Placement of columns before the start of the installation of inclined walls in the two shorter test panels.

3.5.7 Installation of inclined lime cement walls (panels) under the embankment

A plan of the ten production panels is given in Figure 68.

After installation of the two short test panels it was decided to use the method moving backwards with the machine. After a local excavation of the embankment to produce a horizontal working platform the installation started with the column nearest to the embankment. The columns were installed with a horizontal centre to centre distance of 0.5 m. The installation depth was supposed to be down to the frictional material below the clay.

The binder was lime/cement in the proportion 50%/50% by weight. The binder content was to be 28 kg per length meter of a column (99 kg/m^3 of soil) and the installation started with that binder content. The panels P4, P7 and the first three columns in panel P5 (columns P5:1, P5:2 and P5:3) were installed with the binder content 28 kg per m column. The rest of columns in the panels (panels P1 to P3 and P8 to P11) were installed with a binder content of 23 kg per meter of a column (81 kg per m^3 of soil). The horizontal centre to centre distance between columns was 0.50 m for panels P4, P5, P7 and P11. The rest of the panels (panels P1 to P3 and P8 to P10) were installed with a centre to centre distance of 0.60 m. The panels were installed in the order P4, P7, P5, P11, P1, P8, P2, P9, P3 and P10. Figure 71 shows the line of lime cement columns creating inclined strengthening wall.

Figure 71 – Row of columns before the start of installation of panel P4 under the embankment.

Panel P4

The installation of the columns in panel P4 was performed 8th July 2008 at the end of the day (18:30 to 21:30). When installing columns P4:3 a large boulder was hit and the mixing tool was lost at 3 m installation depth. Later on it was recovered by excavation. For the column P4:4 there was a stop when the tool stopped on a boulder at 1.8 m installation depth. This boulder was excavated directly and the fill adjusted for the further installations. These were the only boulders missed by the first excavation. Typically it was at the first installed panel. A total of 16 columns were installed in this panel. After the installation was finished the local excavation at the toe of the embankment was refilled and adjusted.

Panel P7

The installation of the columns in panel P7 was performed 9th July 2008 (10:00 to 14:00). When installing columns P7:13 there was a part of about 2 m of the installed ground water pipe at the reference station km 193+900 surrounding the installation tool. When installing the next columns in the panel, clay was coming out from the ground water pipe making it obvious that the tool had hit and cut the pipe. Installation of plastic tubes for the geophysical measurement was performed in column P7:4 and P7:16. A total of columns were installed in this panel. After the installation was finished the local excavation at the toe of the embankment was refilled and adjusted.

Panel P5

The installation of the columns in panel P5 was performed 9th July 2008 (15:30 to 19:30). After installation of column P5:3 it was decided to use a somewhat lower binder quantity of 23 kg per meter of column (81 kg per m^3 of soil). There were problems keeping an even distribution of binder along the whole length of the column caused by the clay having clearly higher strength than the original soil just outside the pressure berm. Installation of plastic tubes for the geophysical measurement was performed in column P5:4 and P5:16. A total of 16 columns were installed in this panel. After the installation was finished the local excavation at the toe of the embankment was refilled and adjusted.

Panel P11

The installation of the first 7 columns (P11:1 to P11:7) in panel P11 was performed 9th July 2008 (20:00 to 21:30). The rest of the columns in the panel P11 were installed 10th July 2008 (09:00 to 10:30). Installation of plastic tubes for the geophysical measurement was performed in column P11:5 and P11:16. A total of 16 columns were installed in this panel. After the installation was finished the local excavation at the toe of the embankment was refilled and adjusted.

Panel P1

The installation of the columns in panel P1 was performed 10th July 2008 (13:00 to 16:00). There were continued problems keeping an even distribution of binder with length even after the change to a smaller amount of binder. After installation of panel 11 it was decided to increase the horizontal centre to centre distance between columns from 0.50 m to 0.60 m. Change of overlapping was due to difficulties of mixing tool to produce homogenous columns. Installation of plastic tubes for the geophysical measurement was performed in column P1:4 and P1:14. A total of 14 columns were installed in this panel. After the installation was finished the local excavation at the toe of the embankment was refilled and adjusted.

Panel P8

The installation of the columns in panel P8 was performed 10th July 2008 (16:30 to 19:30). After the installation of column P8:4 the heave of the western track was in the order of the maximum allowable track movement of 4 mm. As the installation continued it was decided to perform measurements next morning and take action if the heave had not decreased. Installation of the tool for the reversed penetration test was performed in column P8:5 and P8:13. The inclination of this tool was the same as for the inclined column. A total of 14 columns were installed in this panel. After the installation was finished the local excavation at the toe of the embankment was refilled and adjusted.

Panel P2

The installation of the columns in panel P2 was performed 11th July 2008 (09:30 to 12:30). The heave of the track had decreased over the night and the installation continued. After the installation of column P2:4 the heave of the western track was 6 mm which was larger than the allowable track movement of 4 mm. After discussion with Banverket the installation continued and it was decided to perform more frequent measurements of track movements. Installation of the tool for the sounding method down-up was performed in column P2:1 and P2:3. The direction of these two tools was vertical. A total of 14 columns were installed in this panel. After the installations were finished the local excavation at the toe of the embankment was refilled and adjusted.

Panel P9

The installation of the columns in panel P9 was performed 11th July 2008 (14:30 to 17:30). Installation of the tool for the reversed penetration test was performed in column P9:1 and P9:3. The direction of these two tools was vertical. A total of 14 columns were installed in this panel. After the installations were finished the local excavation at the toe of the embankment was refilled and corrected.

Panel P3

The installation of the columns in panel P3 was performed 14th July 2008 (09:30 to 12:30). For columns P3:3, P3:10 and P3:14 there were indications of larger objects (stones or boulders) in the soil that caused movements of the mixing rod when installing the column. Installation of plastic tubes for the geophysical measurement was performed in column P3:4 and P3:14. A total of 14 columns were installed in this panel. After the installations were finished the local excavation at the toe of the embankment was refilled and adjusted.

Panel P10

The installation of the columns in panel P10 was performed 14th July 2008 (14:00 to 15:30). For columns P10:4, P10:5 and P10:11 there were indications of larger objects (stones or boulders) in the soil that caused movements of the mixing rod when installing the column. Installation of plastic tubes for the geophysical measurements was performed in column P3:4 and P3:14. Total 14 columns were installed in this panel. After the installations were finished the local excavation at the toe of the embankment was refilled and adjusted.

The overview of construction site with installation machine under production of inclined lime cement walls to strengthen subsoil under railway operated embankment can be seen in the Figure 72.

Figure 72 – View over construction site for installation of inclined lime cement walls at Torp

3.5.8 Adjustment of pressure berm

An overall readjustment of the geometry of the area after performed works in the autumn was demanded by the project for the future.

In the end of December 2008 an adjustment of the area was performed. The adjustment was made with a 0.5 m fill in the area with the 10 panels and a similar adjustment was made of the ground surface in the test area with the 17 inclined columns were the ground surface was clearly lower than the surrounding area.

In Figure 73 to 74 are photos of the area where panels were installed and situation after the adjustment of the pressure berms in December 2008.

Figure 73 – After readjustment of installation area. Photo taken against south 2009-01-27.

Figure 74 – After readjustment of installation area. Photo taken against North 2009-01-27.

3.6 In-Situ tests in stabilised and natural soil

3.6.1 General

A vital part in the design of a stabilization measure is to control the properties of the stabilized soil in-situ. In this case penetration tests have been performed at different times after installation and geophysical measurement have been performed in the middle of September 2008 and in the beginning of October 2008.

The penetration tests have been performed as traditional column penetration tests with the direction top-down and also as reversed tests with the direction down-up. The tests have been performed at different times after installation to get indications of the change in strength with time. Normally the tests are performed about 28 days after the installation of columns.

The geophysical measurement was performed by Håkan Mattson, Geovista, (Mattson, 2008) as cross-hole seismic tomography in the natural soil and in the stabilized soil.

Resistivity measurements and seismic measurements have been performed by G Impuls Prague in the natural soil and the stabilized soil (INNOTRACK SP 2, report D 2.1.5 “Methodology of geophysical investigation of railway track defects").

3.6.2 Results from sounding in columns

The penetration tests in columns were performed as a traditional column penetration tests with the direction down (starts at top of column and with direction down) and as reversed tests with the direction up (starts below column and with direction up).

For the penetration test with direction up the tool need to be installed during or directly after installation of the column. In this case three tools in the ring area and six tools in the area with the ten production panels were installed.

For the penetration test with direction down a steering hole is created by first performing a CPT-test. Directly afterwards the column penetration test was performed.

The tools used for both types of penetration tests have a thickness of 15 mm and a width of 500 mm for all tests except for column R10. For test in column R10 a tool with a thickness of 20 mm and a width of 400 mm was used.

Empirically the relation between the undrained shear strength of the stabilized soil and the corrected penetration resistance, for the tools used at Torp, is about 10-12 kPa per kN for the test performed with direction down and 15 kPa per kN for the test performed with direction up.

The penetration resistance should be corrected for the shaft resistance and is normally done by using the part of the test that is performed in the not stabilized soil.

Direction up

The tests in the ring columns R1, R5 and R9 were performed about 27 days after installation. The results from these tests are presented in Figure 75 to 77.

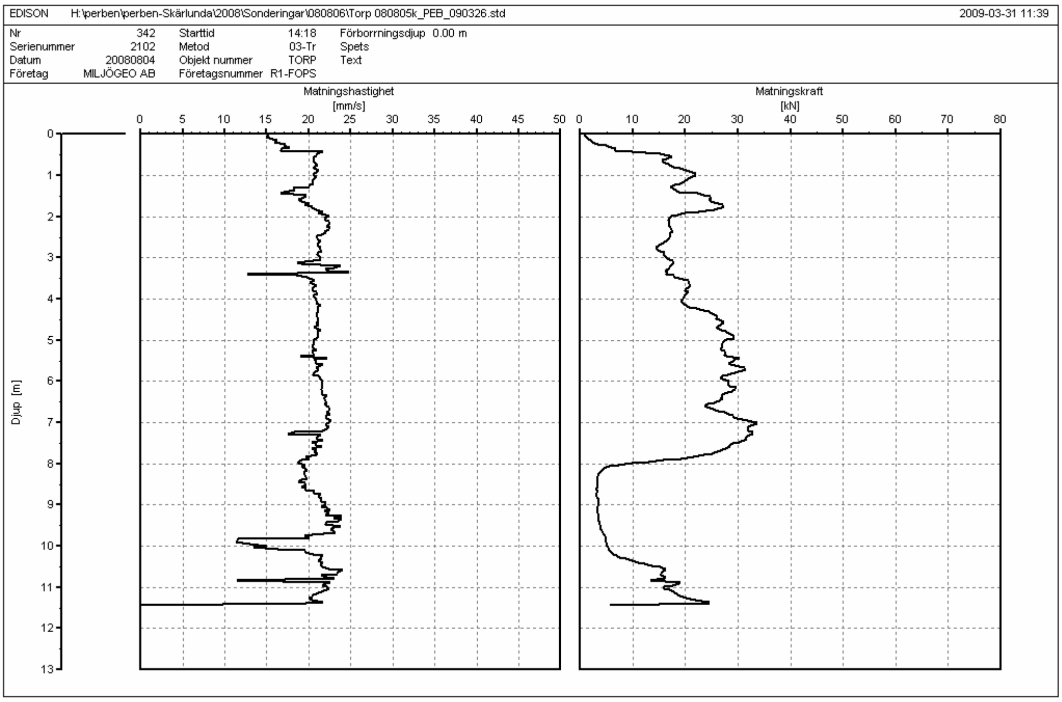


Figure 75 – Results from column penetration test with direction up in Column R1. Performed 27 days after installation. (Binder L/C 50%/50% 28 kg/m).

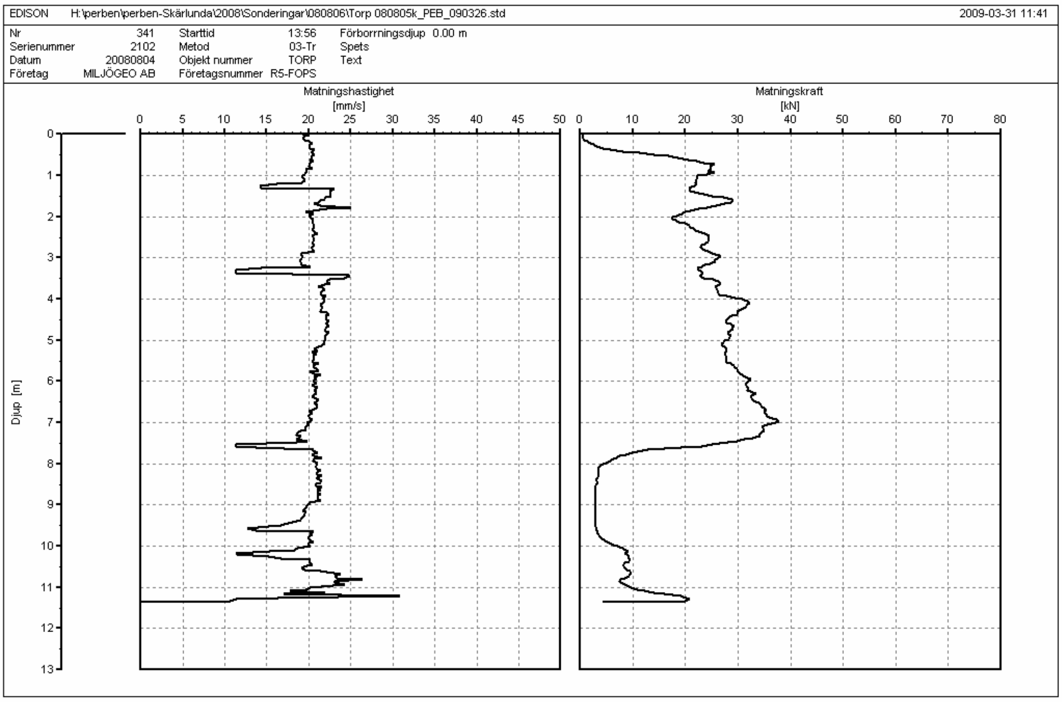


Figure 76 – Results from column penetration test with direction up in Column R5. Performed 27 days after installation. (Binder L/C 50%/50% 23 kg/m).

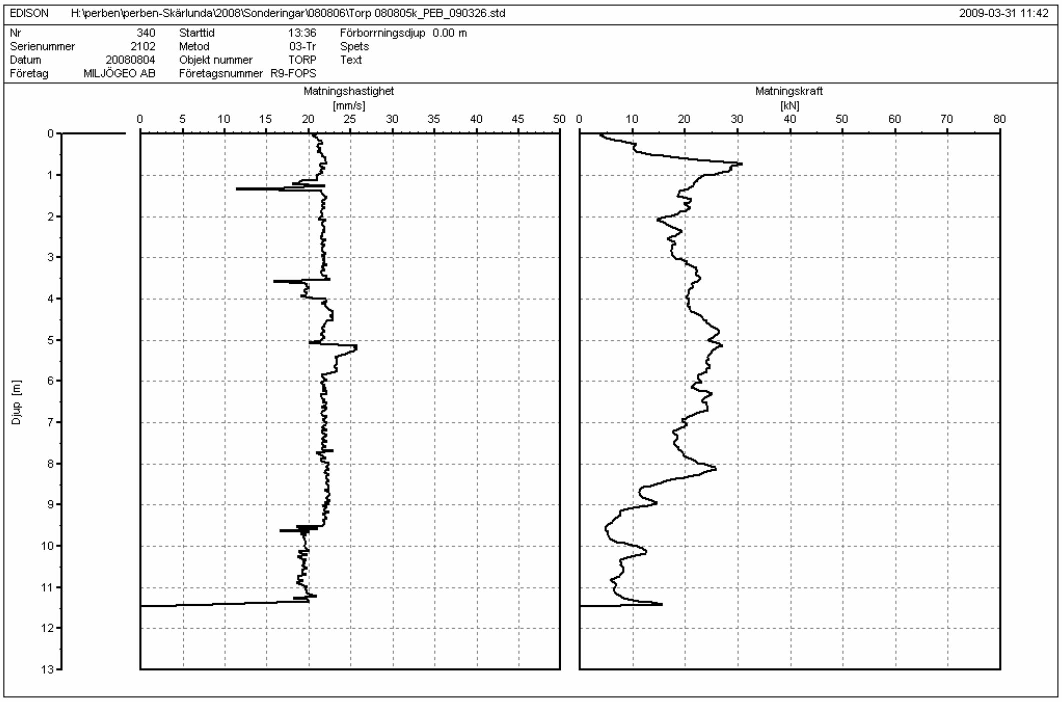


Figure 77 – Results from column penetration tests with direction up in Column R9. Performed 27 days after installation. (Binder L/C 25%/75% 28 kg/m).

In the panels the tests were performed in the column

- P2:1 direction vertical, 6 days after installation
- P8:5 inclined 40 degree to the vertical, 7 days after installation
- P2:2 direction vertical, 13 days after installation
- P9:1 direction vertical, 13 days after installation
- P9:4 direction vertical, 13 days after installation
- P8:14 inclined 40 degree to the vertical, 14 days after installation

Results from these tests are presented in Figure 78 to 83.

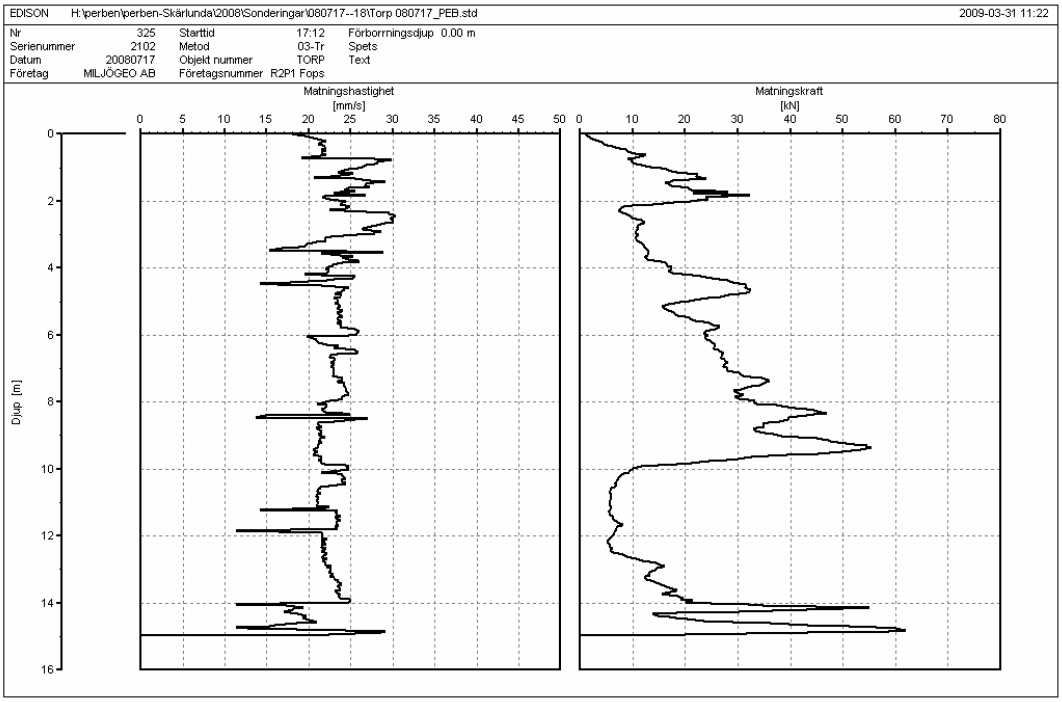


Figure 78 – Results from column penetration test with direction up in Column P2:1. Performed 6 days after installation. Vertical. (Binder L/C 50%/50% 23 kg/m).

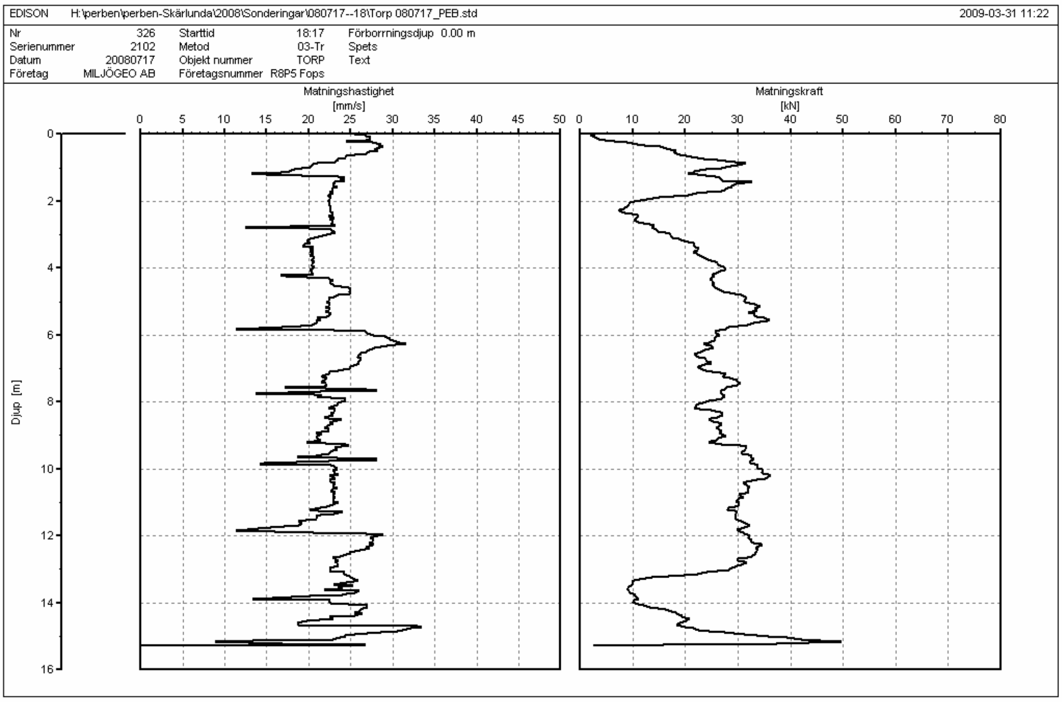


Figure 79 – Results from column penetration test with direction up in Column P8:5. Performed 7 days after installation. Inclined 40 degree to vertical. (Binder L/C 50%/50% 23 kg/m).

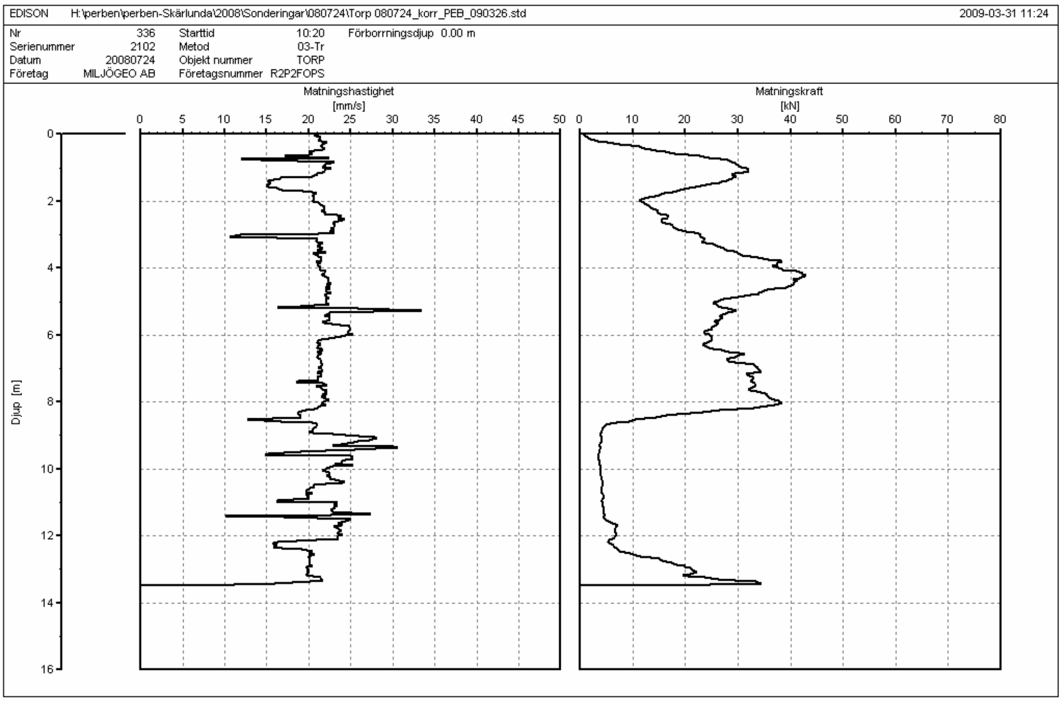


Figure 80 – Results from column penetration test with direction up in Column P2:2 Performed 13 days after installation. Vertical. (Binder L/C 50%/50% 23 kg/m).

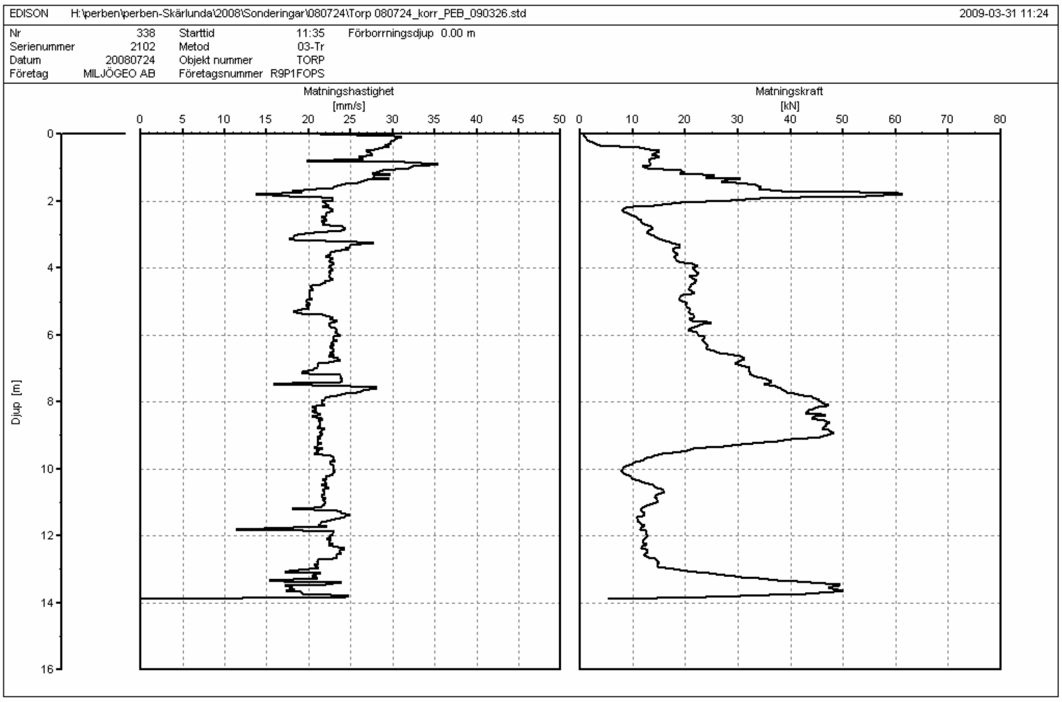


Figure 81 – Results from column penetration test with direction up in Column P9:1. Performed 13 days after installation. Vertical (Binder L/C 50%/50% 23 kg/m).

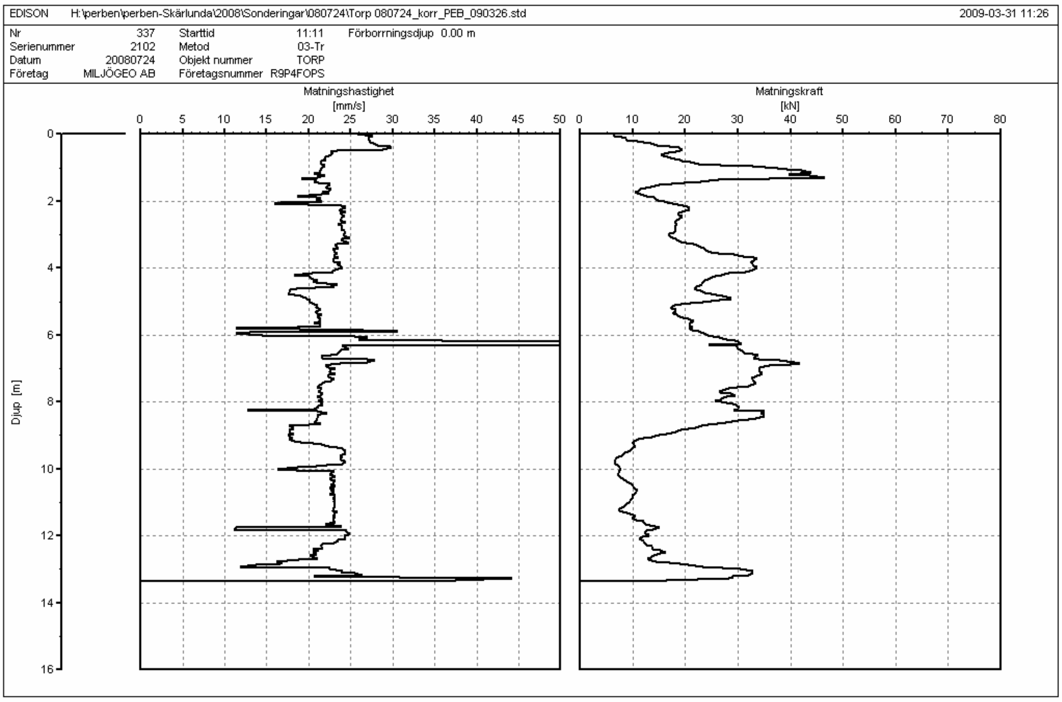


Figure 82 – Results from column penetration test with direction up in Column P9:4. Performed 13 days after installation. Vertical. (Binder L/C 50%/50% 23 kg/m).

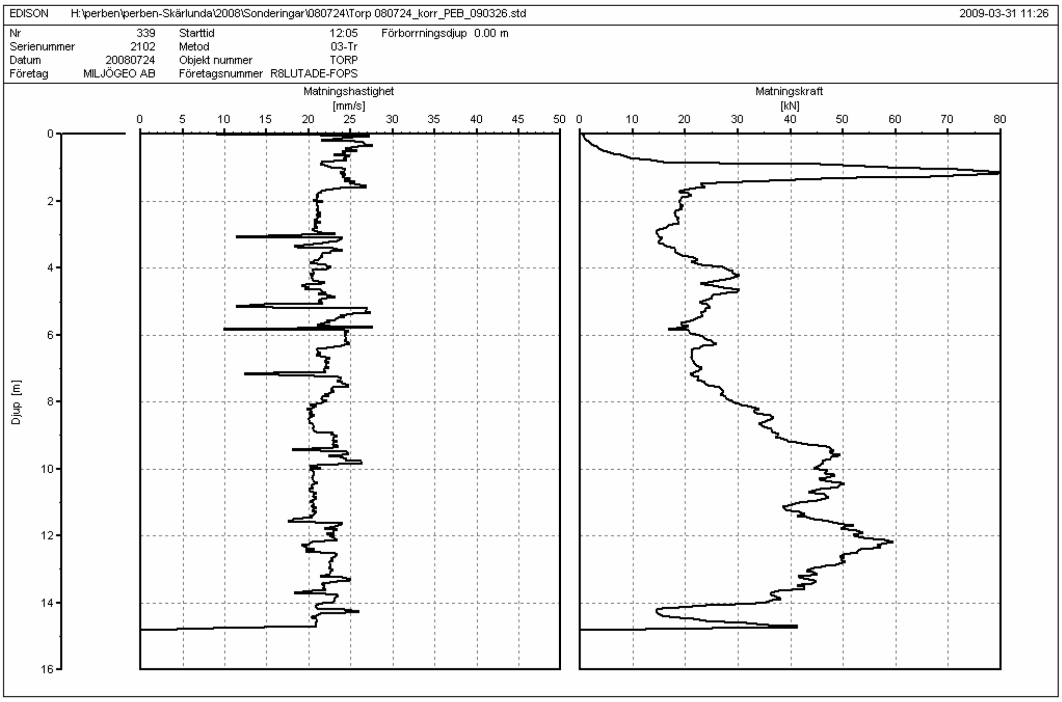


Figure 83 – Results from column penetration test with direction up in Column P8:14. Performed 14 days after installation. Inclined 40 degree to vertical. (Binder L/C 50%/50% 23 kg/m).

The results form the penetration tests with direction up indicates that shear strengths are higher than those used in design. In design an undrained shear strength of 50 kPa was used for the upper organic clay and 100 kPa for the clay.

Direction down

The tests in the ring in the columns R2, R6 and R10 were performed 9 days after installation and in columns R4, R8 and R12 about 28 days after installation. Results from these tests are presented in Figures 84 to 89.

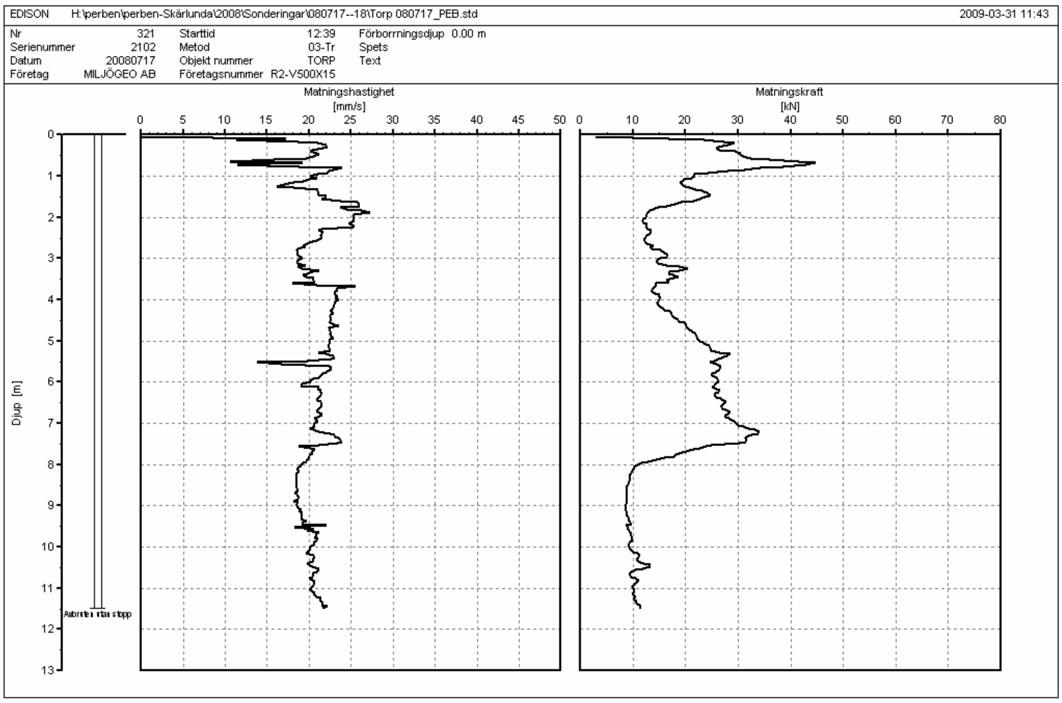


Figure 84 – Results from column penetration test with direction down in Column R2. Performed 9 days after installation. (Binder L/C 50%/50% 28 kg/m).

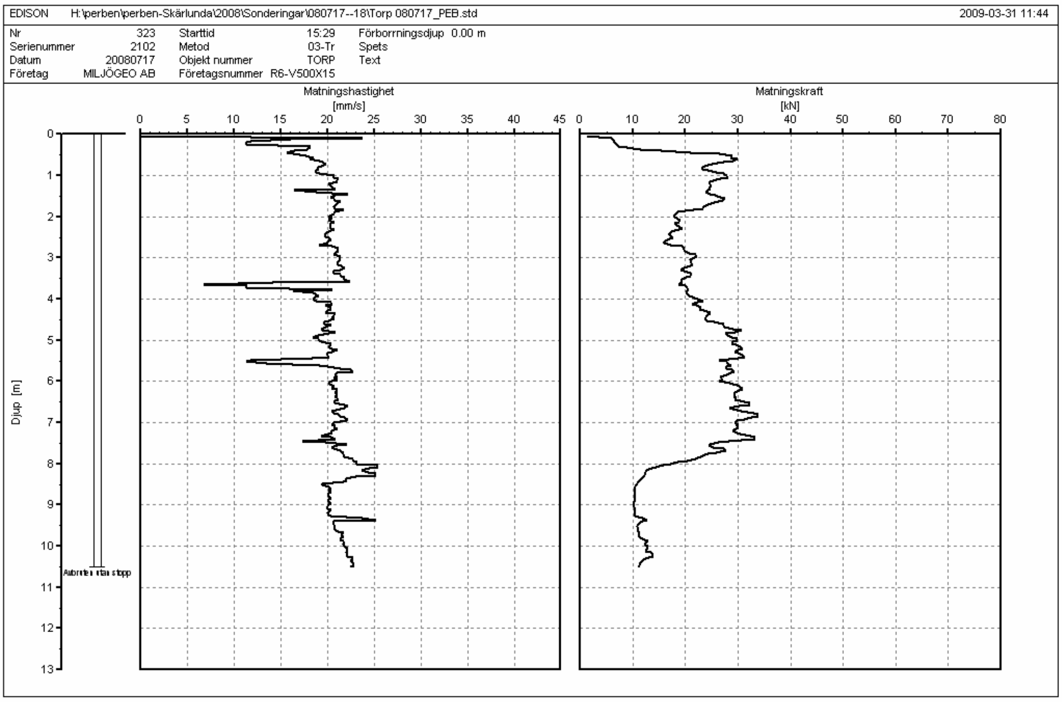


Figure 85 – Results from column penetration test with direction down in Column R6. Performed 9 days after installation. (Binder L/C 50%/50% 23 kg/m).

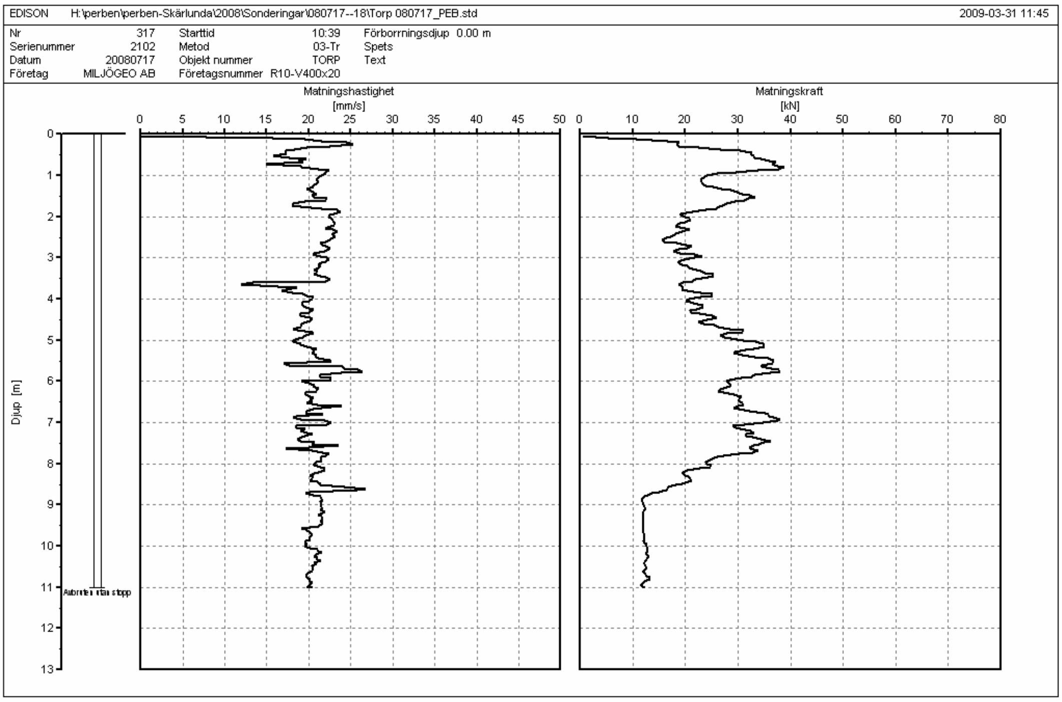


Figure 86 – Results from column penetration test with direction down in Column R10. Performed 9 days after installation. (Binder L/C 25%/75% 28 kg/m).

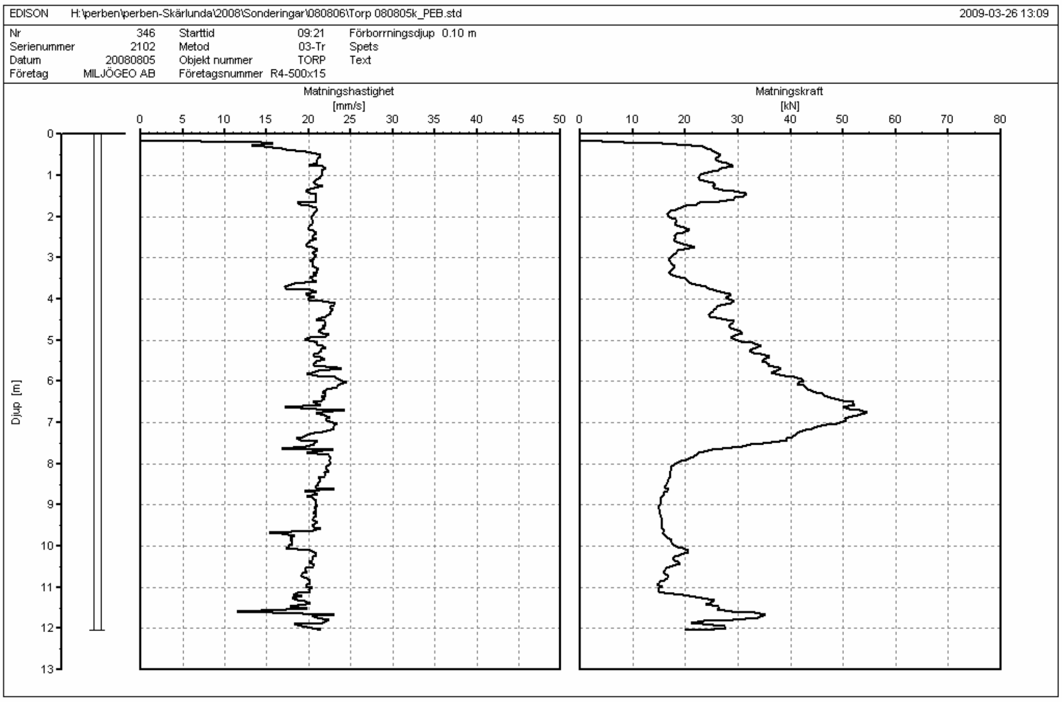


Figure 87 – Results from column penetration test with direction down in Column R4. Performed 28 days after installation. (Binder L/C 50%/50% 28 kg/m).

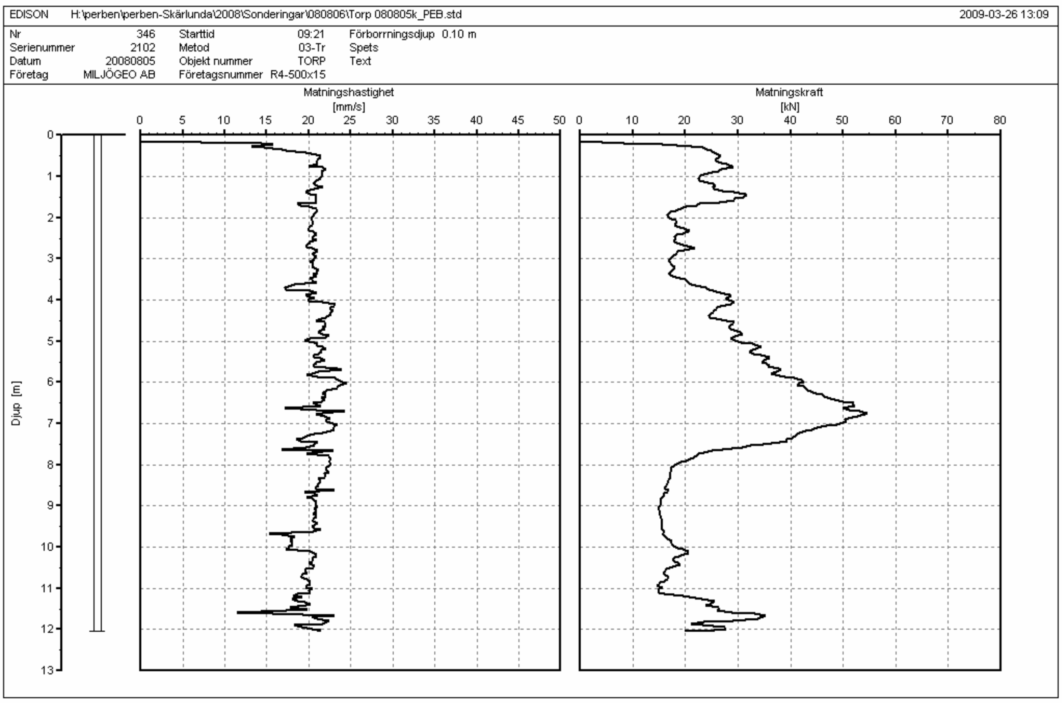


Figure 88 – Results from column penetration test with direction down in Column R8. Performed 28 days after installation. (Binder L/C 50%/50% 23 kg/m).

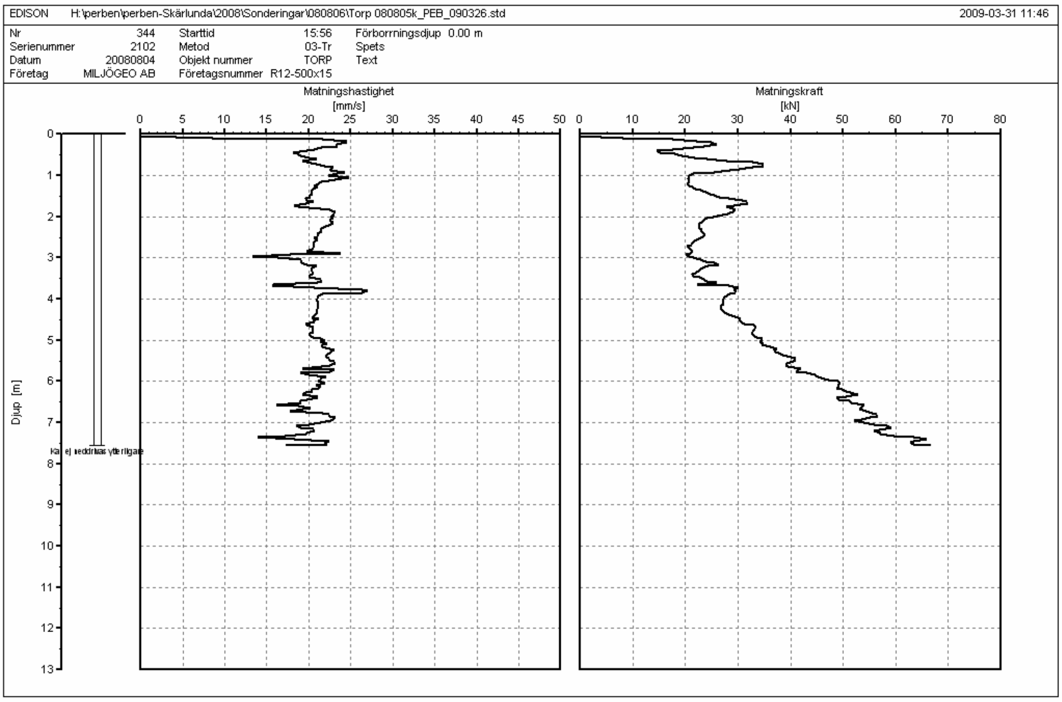


Figure 89 – Results from column penetration test with direction down in Column R12. Performed 27 days after installation. (Binder L/C 25%/75% 28 kg/m)

In the panels the tests were performed in the columns:

- P7:7 inclined 40 degree to the vertical, 5 hours after installation
- P10:5 direction vertical, 4 days after installation
- P10:5 inclined 20 degree to the vertical, 4 days after installation
- P4:5 direction vertical, 10 days after installation
- P4:5 inclined 20 degree to the vertical, 10 days after installation
- P2:5 inclined 20 degree to the vertical, 26 days after installation
- P9:5 inclined 20 degree to the vertical, 26 days after installation

Results from these tests are presented in Figures 90 to 96.

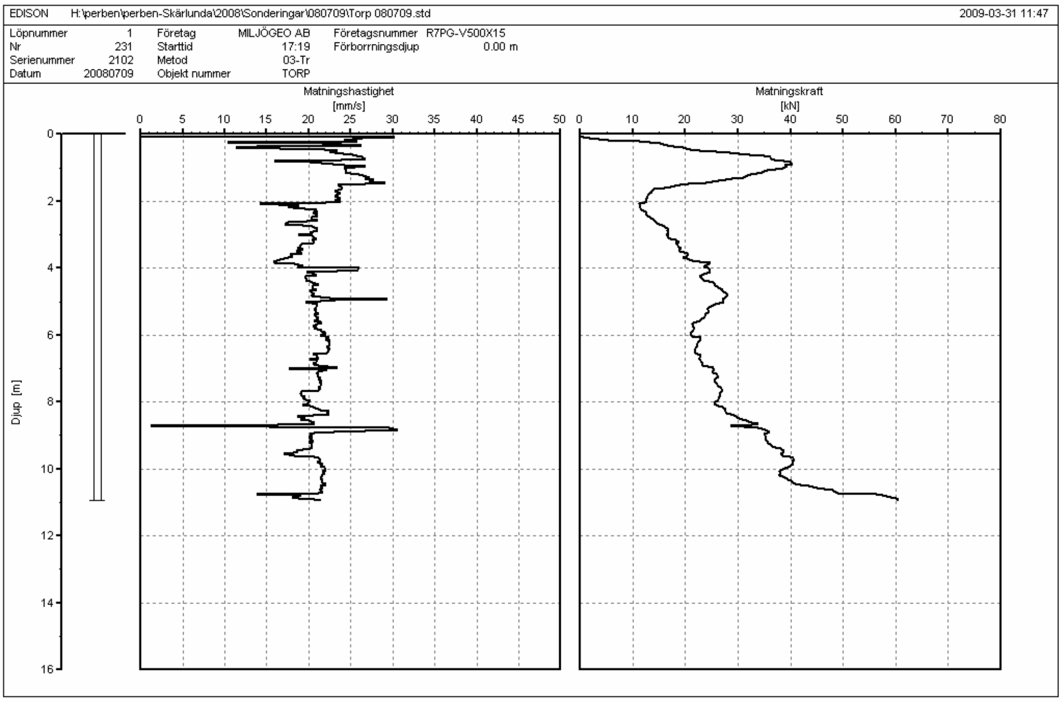


Figure 90 – Results from column penetration test with direction down in Column P7:7. Performed 5 hours after installation. Inclined 40 degree to the vertical axis. (Binder L/C 50%/50% 28 kg/m).

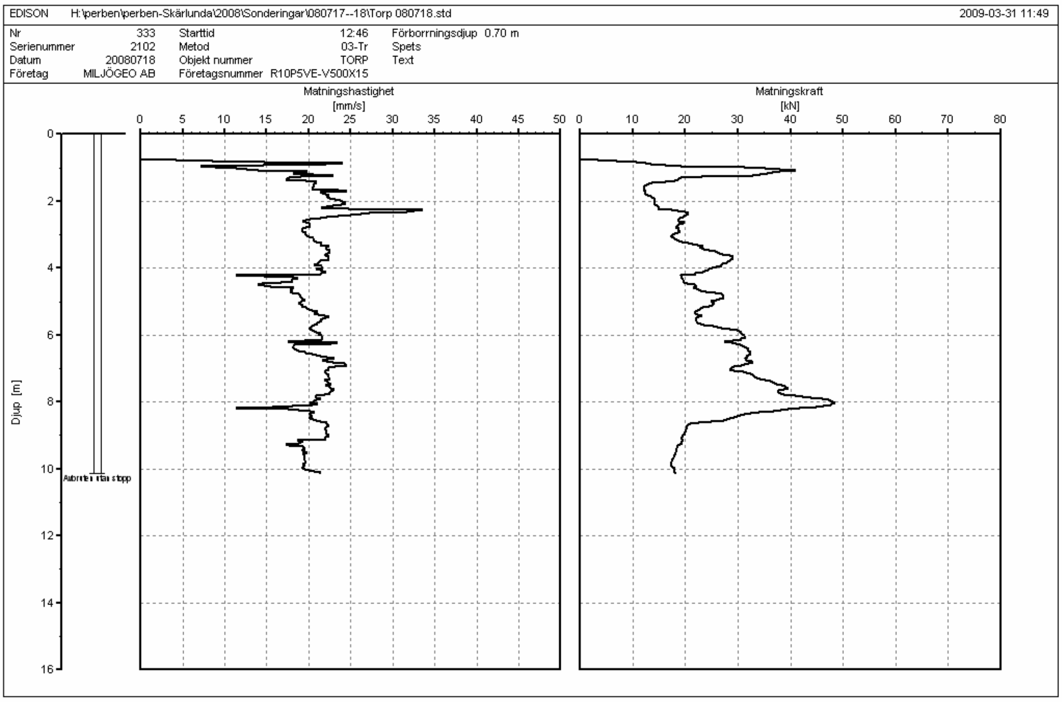


Figure 91 – Results from column penetration test with direction down in Column P10:5. Performed 4 days after installation. Vertical. (Binder L/C 50%/50% 23 kg/m).

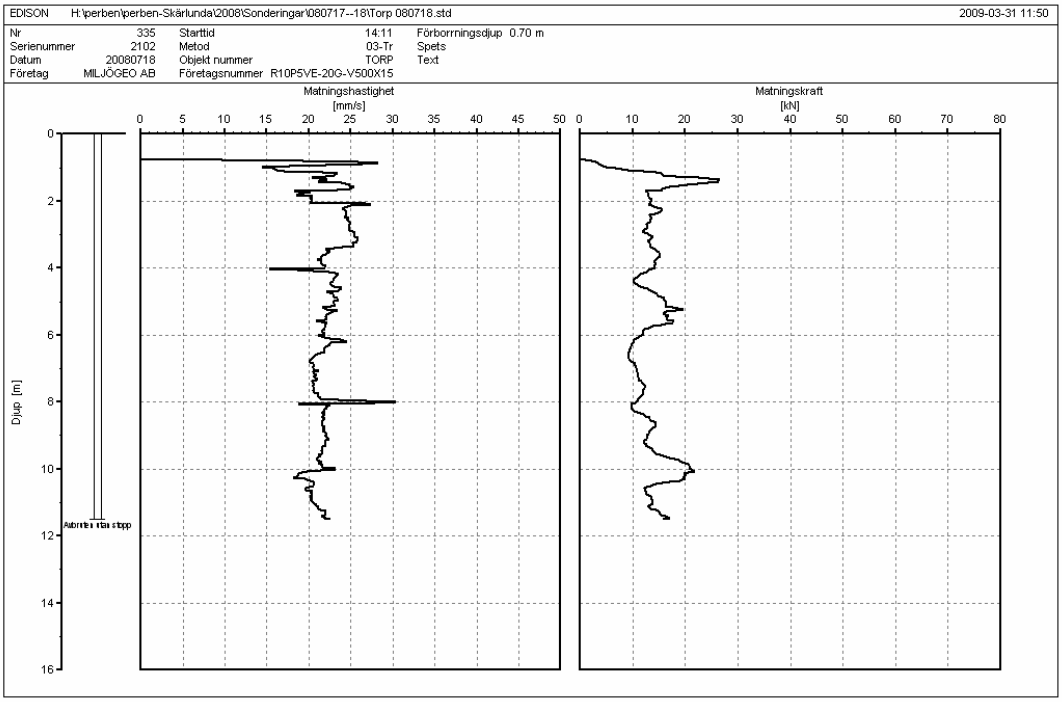


Figure 92 – Results from column penetration test with direction down in Column P10:5 Performed 4 days after installation. Inclined 20 degree to vertical. (Binder L/C 50%/50% 23 kg/m).

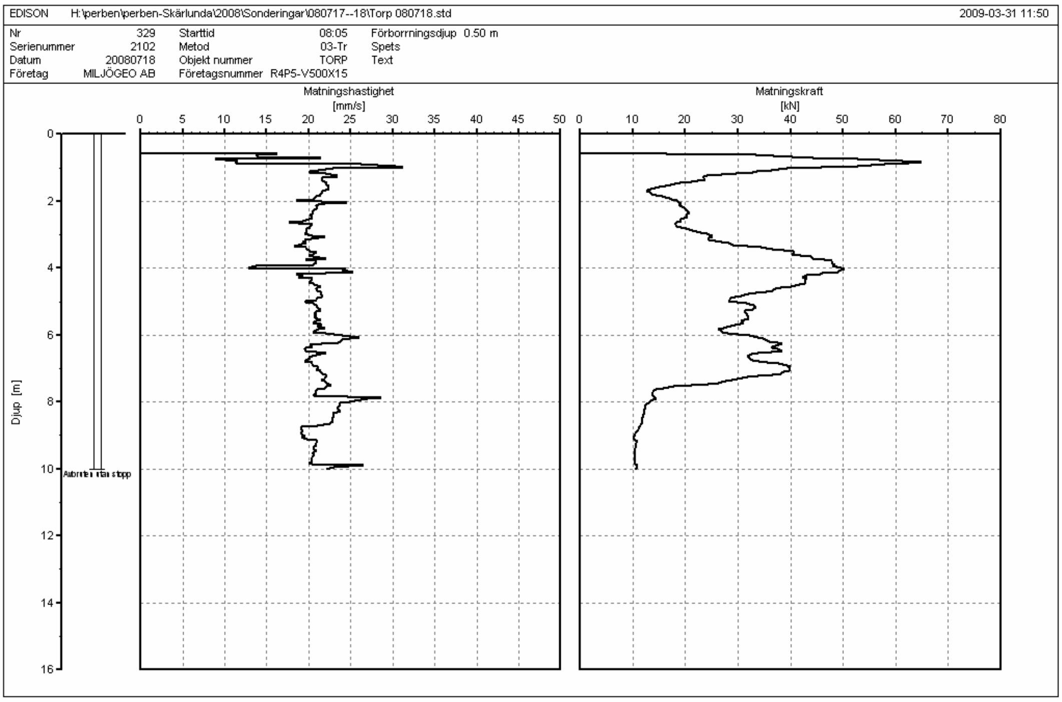


Figure 93 – Results from column penetration test with direction down in Column P4:5. Performed 10 days after installation. Vertical. (Binder L/C 50%/50% 28 kg/m).

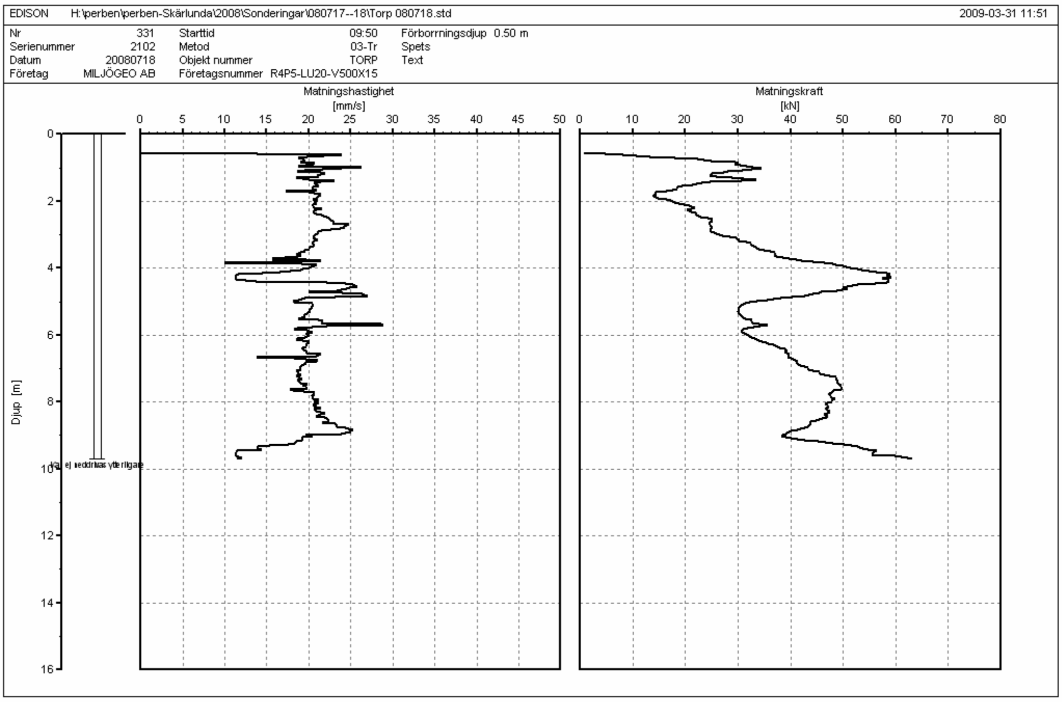


Figure 94 – Results from column penetration test with direction down in Column P4:5. Performed 10 days after installation. Inclined 20 degree to vertical. (Binder L/C 50%/50% 28 kg/m).

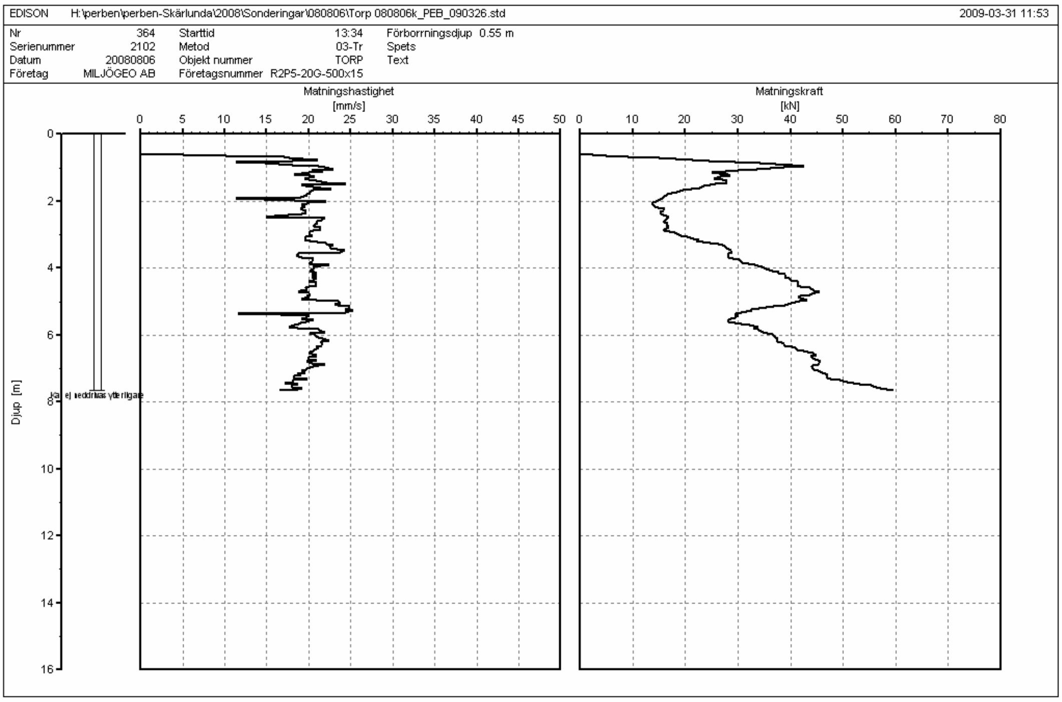


Figure 95 – Results from column penetration test with direction down in Column P2:5. Performed 26 days after installation. Inclined 20 degree to vertical. (Binder L/C 50%/50% 23 kg/m).

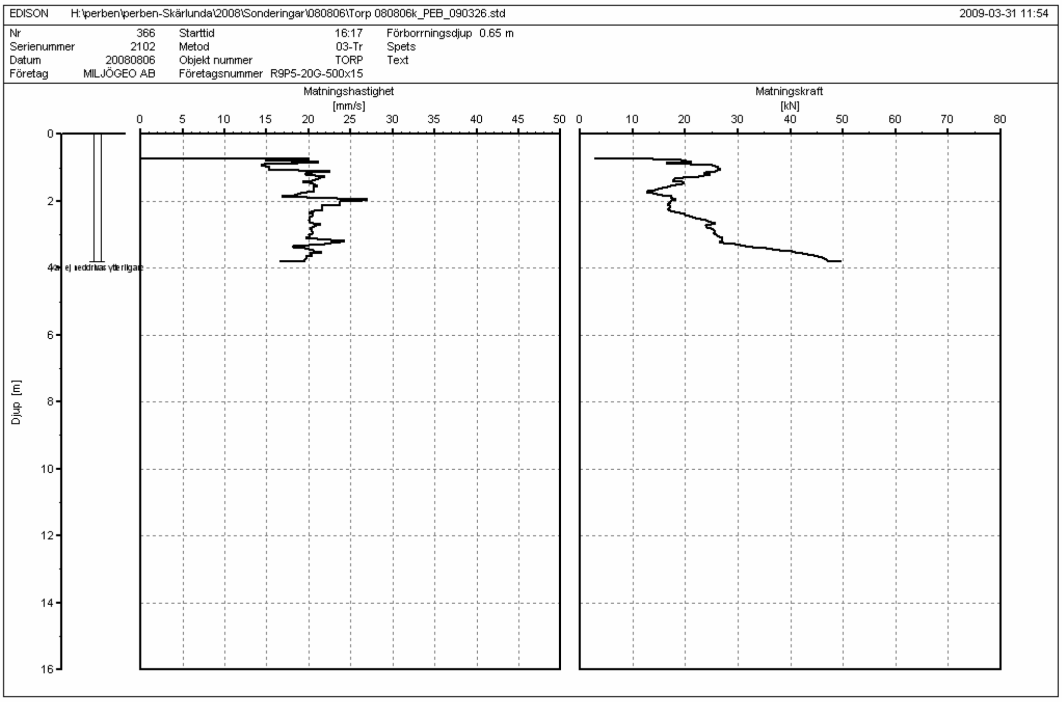


Figure 96 – Results from column penetration test with direction down in Column P9:5. Performed 26 days after installation. Inclined 20 degree to vertical. (Binder L/C 50%/50% 23 kg/m).

The results form the penetration tests with the direction down indicates that shear strengths are higher than used in the design. In the design an undrained shear strength of 50 kPa was used for the upper organic clay and 100 kPa for the clay.

The two shorter inclined test panels

The shorter two panels with inclined test columns were considered not well positioned. However some tests were performed in the panels and also outside the panels in the natural soil. In Figures 97 to 102 are presented the results for the column penetration tests (direction down) in the columns.

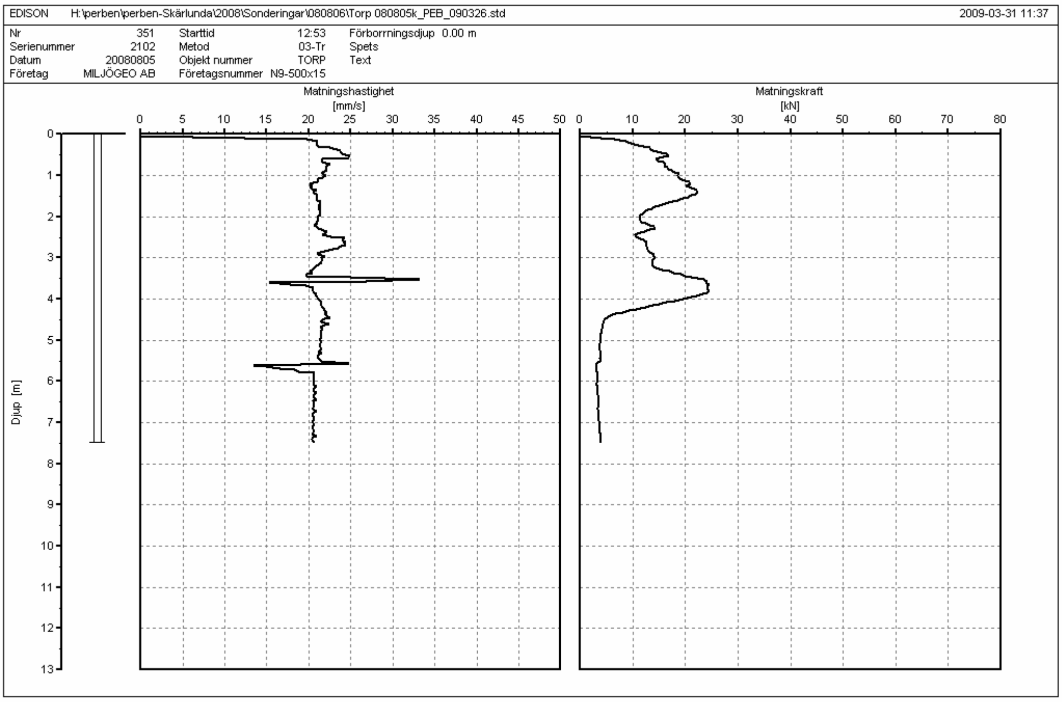


Figure 97 – Results from column penetration test with direction down in Column L11. Performed 28 days after installation. Inclined 40 degree to vertical. (Binder L/C 50%/50% 28 kg/m).

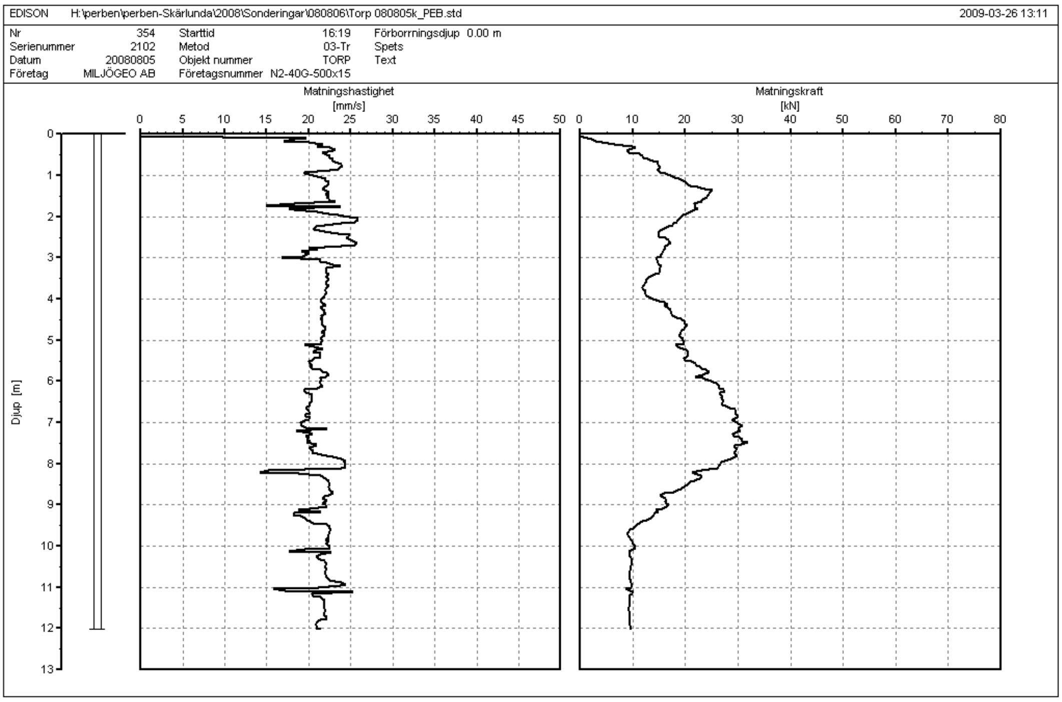


Figure 98 – Results from column penetration test with direction down in Column L16. Performed 28 days after installation. Inclined 40 degree to vertical. (Binder L/C 50%/50% 28 kg/m).

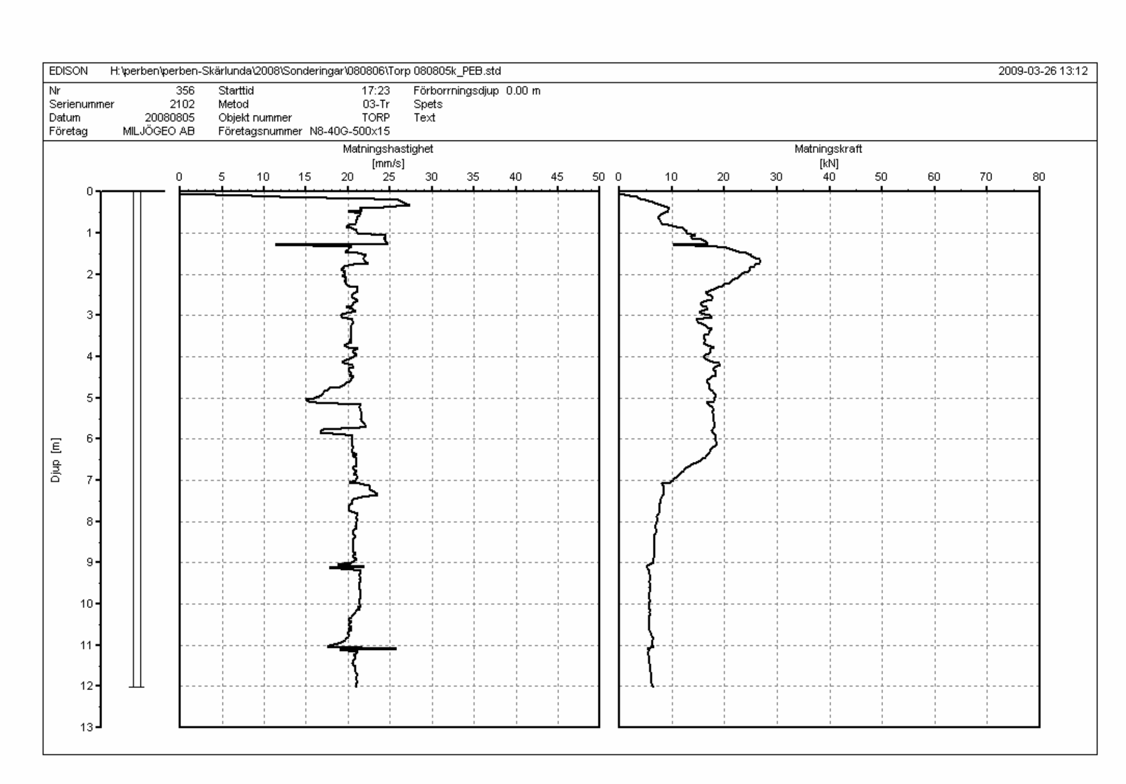


Figure 99 – Results from column penetration test with direction down in Column L12. Performed 28 days after installation. Inclined 40 degree to vertical. (Binder L/C 50%/50% 28 kg/m).

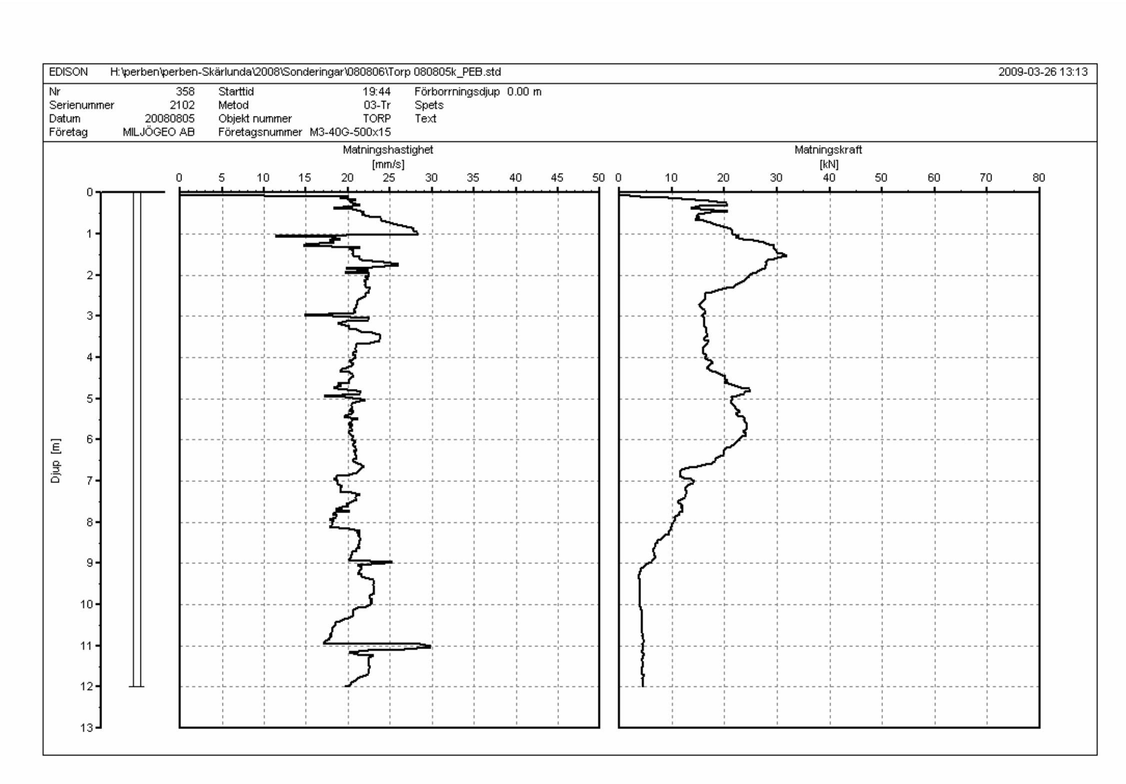


Figure 100 – Results from column penetration test with direction down in Column L3. Performed 28 days after installation. Inclined 40 degree to vertical. (Binder L/C 25%/75% 28 kg/m).

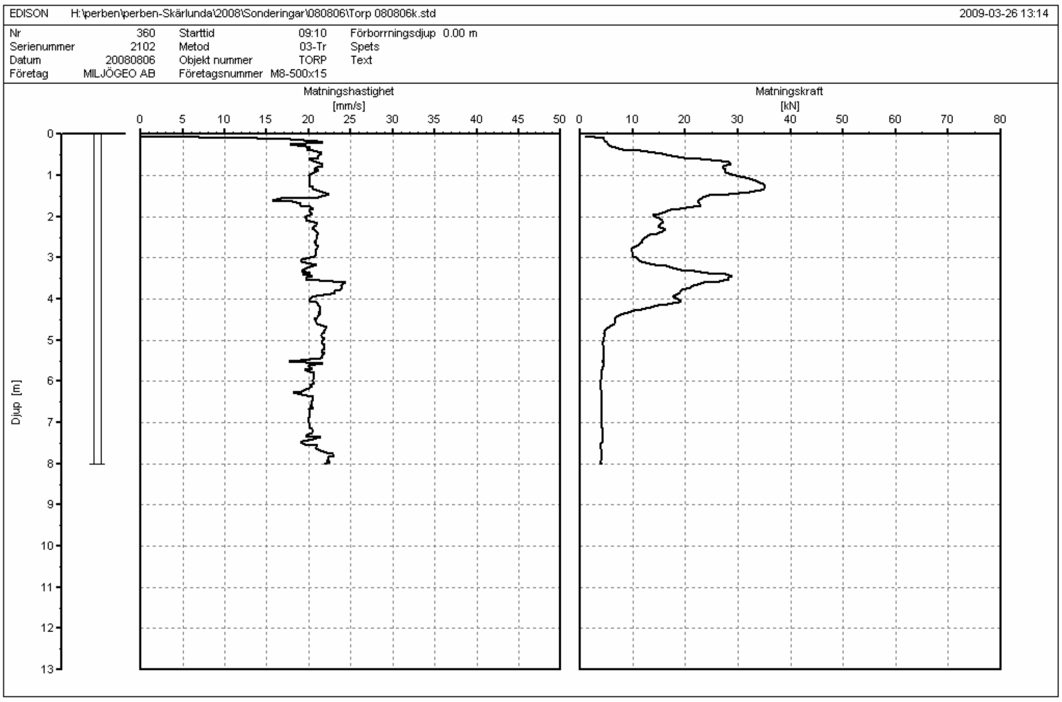


Figure 101 – Results from column penetration test with direction down in Column L8. Performed 29 days after installation. Vertical. (Binder L/C 25%/75% 28 kg/m).

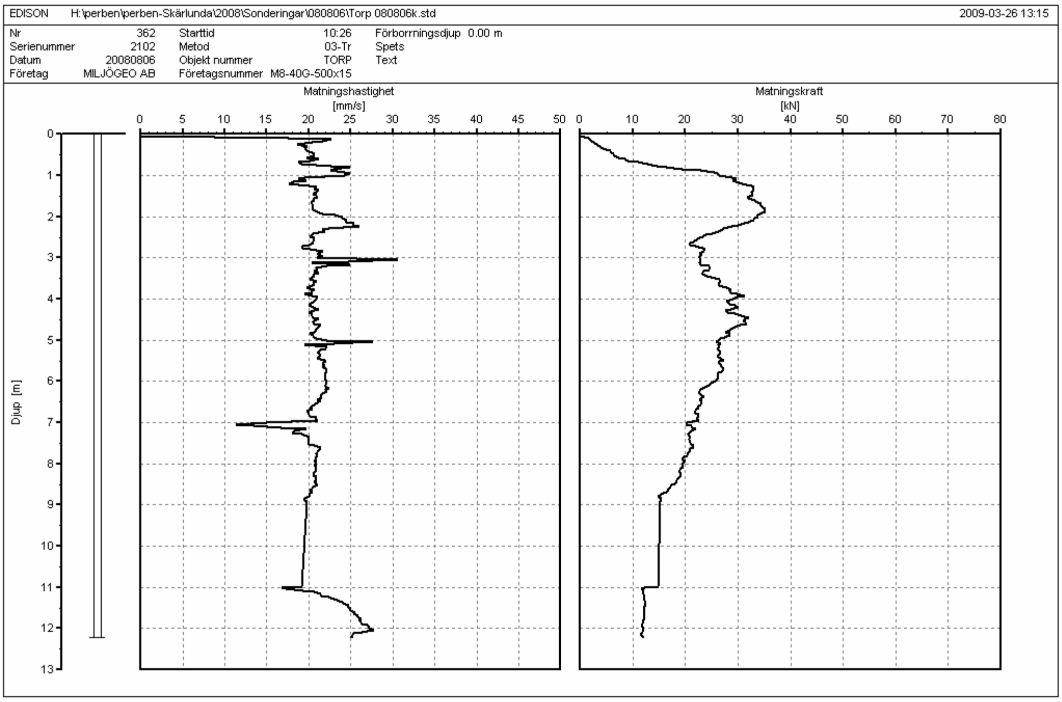


Figure 102 – Results from column penetration test with direction down in Column L8. Performed 29 days after installation. Inclined 40 degree to vertical. (Binder L/C 25%/75% 28 kg/m).

3.6.3 Results from geophysical tests

The geophysical tests were performed in cooperation with Swedish geophysical company GeoVista AB and GImpuls Prague. The results from Swedish measurements are presented in this section and results from measurements carried out by GImpuls are presented in INNOTRACK SP 2, report D 2.1.5 “Methodology of geophysical investigation of railway track defects".

The seismic measurements were performed in order to supply information regarding seismic wave distribution (compression and shear waves) and elastic properties of the soil under the railway embankment, in the natural soil beside the embankment, in five sections of inclined dry mix column slabs and also in a ring formation of dry mix columns.

Cross-hole seismic tomography

Tomography is a well-known technique in many branches of science to create images of projections (tomograms) of hidden objects by the use of X-rays, ultrasound or electromagnetic waves (tomo = slice, graph = picture).

There are different kinds of tomographic measurement techniques and what was used in this project is termed *seismic crosswell direct wave travel time tomography*. However, it is commonly called cross-hole tomography. The basic principle of the technique is to estimate a velocity model of the ground by measuring the time it takes for elastic waves to propagate from a source to a receiver. To perform seismic crosswell tomography measurements it is necessary to have two (or more) boreholes, Figure 103. An array of geophones is inserted in one hole and an elastic wave is generated in the other. A seismograph measures the time it takes for the wave to propagate from the source point to the geophones. The source is then moved to another position in the hole and the procedure is repeated. The measurements will produce a number of arrival times of waves that have crossed the investigated area. The geophone distance and the wave frequency mainly govern the data resolution; the shorter distance and the higher frequency, the better the resolution. The geometry of the investigated area, meaning the spatial relation between the depth of the boreholes and the distance between the boreholes is also an important parameter since shallow boreholes and a large distance will lead to poor ray coverage.

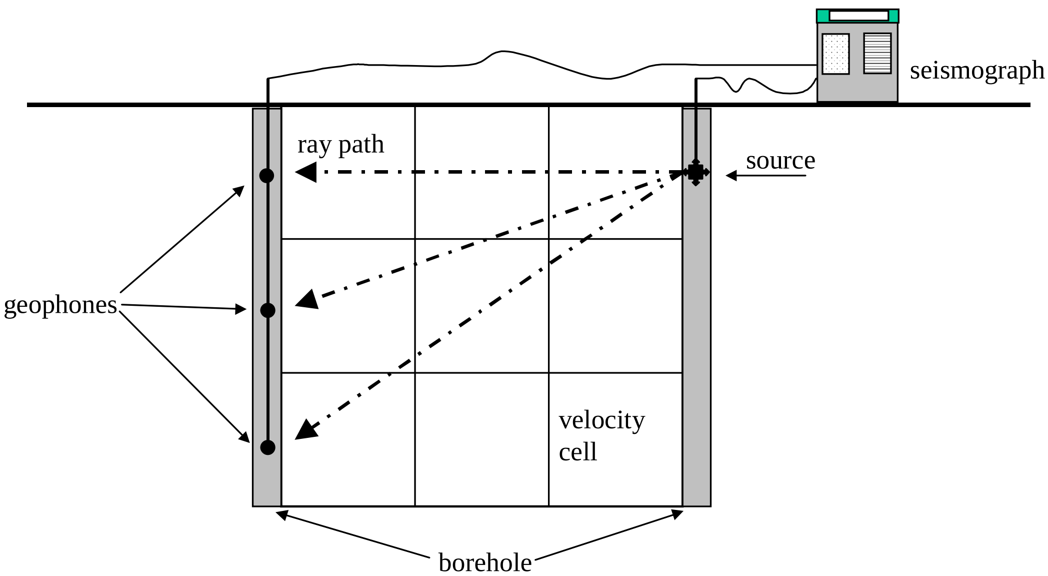


Figure 103 – Schematic picture of instrument setup and ray paths during a seismic tomography measurement.

The cross-hole tomography measurements were performed in five sections of inclined dry mix column slabs (sections P1, P3, P5, P7 and P11), in the soil beneath the railway embankment (boreholes W1 and W2) and in the natural soil (boreholes W15 and W16). In the ring formation of dry mix columns two down-hole measurements were performed (boreholes W13 and W14). Position and situation of measurements can be seen in Figures 104, 105 and 106.

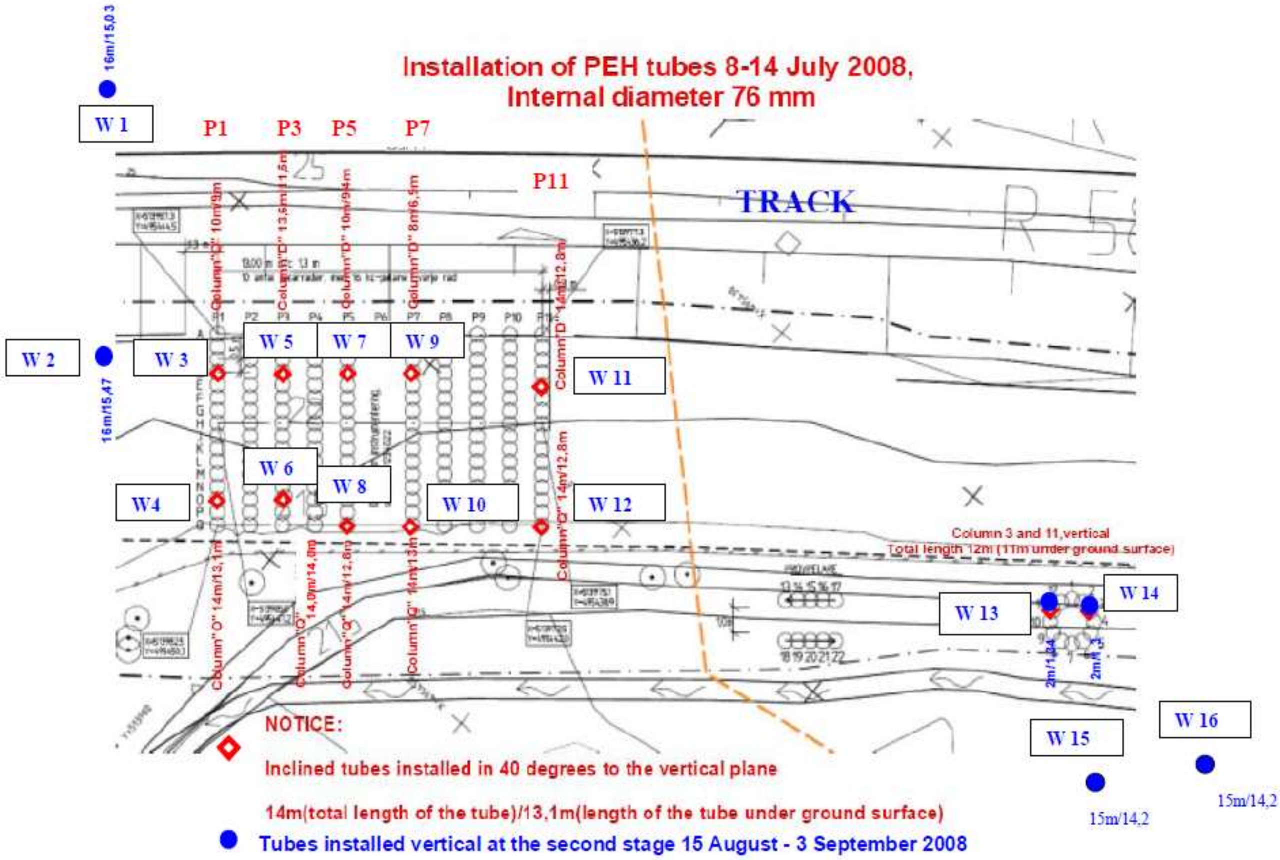


Figure 104 – Sketch showing borehole positions. The five investigated dry mix column slabs are denoted P1, P3, P5, P7 and P11. Boreholes W1 and W2 are located on either side of the embankment in soft natural soil, boreholes W15 and W16 are located in natural soil and boreholes W13 and W14 are situated in a ring of dry mix columns.

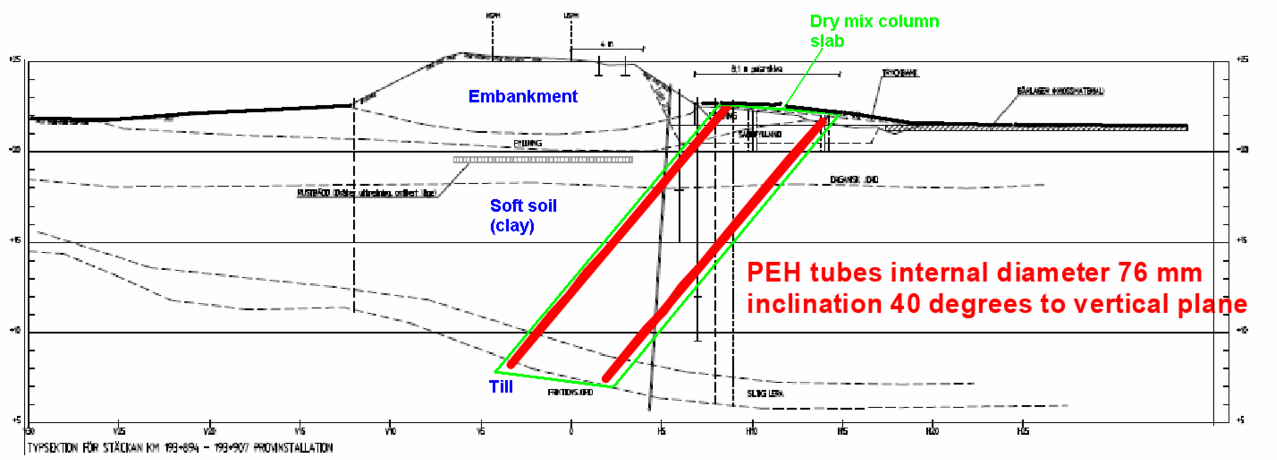
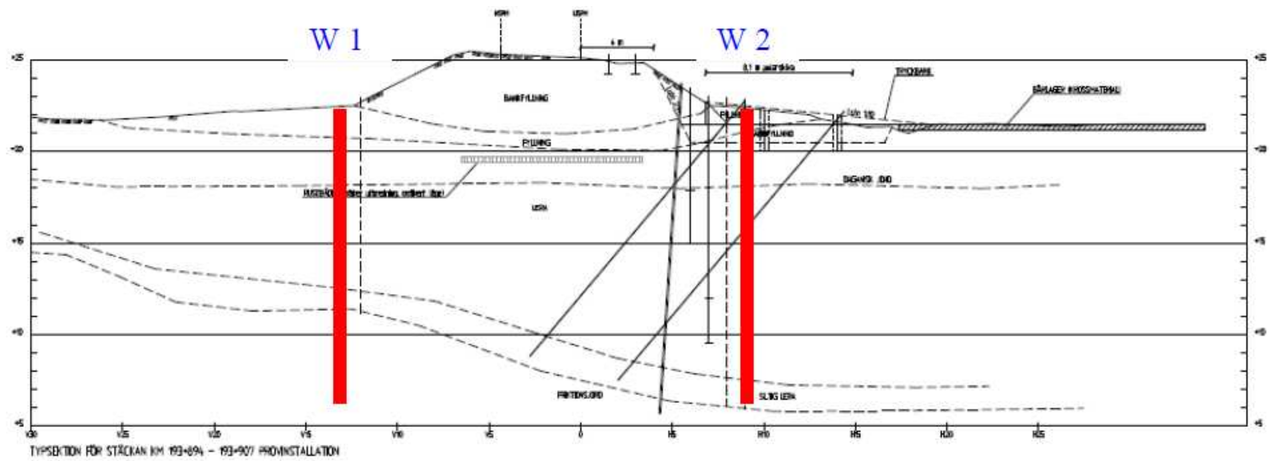


Figure 105 – Sketch showing a vertical section of the embankment and the underlying soil. The green polygon indicates a dry mix column slab and the two red lines indicate boreholes within the slab.



PEH tubes internal diameter 76 mm
inclination 40 degrees to vertical plane

Figure 106 – Sketch showing a vertical section of the embankment and the underlying soil. The two red lines indicate boreholes located in the soft natural soil.

Precise knowledge of the borehole geometry, i.e. borehole orientation, is important for the estimation of the velocity model in the tomographic inversion calculation. This is especially important in the dry mix column slabs because of the combination of closely spaced boreholes and a fairly solid, thus fast, material. Minor deviations in the borehole orientation could lead to large errors in the velocity model.

Borehole orientation measurements were therefore performed in the 10 boreholes of the sections P1, P3, P5, P7 and P11 that is obvious from Figure 104. The measurements were performed by Malå GeoScience using the “Flexit Multi Smart” system, and were conducted from the ground surface, down to the bottom of the borehole, with a point distance of 1.0 m.

In the Figure 107 the view of position of plastic tubes used for seismic tomography measurements is shown.

Figure 107 – Plastic tubes prepared for seismic tomography measurements.

Results of measurements in the soft soil under the embankment and in the natural soil

The result of the evaluated shear wave velocity measurements from beneath the embankment and in the natural soil are presented in Figure 108. The shear wave velocity varies in the range 40 – 300 m/s. Two applied geophysical velocity models show nice fits to the measured data with average relative errors of c. 4% (average RMS related to average measured travel time).

In the natural soil the velocity averages at c. 40-100 m/s from the ground surface and down to c. 8-9 m depth. The lowest velocities are identified close to the ground surface, and there is a general velocity increase with increasing depth. These velocities are well in accordance with what one would expect in soft clay. Below c. 9 m there is a distinct increase in velocity to c. 150-300 m/s, which indicates the existence of a layer of relatively stiffer material.

Under the embankment we can easily identify the "stiffer" high velocity layer at c. 9-10 m depth. However, the upper 10 m of the tomogram beneath the embankment shows a significantly different velocity distribution compared with the natural soil. Immediately under the embankment and down to c. 5 m depth the shear wave velocity is in the range 80-140 m/s and the velocity model indicates layering with c. 1 m thick high and low velocity layers. From about 5 m depth and down to 10 m depth the soil material seems to be more homogenous with shear wave velocities in the range 110-140 m/s. At c. 10 m depth there is a rather distinct velocity increase as mentioned earlier, indicating a relatively stiffer material.

The results of the seismic measurements in the soft soil under the embankment and in the natural soil beside the embankment are geologically reliable. The shear wave velocity ranges and distributions are well in accordance with previous results from this area and agree well with data reported in the literature. The compression wave velocity models also show reasonable results, even though the absolute velocities are some 10-15% higher than what normally would be expected in soft clay.

The shear wave velocity is significantly higher under the embankment compared to the natural soil. In Figure 109 below we present a compilation of average shear wave velocities from beneath the embankment and in the natural soil. Close to the ground surface the average shear wave velocity is c. 70% higher under the embankment compared to the natural soil. Some 4 m below the ground surface the shear wave velocity is about 40% higher under the embankment and the results indicate that the shear wave velocity is increased down to c. 8-9 m depth below the ground surface (elevation of c. 13-14 m).

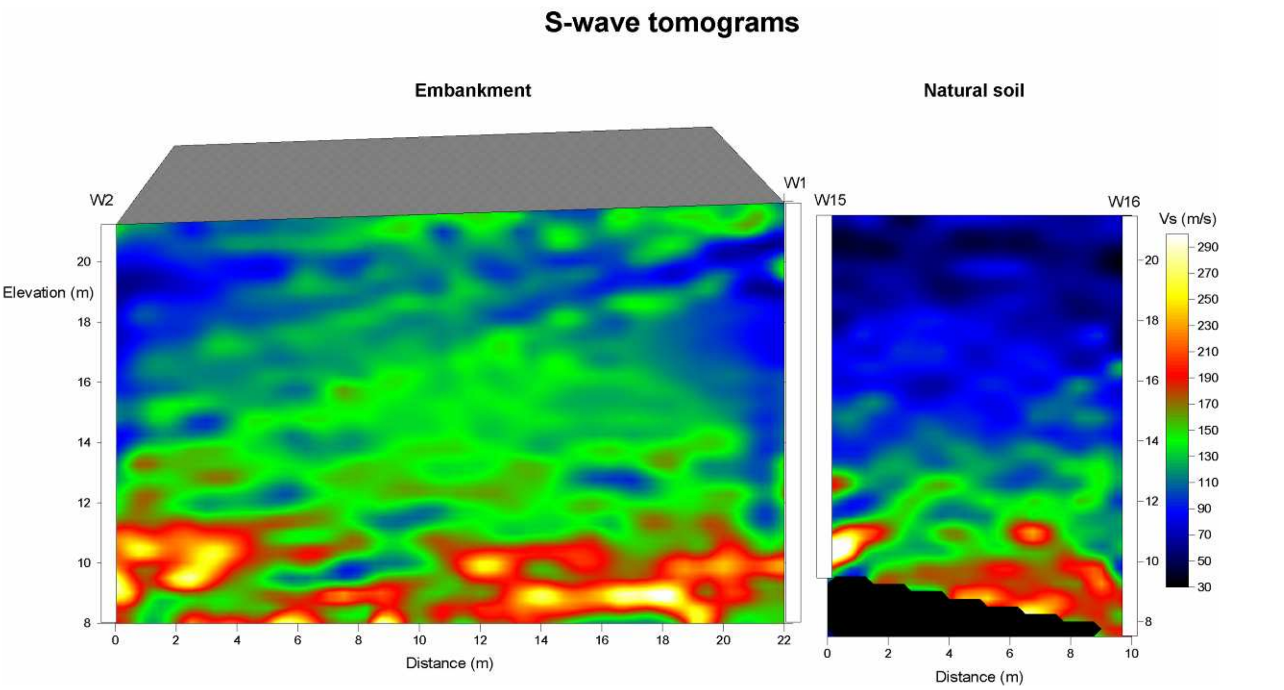


Figure 108 – Shear wave tomograms from under the embankment and in natural soil

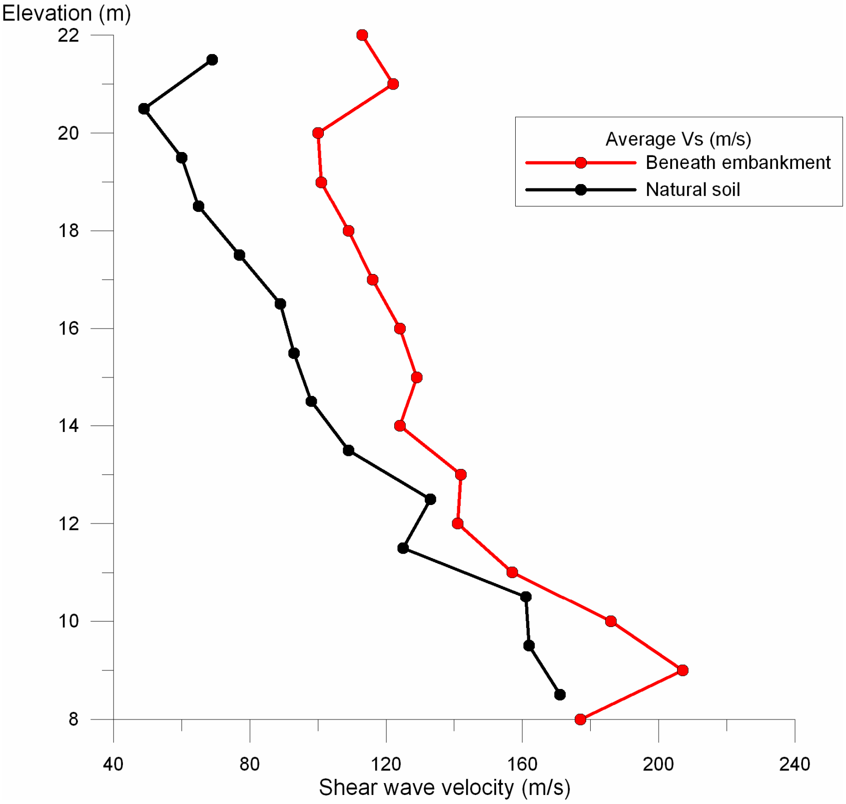


Figure 109 – Diagram showing average compression wave velocity versus elevation for the section under the embankment and in natural soil.

Results of measurements in the lime cement inclined walls

The measurements have been performed in the five dry mix column slabs. Figure 110 shows an example of evaluation of shear wave and compression wave in form of tomograms.

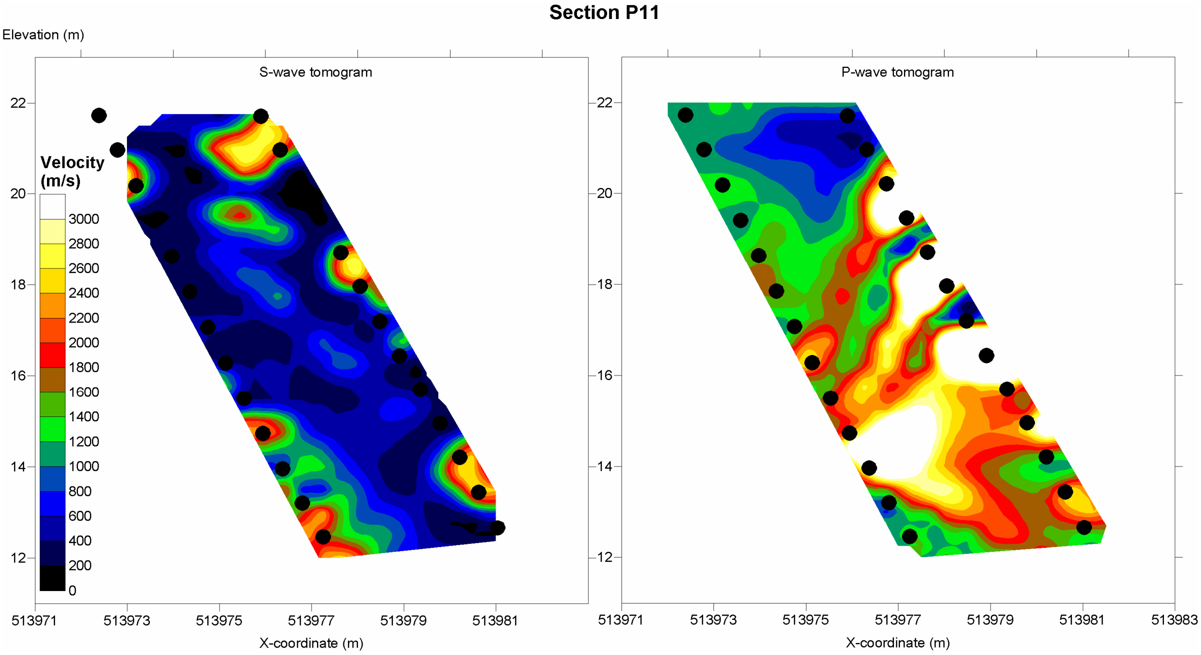


Figure 110 – Shear wave (left) and compression wave velocities (right) tomograms from dry mix column slab section P11.Change “elevation” to Depth (m)

The velocity distributions diagrams presented in Figure 111 are directly related to homogeneity of the dry mix column slabs.

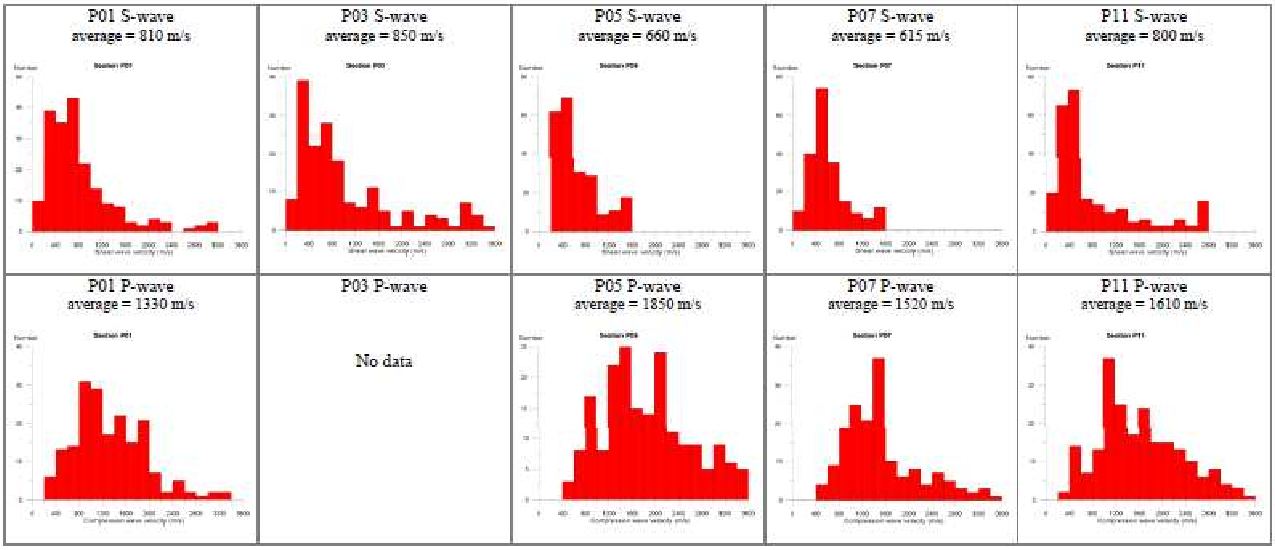


Figure 111 – Histogram showing shear wave (upper row) and compression wave (lower row) distribution for the dry mix column sections P01, P03 (no P-wave data), P05, P07 and P11. Average velocities (normal distribution assumed) are displayed for each histogram.

The measurements in the five dry mix column slabs seem to result in fairly consistent velocity models indicating median shear wave velocities in the range 500-850 m/s and median compression wave velocities in the range 1200-2000 m/s. The shear wave velocity models generally show poorer fits to the measured data compared with the compression wave models. The seismic data consistently indicate that the part of the slab that is located immediately below the embankment shows significantly higher wave velocity compared with the part located beside of the embankment. This is possibly related to relatively stiffer material beneath the embankment as indicated by the increase in seismic velocities in the soil reported in Figure 109.

The section P07 shows the narrowest distribution in both P- and S-velocity, which suggests that this slab is most homogenous of the five. However, the average and median velocities of this slab are comparably low, indicating relatively weaker average material properties.

The two sections P01 and P03 show the widest shear wave velocity distributions of all five sections, which indicates that poorer mixing occurred in these two slabs compared with e.g. P07. This relatively higher degree of heterogeneity of P01 and P03 is also supported by visual inspection of the velocity models. It is however worth noting that the sections P01 and P03 have the highest average shear wave velocities of the five slabs.

The sections P05 and P11 show the widest compression wave distributions in Figure 111, whereas their shear velocity distributions are relatively narrower; at least for P11. This somewhat contradictive result is supported also when viewing the contour plots of the velocity models. The reason for this is not fully understood. In the case of P11 we know that the shear wave model fit is a bit poor, which could contribute to scattered model velocities, but this is not the case of P05 where both S- and P-velocity model fits are good. The differences in compression wave and shear wave distribution could perhaps be related to the fact that different material properties affect the two velocities in different ways, e.g. such as for the water content of soft soil, which we clearly see e.g. in the velocity models of natural soil. If such an assumption is valid also for dry mix columns is however not known.

The compression wave velocity of the dry mix column slabs is mainly in the range of 800-2500 m/s, which is fairly close to the P-velocities of the natural soil below the ground water table. It seems likely to assume that the water content plays such a big role for the compression wave velocity that it overwhelms the differences in material properties that clearly must be present in the soft soil compared to the dry mix column material.

In conclusion we can state that average (and median) shear wave velocities of the dry mix column slabs are in the range of 500-850 m/s, which is well in accordance with the results from the down-hole measurements. These velocities are significantly higher than the velocities we measure in the soft soil of 60-200 m/s (also under the embankment), see Figure 112. There are very few shear wave velocities < 200 m/s in the dry mix columns. We can thus conclude that the dry mix column slabs must be significantly more rigid compared with the natural soil. However, the results of the investigation also indicate clear differences in homogeneity of the dry mix columns, where P07 seems to the most homogenous. The slab P03 is the most heterogeneous.

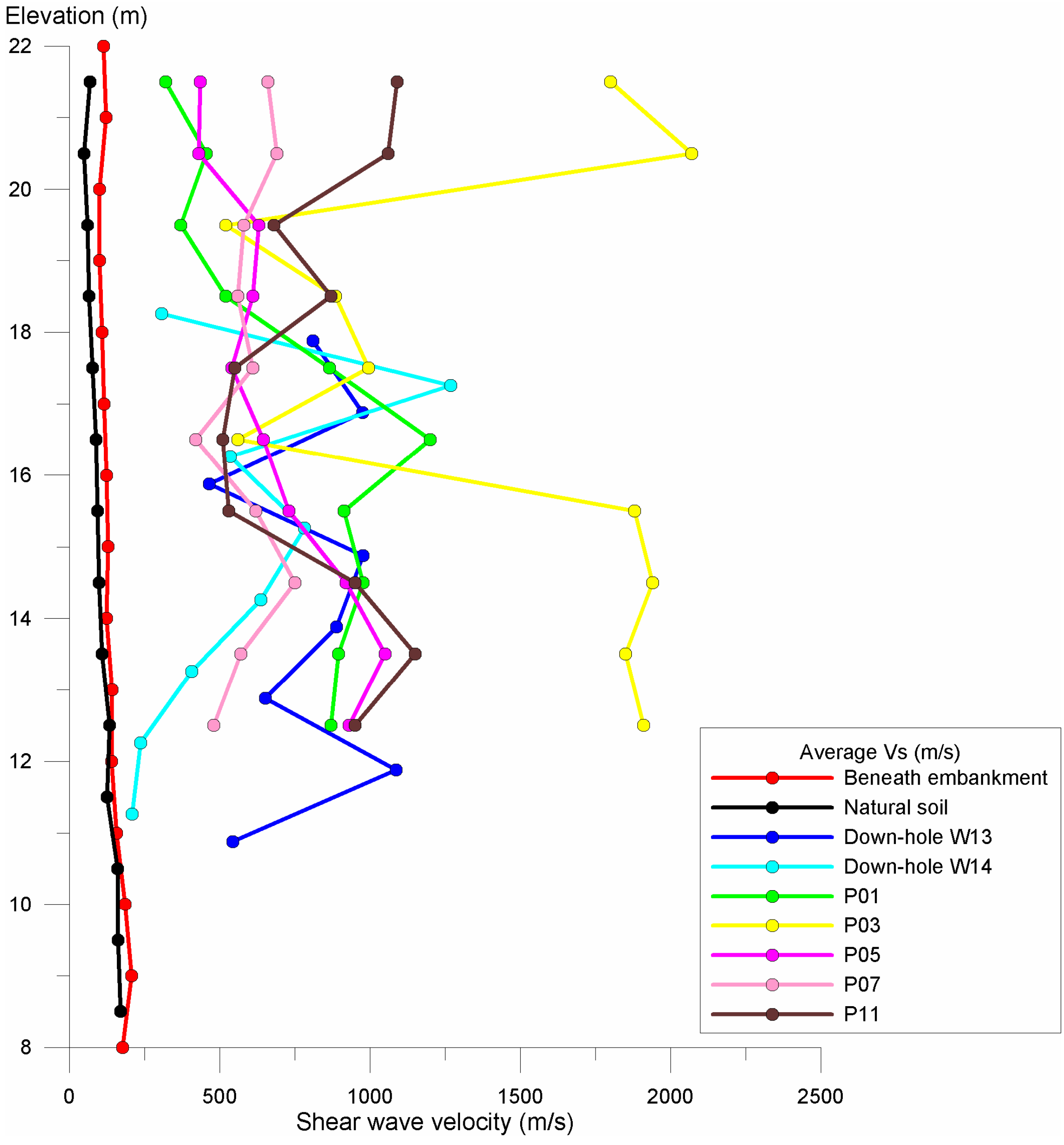


Figure 112 – Diagram showing average shear wave velocities for all measurements performed in this investigation (soil beneath embankment, natural soil, down-hole in dry mix column ring and tomography in the five dry mix column slabs).

3.7 Monitoring results

3.7.1 General

All monitoring results are presented from the start of this project (end of June 2008) or from the time new measurement points were installed.

A track adjustment of the western track was performed 23rd October 2008.

Below is presented the monitoring results for the different measurement of

- twist of the track
- displacement of the track
- displacement at the surface of the soil at different distances from the track
- distribution of the vertical displacement with depth
- distribution of the horizontal displacement with depth
- distribution of the pore pressure with depth

3.7.2 Twist of track

In order to have the possibility to keep operation on the track during installation of inclined columns under the embankment it was important to check that a 'stable' track was maintained and that the track did not show too much difference in deterioration of track geometry. Detailed measurements of the twist of track at every 5 m in the area of installation were performed during the installation of columns. Normally these measurements were performed after the installation of each panel was finished and also at the start and end of each working day. After the installation the measurements were continued as a part of the other performed measurements.

The change in the twist of track was smaller than the allowable limits under the whole installation and also after.

Another indication showing that the twist of the track was small is that the measurement of settlement of the steel nails at ends of the sleepers showed very small differences in settlement, thus indicating a small twist of the track.

3.7.3 Settlements (displacements) of track – end of sleepers

The settlement of the track was measured by positioning using total station measurements of steel nails at the end of sleepers.

The measured settlement of the sleepers on the western track at a distance of more than 20 m from the panel section P6, km 193+900, was very small in the order of less than 2 mm.

The settlements of the steel nails on the western part of the sleepers on the western track with time for the different measurement points along the track are presented in Figure 113.

The measurements show a heave of the sleepers in the order of 5-6 mm when the installation of the columns was performed in the beginning of July 2008. After that a settlement takes place and the results from the last measurements performed shows about 30 mm of settlement in the middle of the installation section (point 131) and approximately 5 mm about 20 m from the middle of the installation section (point 127 and 135). In Figure 114 is shown the measured settlements along the track at different dates. It is obvious that there is a significant settlement in the measurement section km 193+900 and that there are considerable settlements also outside the width of the installed zone of panels P1 to P11 (with a total width of about 14 m). This corresponds to an area between -6.5 to +6.5 m in Figure 114. The total width of the settlement area along the track as shown in Figure 114 was in the order of 40 to 50 m.

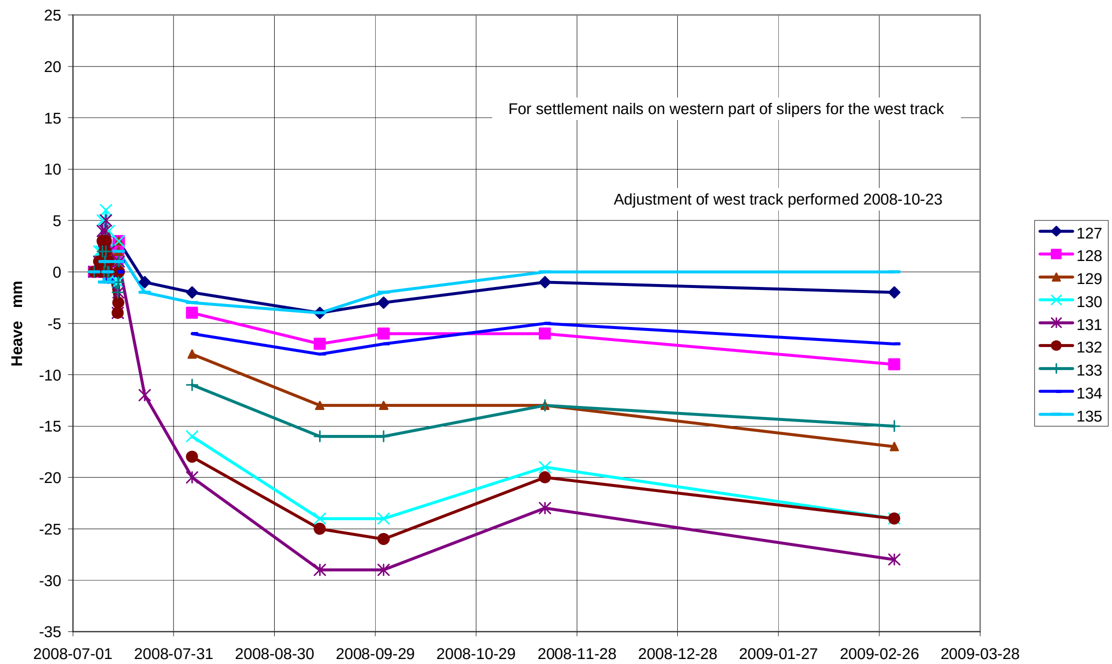


Figure 113 – Settlements of the track versus time for points on the western part of the sleepers of the western track. Points with about 5 m distance and point 131 is in section km 193+900.

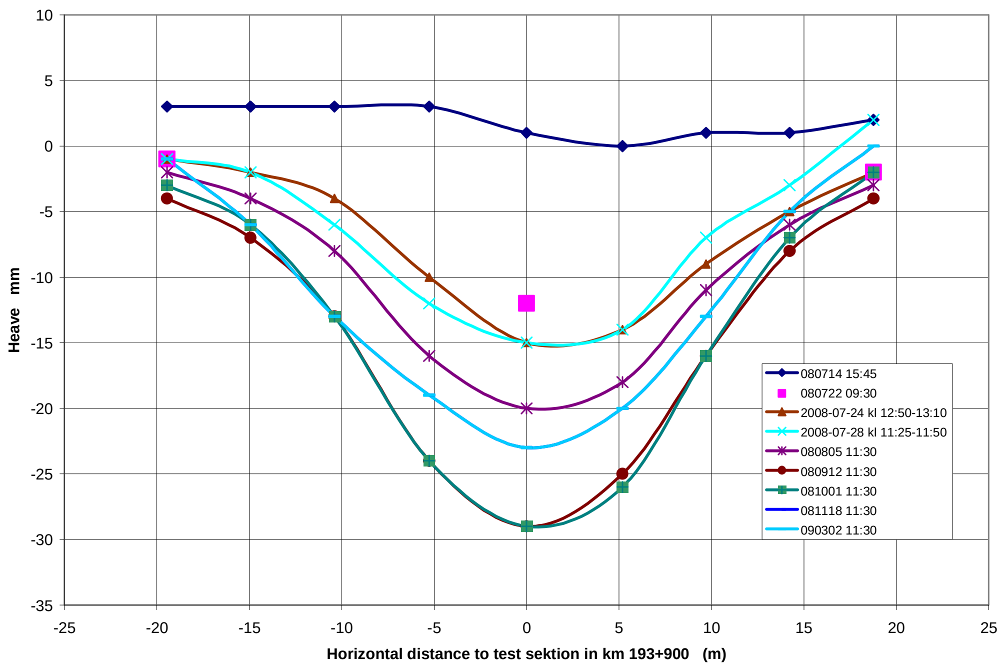


Figure 114 – Settlements of the track versus distance from section km 193+900 for points on the western part of the sleepers of the western track.

In Figures 115 and 116 are presented the settlements measured at the east steel nails on the western track. The measurements show a settlement of about 23 mm at section km 193+900 (point 122) and 5 mm about 20 m from the middle of the section km 193+900 (point 118 and 126) taking the adjustment of the track at 23rd October 2008 into account.

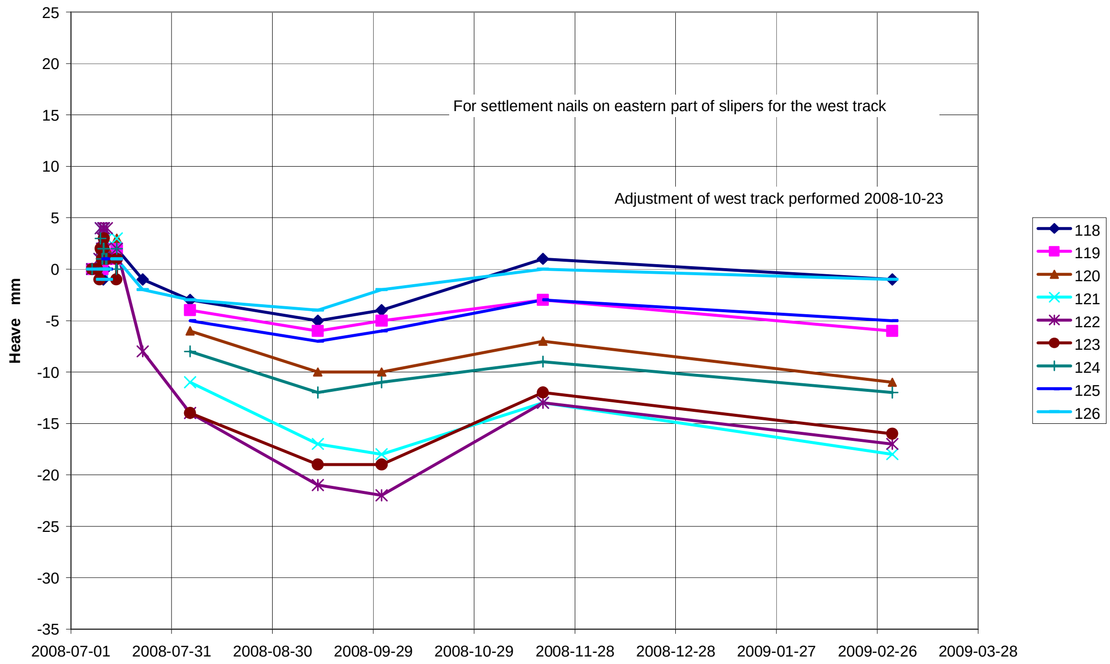


Figure 115 – Settlements of the track versus time for points on the eastern part of the sleepers on the western track. Points with about 5 m distance and point 122 is in section km 193+900.

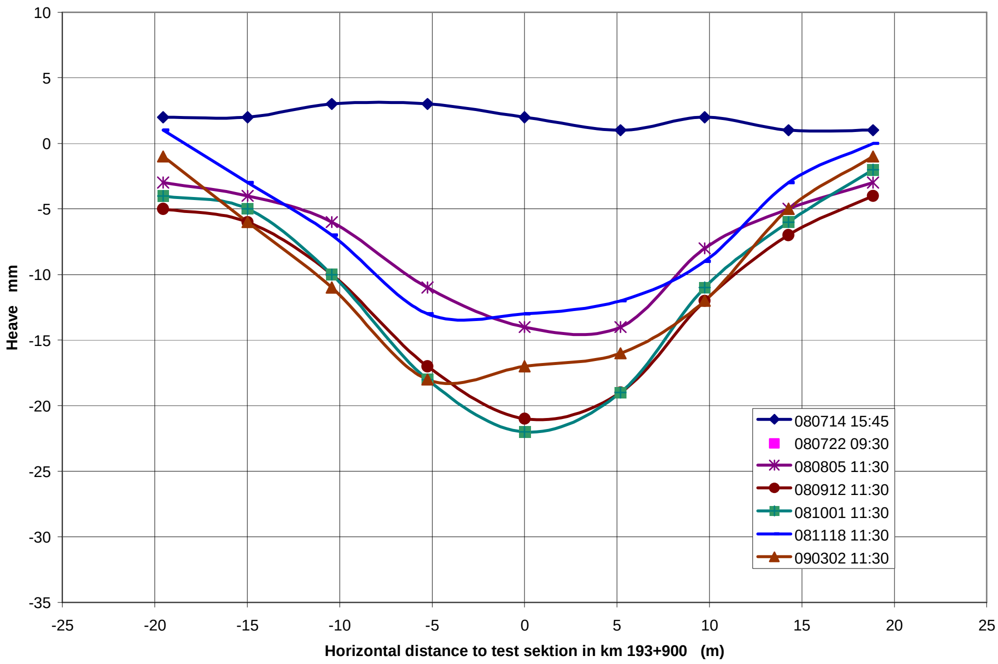


Figure 116 – Settlements of the track versus distance from section km 193+900 for points on the eastern part of the sleepers on the western track.

In Figures 117 and 118 are presented the settlements measured at the western steel nails on the eastern track. The measurements show a settlement of about 20 mm at section 193+900 (point 113) and 0-5 mm about 20 m from the middle of the section 193+900 (point 109 and 117).

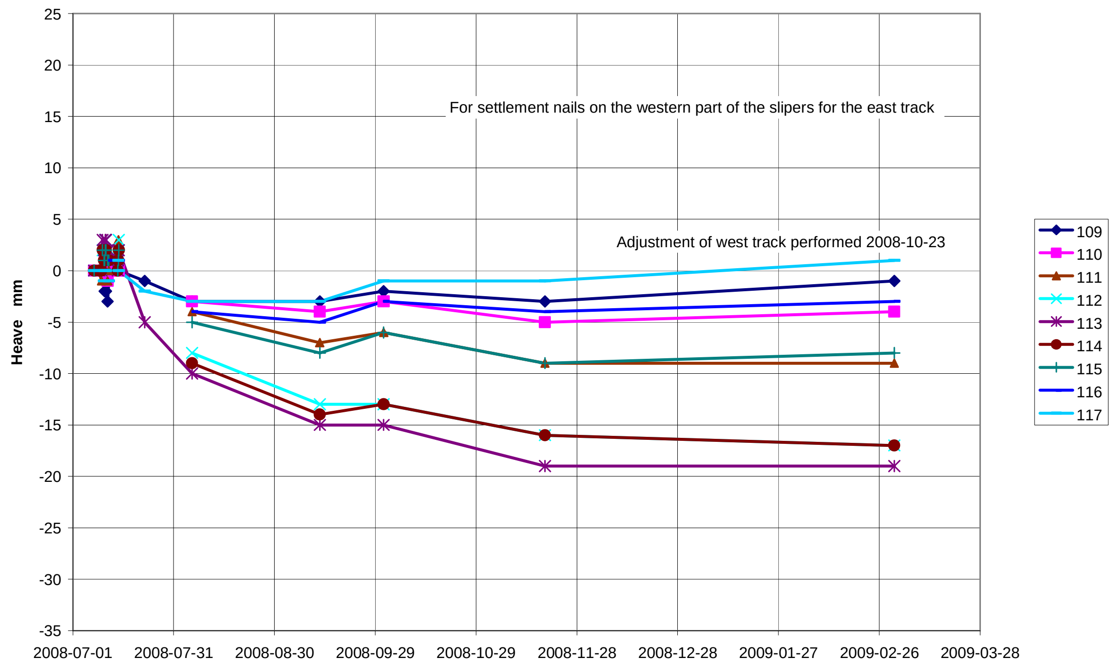


Figure 117 – Settlements of the track versus time for points on the western part of the sleepers on the eastern track. Points with about 5 m distance and point 113 is in section km 193+900.

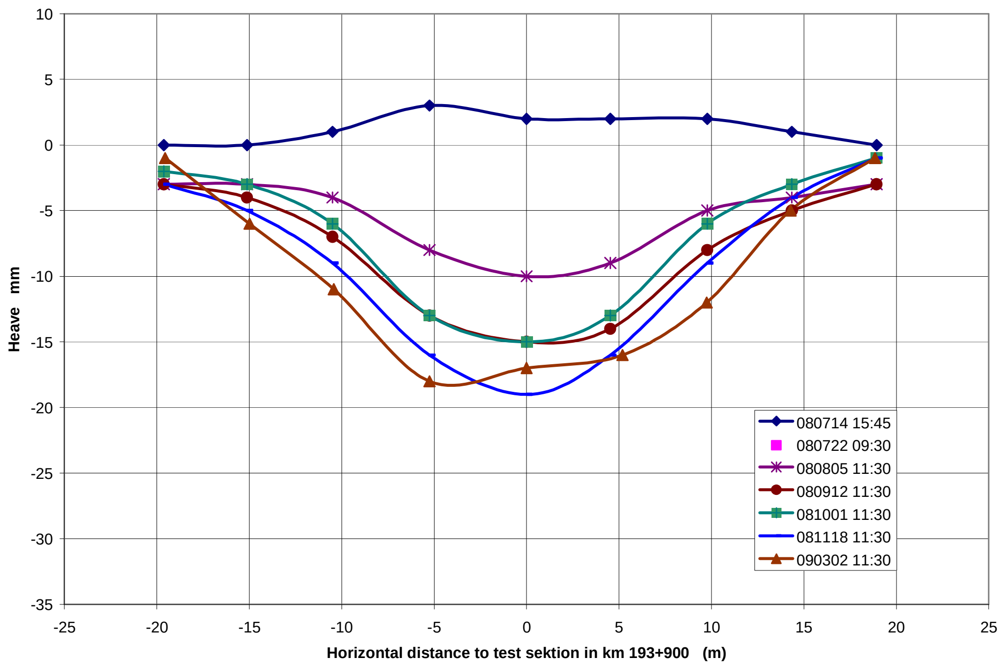


Figure 118 – Settlements of the track versus distance from section km 193+900 for points on the western part of the sleepers on the western track.

In Figures 119 and 120 are presented the settlements measured at the eastern steel nails on the eastern track. The measurements show a settlement of about 15 mm at section 193+900 (point 104) and 0-3 mm about 20 m from the middle of the section 193+900 (point 100 and 108).

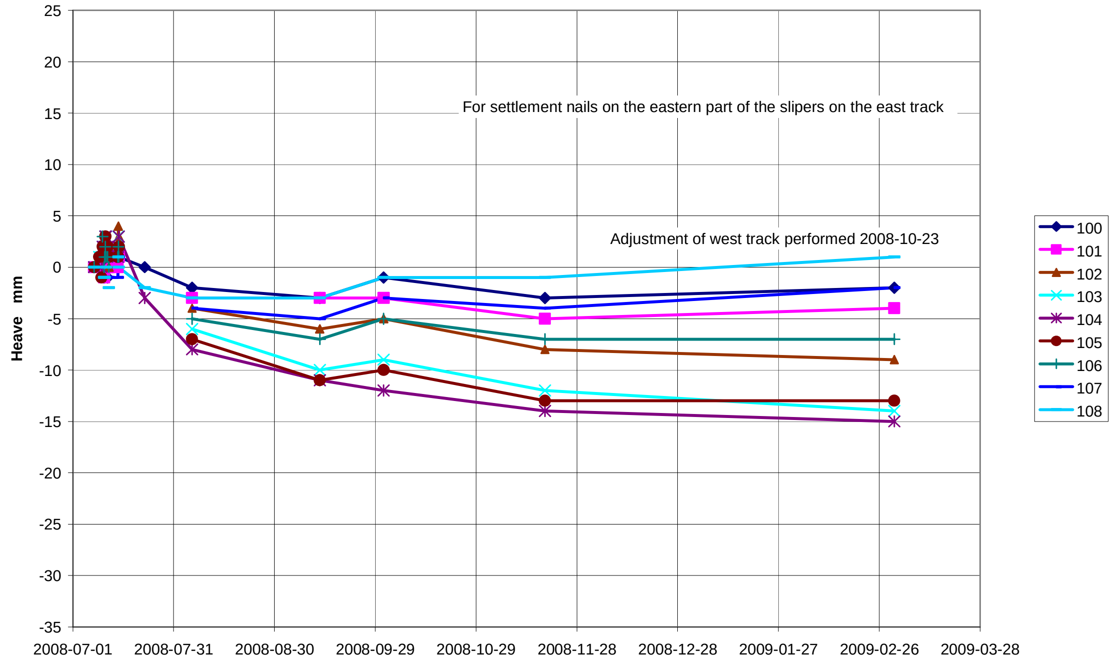


Figure 119 – Settlements of the track versus time for points on the eastern part of the sleepers on the eastern track. Points with about 5 m distance and point 104 in section km 193+900.

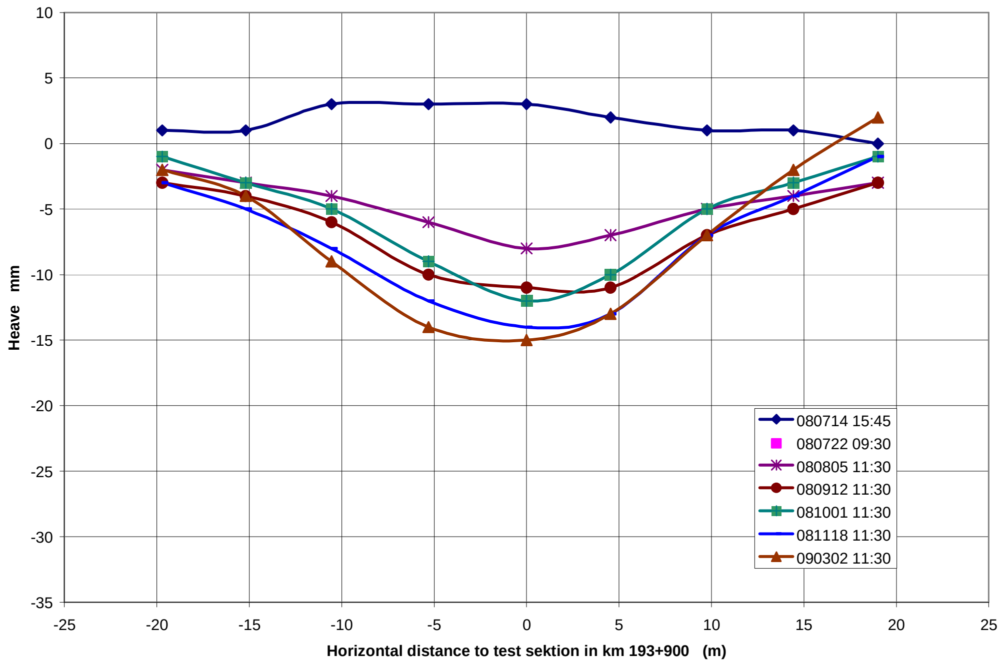


Figure 120 – Settlements of the track versus distance from section km 193+900 for points on the eastern part of the sleepers on the eastern track.

The results show that the installation of the columns caused a heave of the order of 4-6 mm of the two tracks and thereafter a settlement in the order of 25-30 mm for the western track (the track closest to the installation of panels) and 15-20 mm for the eastern track. However the adjustment of the western track 23rd October 2008 makes it difficult to correctly evaluate the total settlement of the track. The settlement of 25-30 mm evaluated from the latest measurement of the western track should probably be somewhat higher, more in the order of 35-40 mm taking into account the effect of track adjustment.

The settlement of the steel nails on the sleepers could be compared with the settlement of the settlement markers just beside the track, see section 3.7.4.

The settlements have been evaluated from the positioning of the steel nails by a total station. This means that there are also results from horizontal positioning but the measurement error for the horizontal displacement is too large for any clear indication of the horizontal displacement.

In Figure 121 – 123 examples of results are presented from the measurement of the steel nails no 127 to 135 on the western part of the sleepers on the western track. The measured horizontal displacements were in the order of – 20 mm to +20 mm.

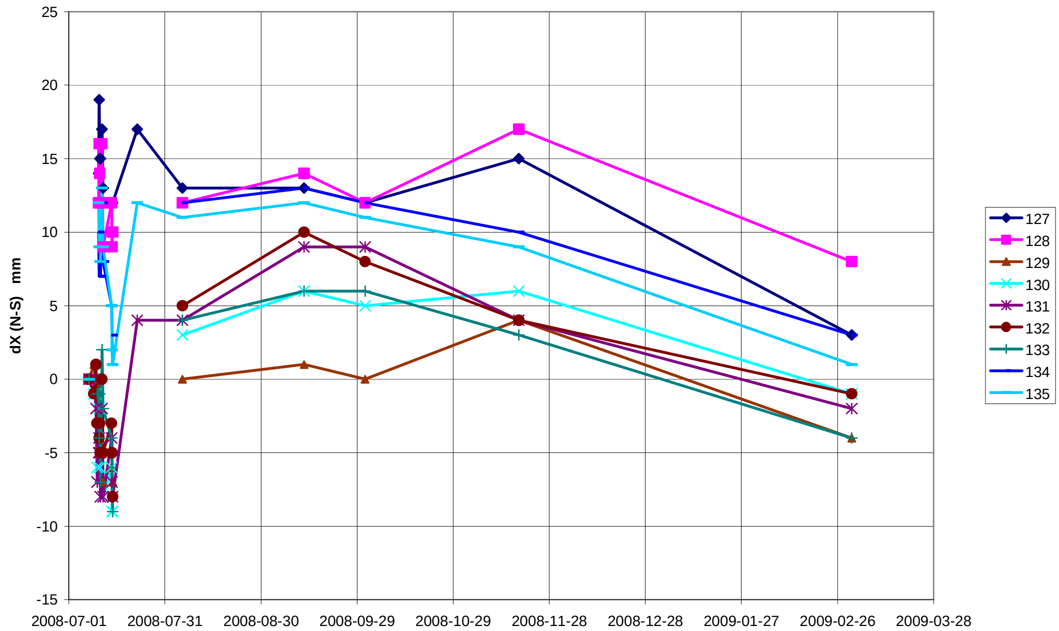


Figure 121 – Horizontal displacements of track in North-South (N-S) direction versus time for points on the western part of the sleepers of the western track. Points with about 5 m distance and point 131 is in section km 193+900.

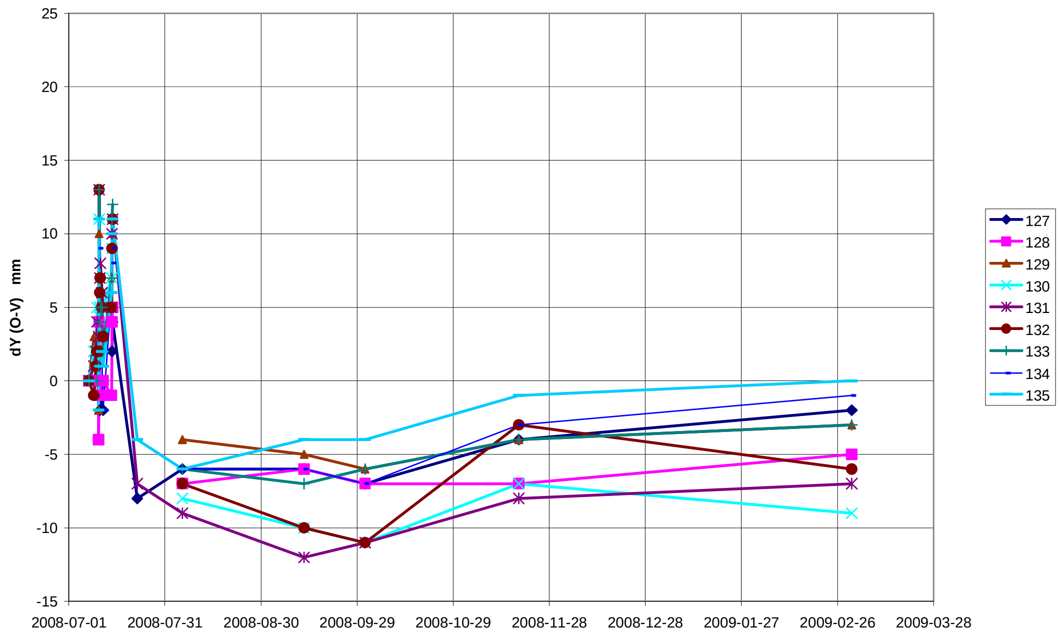


Figure 122 – Horizontal displacement of the track in East-West (O-V) direction versus time for points on the western part of the sleepers of the western track. Points with about 5 m distance and point 131 is in section km 193+900.

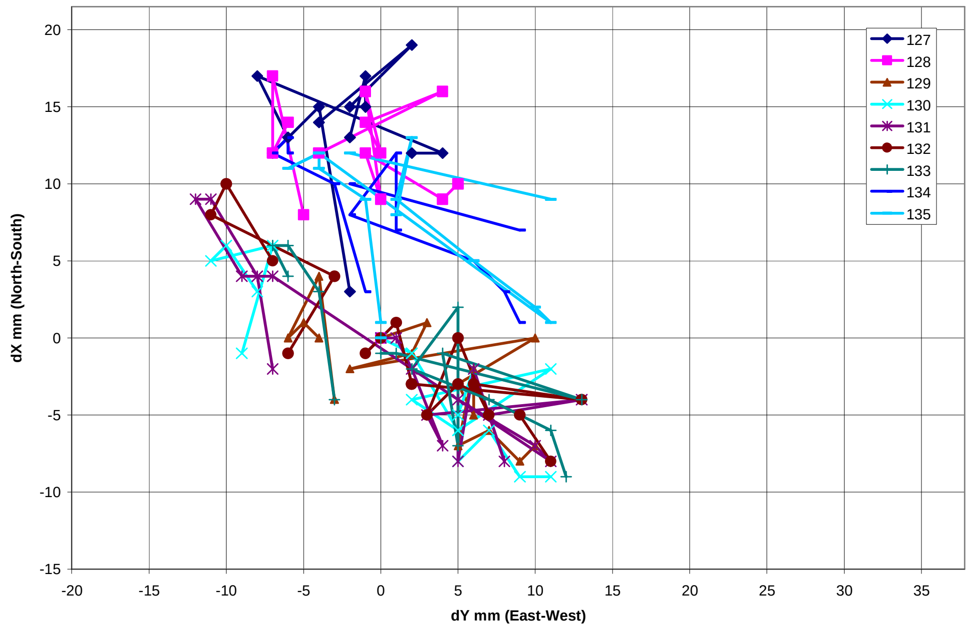


Figure 123 – Horizontal displacements of the track in North-South direction versus horizontal displacement in East-West direction at different times for points on the western part of the sleepers of the western track. Points with about 5 m distance and point 131 is in section km 193+900.

In Figure 124 the results given in Figure 123 are only presented for points 127, 131 and 135. The points 127 and 135 are located about 20 m from the point 131 in the middle of the measurement section km 193+900. The point 131 shows as expected somewhat larger horizontal displacements than at point 127 and 135.

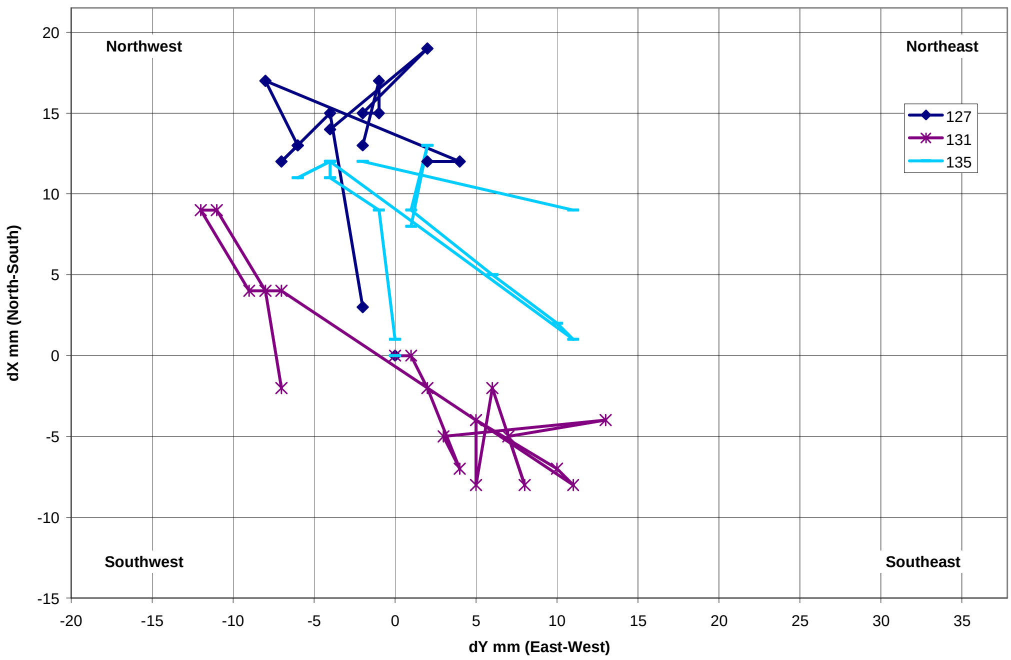


Figure 124 – Horizontal displacements of the track in North-South direction versus horizontal displacement in East-West direction at different times for three points on the western part of the sleepers of the western track. Points with about 20 m distance and point 131 is in section km 193+900.

3.7.4 Displacements of settlement markers in soil

The six settlement markers P1 to P6 were installed in June 2007 with the objective to check if there were any ongoing settlements before the installation of columns, planned to start in September 2007. The installation was postponed and started instead in July 2008 but this still gave the opportunity to gather information about the ongoing settlement of the embankment.

In Figure 125 is presented the results from the measurements of settlement for P1 to P6 from June 2007 to March 2009. The settlement markers P1 to P3 located just outside the sleepers were damaged in the winter season 2007 and were reinstalled in March 2008. They were installed at a lower level but are presented in the Figure 125 with no correction for the changed level. The change of level for P1 to P3 was in the order of 150 to 200 mm, see Figure 125. The trends from P1 to P6 indicate that the ongoing settlements before the installation of columns in July 2008 are rather small, less than 2 mm per year.

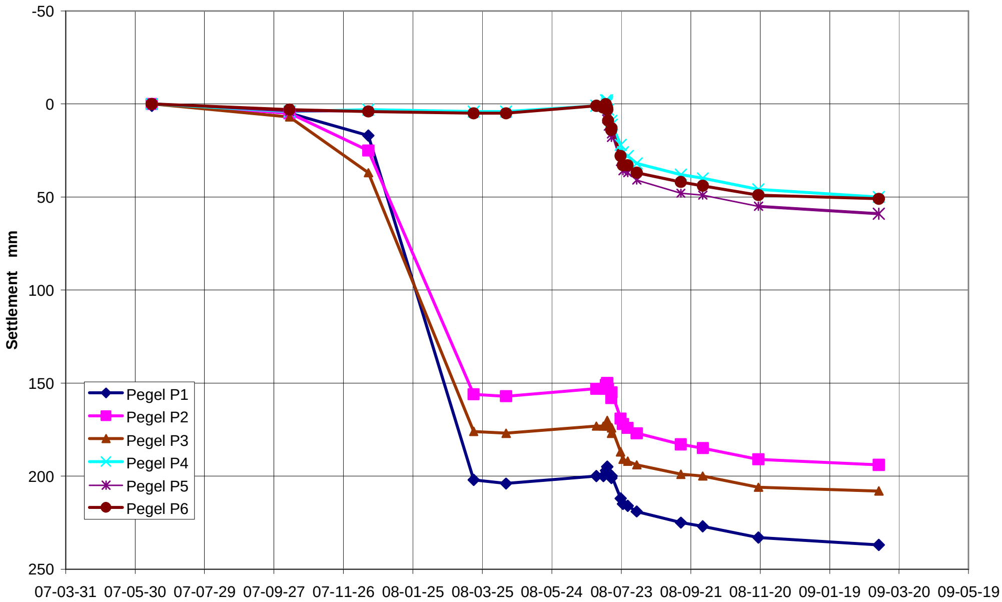


Figure 125 – Settlement versus time for settlement markers P1 to P6 at the section km 193+900 with reference to June 2007.

In Figure 126 the results are presented starting instead with the measurement performed at 1st July 2008 just before the installation of the columns started. From Figure 126 it could be concluded that the heave of the settlement markers P1 to P6 during the installation of the stabilized panels in July 2008 was in the order 3-6 mm, which is about the same as the heave of the western part of the sleepers on the western track. After the installation was finished the results shows a continuing settlements for marker P2 just outside the end of the sleepers was about 40 mm in March 2009. The settlement of the marker P5 about 3.0 m from track centre was about 60 mm at the same time.

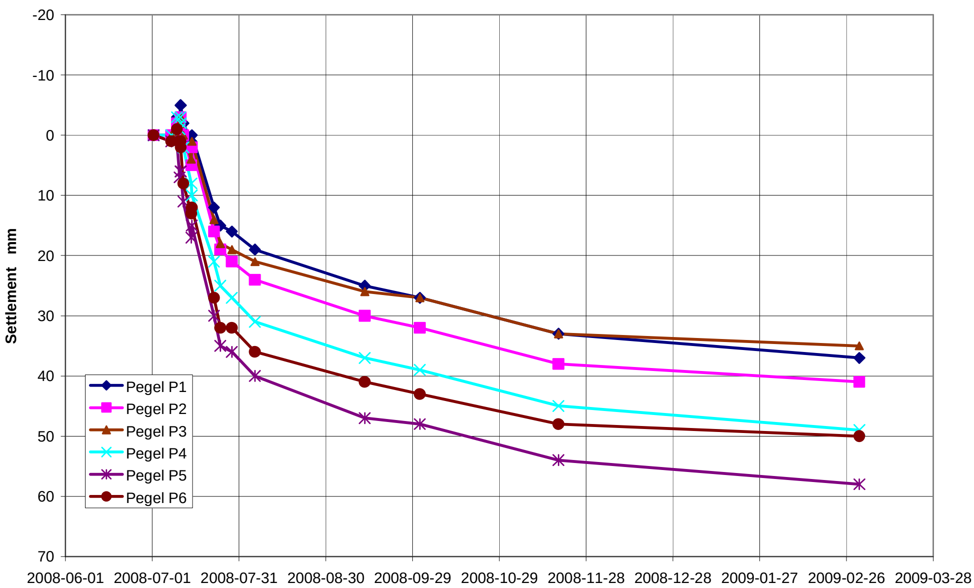


Figure 126 – Settlements versus time for settlement markers P1 to P6 at the section km 193+900 with reference to 1st July 2008.

The settlement markers P7 to P9 were installed in the end of June 2008 about two weeks before the installation started.

In Figure 127 is presented the measured settlement versus time for all the nine settlement markers P1 to P9. In Figure 128 the extra levelling performed 2008-06-24 and 2008-07-28 are also included. In Figure 127 and 128 are also included results from measurements of the head of the pipe for the bellow hose.

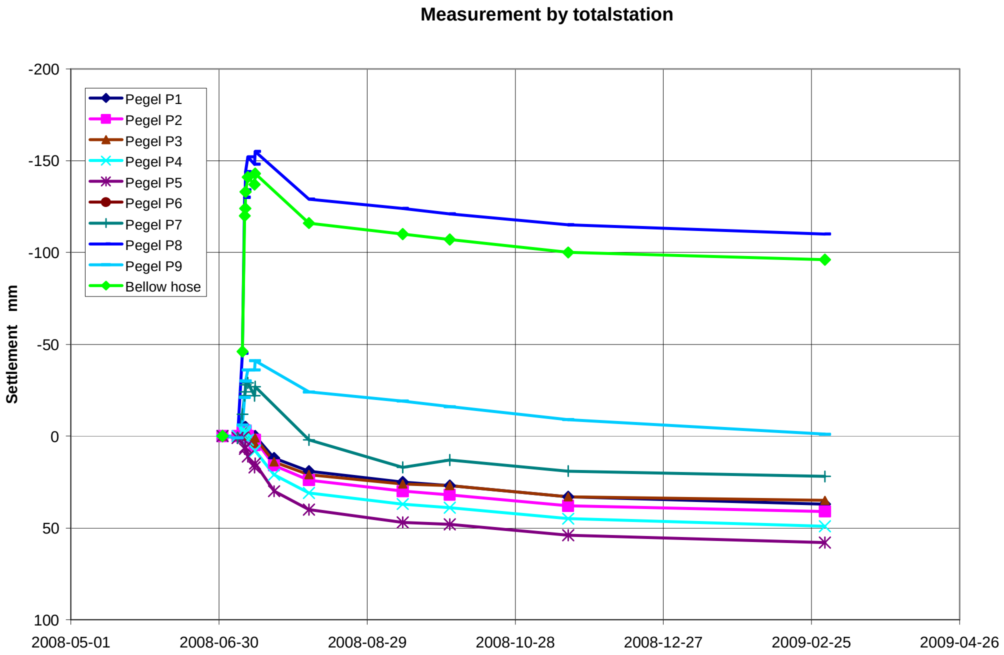


Figure 127 – Settlements versus time for settlement markers P1 to P9 at the section km 193+900 with reference to 1st July 2008.

The results in Figure 127 and 128 indicates that there was a substantial heave in the test area after installation of the inclined overlapping columns, in the order of a heave of 150 mm at point P8 and the bellow hose.

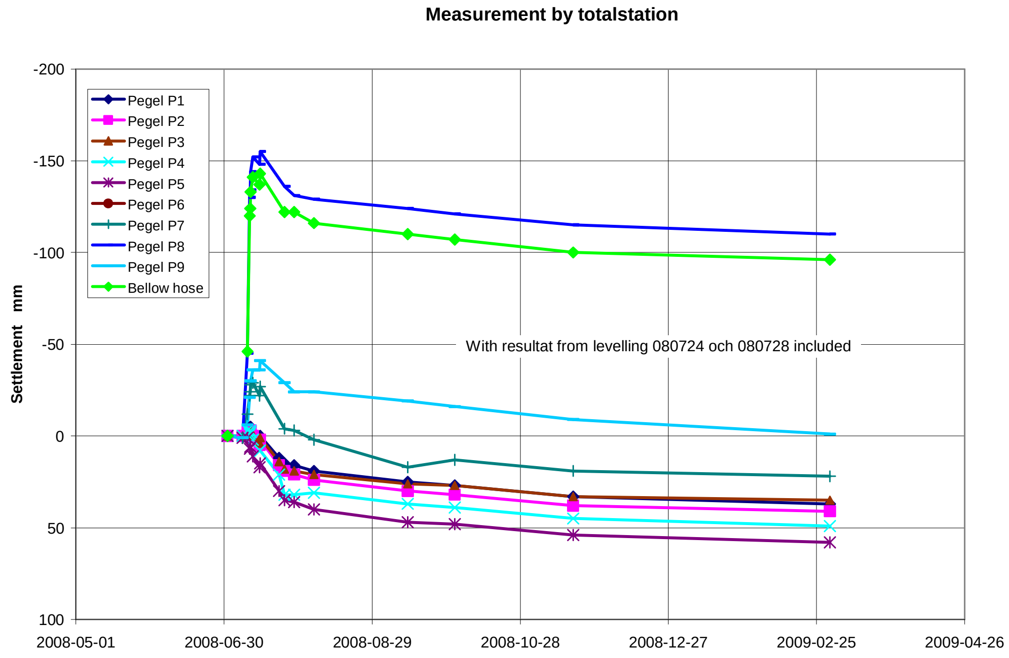


Figure 128 – Settlements versus time for settlement markers P1 to P9 at the measurement section. Results from levelling performed 2008-07-24 and 2008-07-28 included.

The point P7 shows a heave of 30 mm and the point P9 about 45 mm at the end of installation of panels.

In Figure 129 are presented the results from the settlement markers P1 to P9 together with the four points on the two sleepers on the western and eastern track. Figure 129 clearly shows the installation of the two tracks. After the installation was finished there was a settlement going on involving a larger area than during installation of columns. After the installation of columns were finished on 14th July 2008 the increase in settlement to March 2009 is in the order of 20 mm for the eastern track, 25-30 mm for the western track and about 40 mm in the area where the inclined overlapping columns were installed.

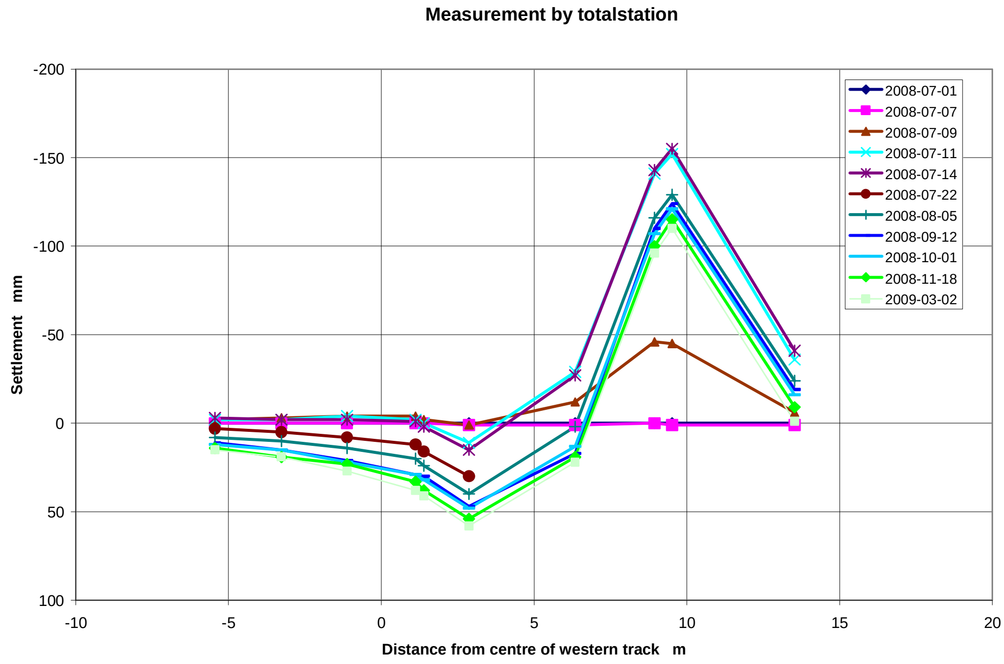


Figure 129 – Settlements versus distance from the centre of the western track for different dates.

3.7.5 Displacements with depth in soil from bellow hose

One bellow hose was installed in the panel row where no columns were installed and it was located in the eastern part of the panel at the ground surface +21.5 and in the western part of the panel at about level +15.5.

The results from the bellow hose measurement are shown in Figure 130. The results indicate a heave that was developed evenly for the whole depth of the soft soil. After the installations were finished a compression of the whole soil profile has started. In the end of January 2009 the soil down to level +15 shows a vertical displacement that in total was a heave and the part below +15 m initially shows heave as a consequence of the installation of the columns but in January 2009 it was in total a settlement.

The result from the upper part of the bellow hose corresponds very well with the results of the settlement markers that were installed in the surface of the soil.

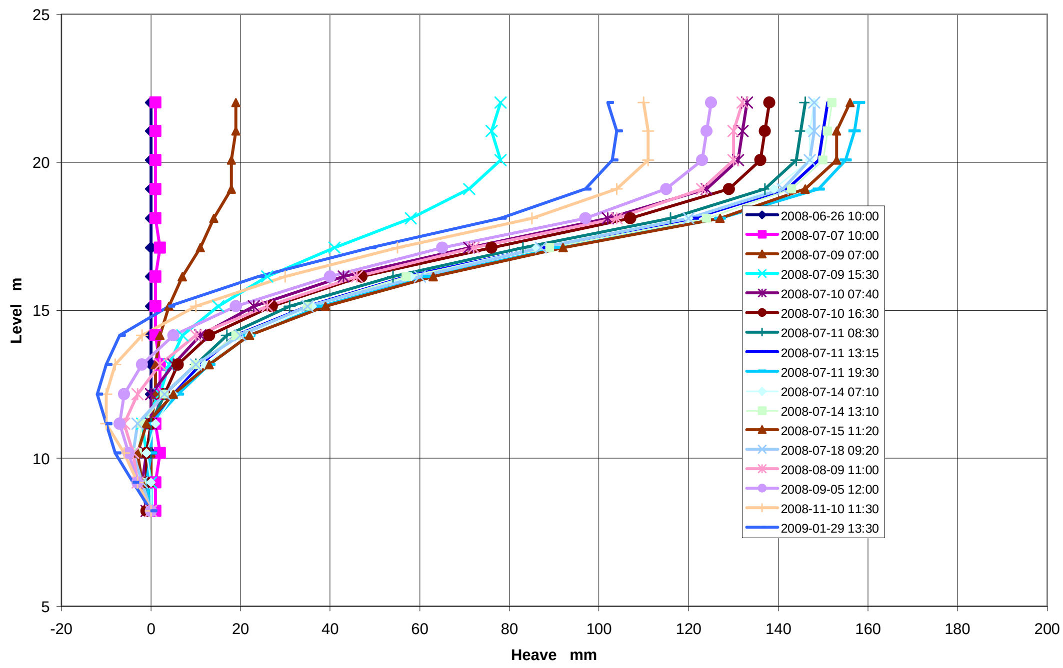


Figure 130 – Heave versus level from the bellow hose measurements at different dates.

In Figure 131 results in Figure 130 are presented only for the dates to the end of installation of the columns 14th July 2008. It shows the gradually increased heave with the number (quantity) of panels installed. The first installed panels P4, P7and P5 were very close to the measurement section (the not installed row P6) causing the heave to rather quickly increase to about 130 mm and then further increased to 160 mm at the end of installation.

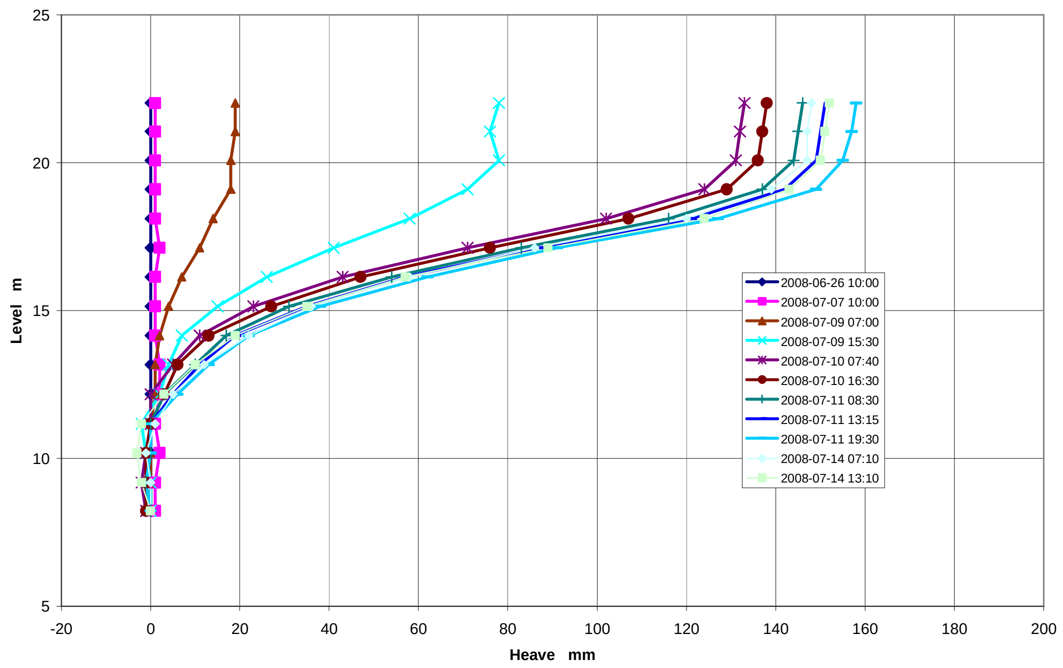


Figure 131 – Heave versus level from the bellow hose measurements at different dates up to 14th July 2008.

In Figure 132 results in Figure 130 are presented only for the part from the end of installation of the columns at 14th July 2008. The settlement was in the order of 50 mm from 14th July 2008 to 28th January 2009.

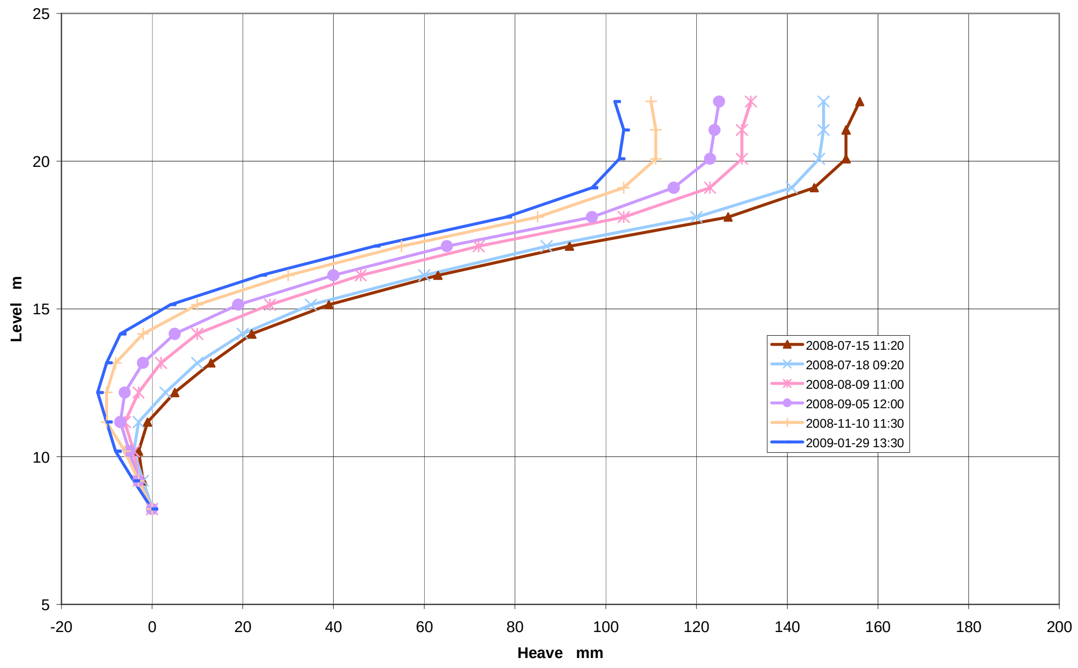


Figure 132 – Heave versus level from the bellow hose measurements at different dates after the installation of the panels were finished on 14th July 2008.

3.7.6 Horizontal displacements from inclinometers

Two inclinometers were installed in the test section km 193+900. One inclinometer was installed at the toe of the embankment on the western side and another one at the toe on the eastern side.

In Figure 133 are presented the results for the inclinometer at the western side of the embankment for a direction transverse (perpendicular to) the railway line and in Figure 134 for a direction along the track. The results are presented as horizontal displacement versus level at different dates.

For the transversal direction the measurements, see Figure 133 indicate an increase of horizontal displacement away from track when installing the panels and a maximum of about 90 mm at level +17 m.

For the direction along the track, see Figure 134 the measurements indicate first an increase of horizontal displacement towards southwest (Linköping) when installing panel P4 and then a change in direction when installing the other panels starting with P7. The horizontal displacement in the direction along the track have a maximum towards Linköping of about 15 mm and a maximum towards Norrköping of about 20 mm at level +17 m.

This means that the total horizontal displacement (taking into account both the direction transverse the track and along the track) took place in a direction that mainly is transversal to the track.

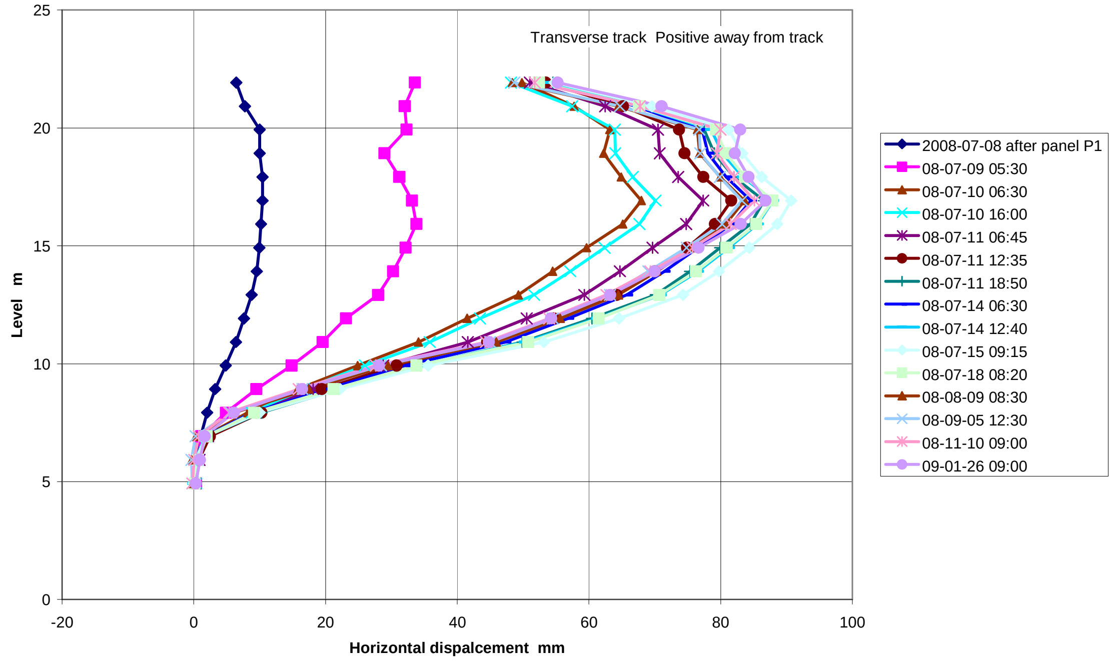


Figure 133 – Horizontal displacements transverse (perpendicular to) the railway line versus level for the inclinometer located at the western toe of the embankment at different dates.

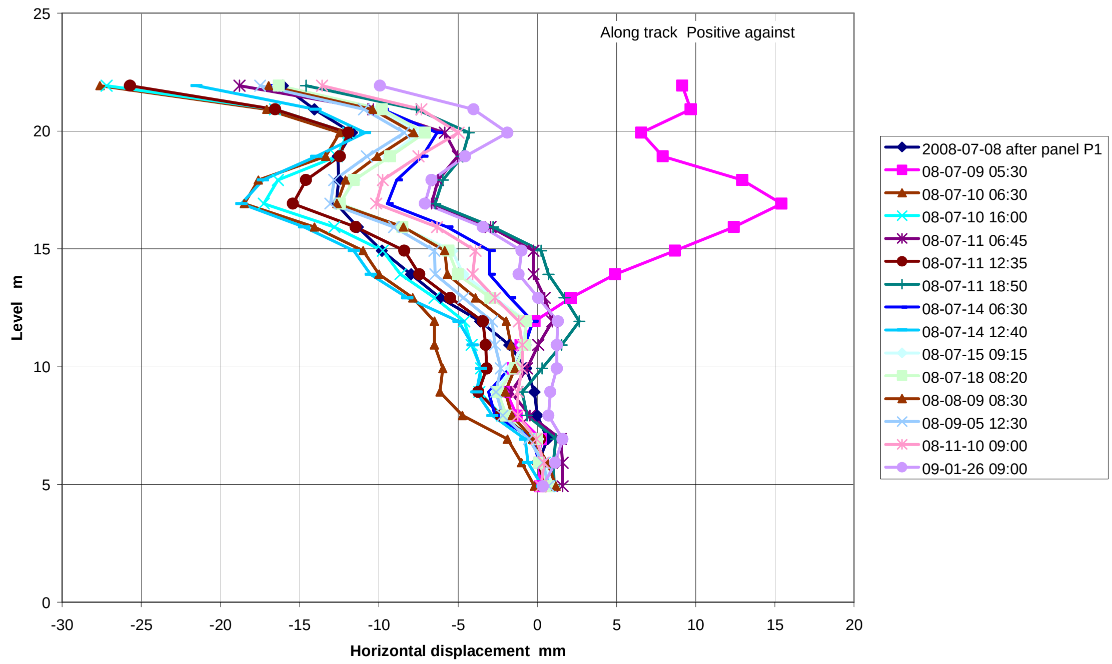


Figure 134 – Horizontal displacements along the railway line versus level for the inclinometer located at the western toe of the embankment at different dates.

In Figure 135 the results of displacement for the transverse direction are presented. But this diagram includes the dates from the beginning to the end of installation 14th July 2008 only. In Figure 136 the same results are shown but for the direction along the track.

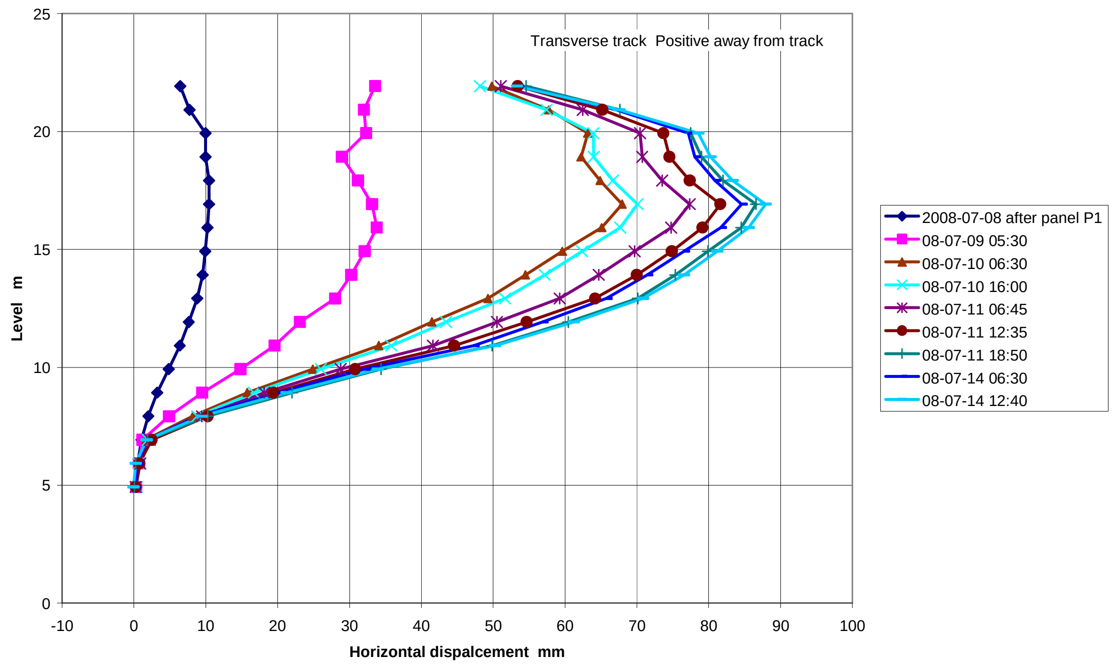


Figure 135 – Horizontal displacements transversal to (perpendicular to) the railway line versus level for the inclinometer located at the western toe of the embankment at different dates to the end of the installation of panels at 14th July 2008.

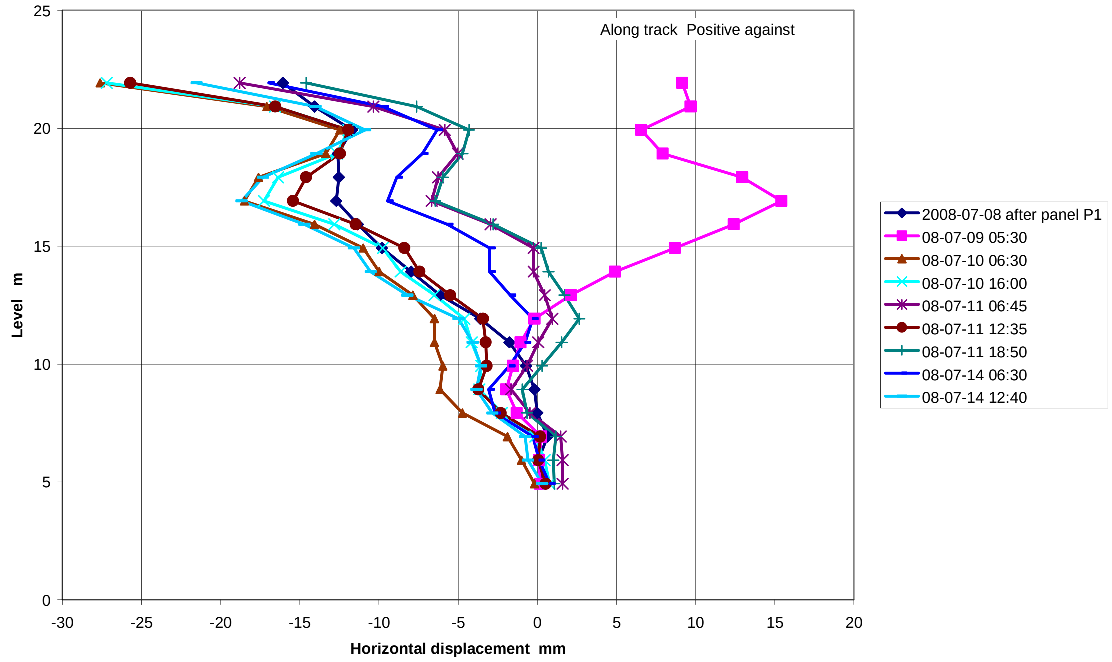


Figure 136 – Horizontal displacements along the railway line versus level for the inclinometer located at the western toe of the embankment at different dates to the end of the installation of panels at 14th July 2008.

In Figure 137 results including only the dates after the end of installation 14th July 2008 are shown for the transverse direction. In Figure 138 the same dates are shown for the direction along the track.

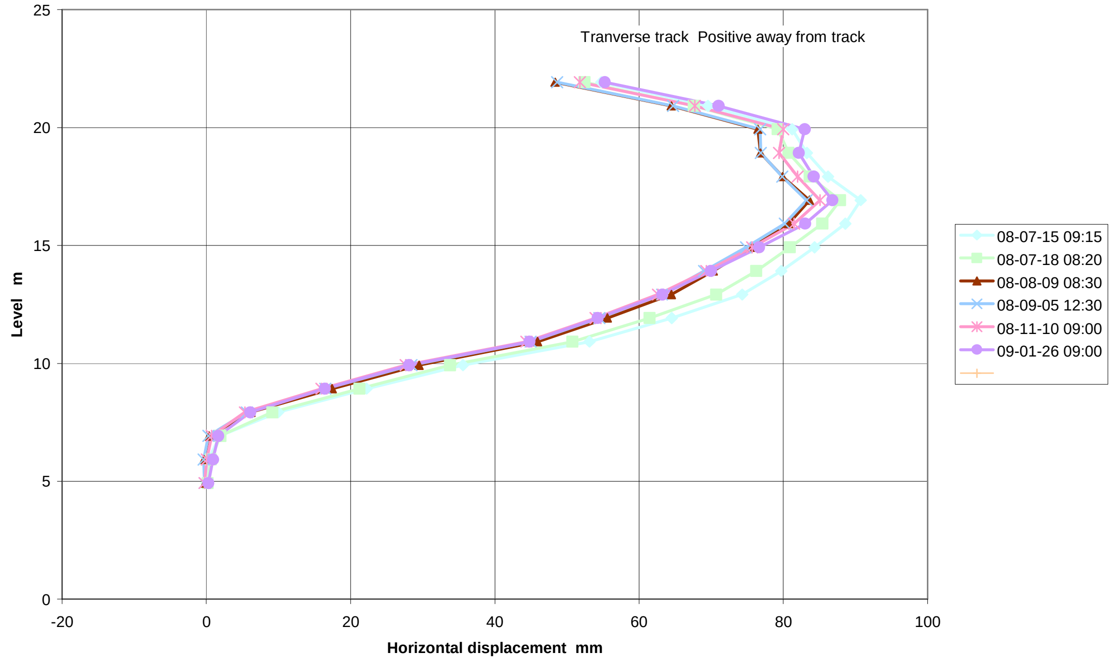


Figure 137 – Horizontal displacements transversal to (perpendicular to) the railway line versus level for the inclinometer located at the western toe of the embankment at different dates after the installation of panels at 14th July 2008.

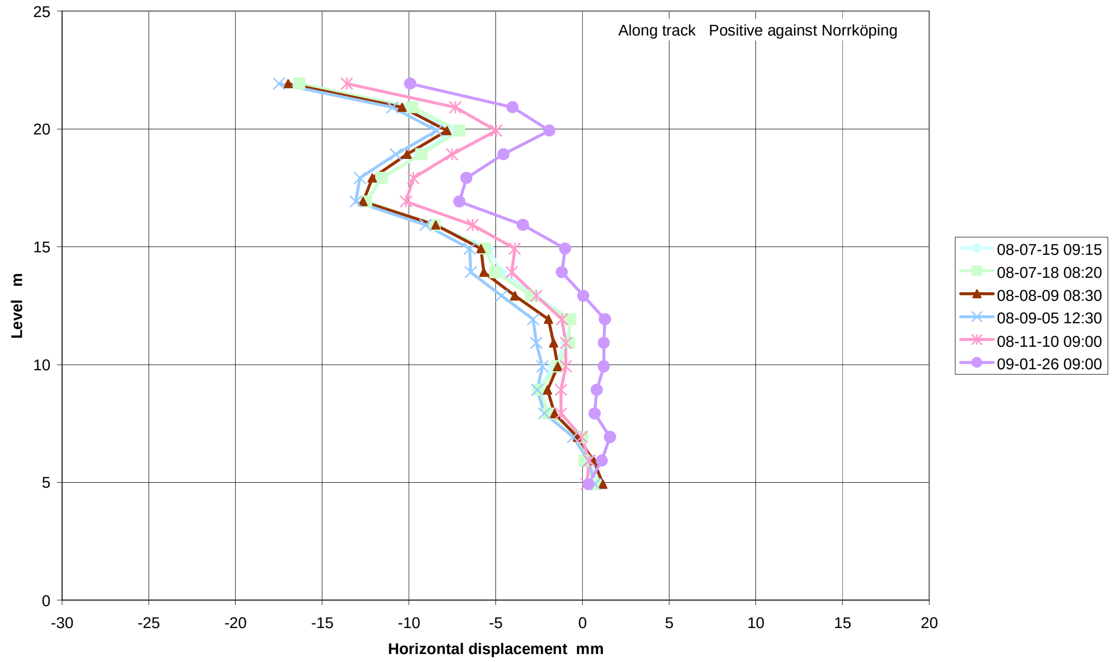


Figure 138 – Horizontal displacements along the railway line versus level for the inclinometer located at the western toe of the embankment at different dates after the installation of panels at 14th July 2008.

In Figure 137 and 138 the change in horizontal displacement is in the order of 5-10 mm from the date 15th July 2008 to 26th January 2009.

In Figure 137 are presented the results for the inclinometer at the eastern side of the embankment for a direction transversal to (perpendicular to) the railway line and in Figure 140 for a direction along the track. The results are presented as horizontal displacement versus level for different dates.

The results indicate small horizontal displacements and there is trend of a small effect (2-4 mm with direction from the track) of the installation of the panels in July 2008. There is also a trend of a horizontal movement towards the installed panels after 14th July 2008 above level +13 and at the top the trend is about 4-6 mm (with direction towards the eastern track).

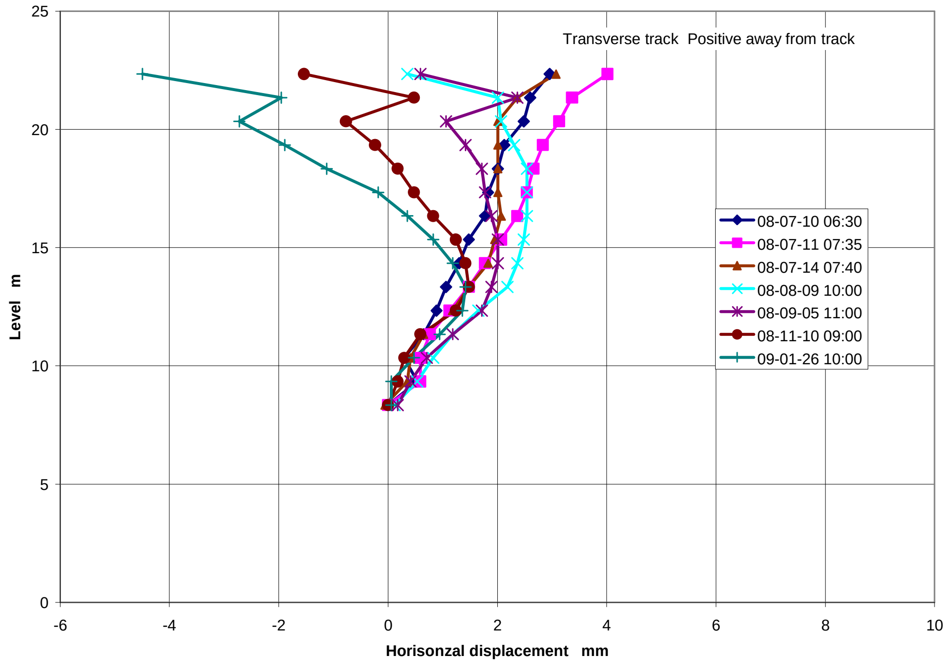


Figure 139 – Horizontal displacements transversal to (perpendicular to) the railway line versus level for the inclinometer located at the eastern toe of the embankment at different dates.

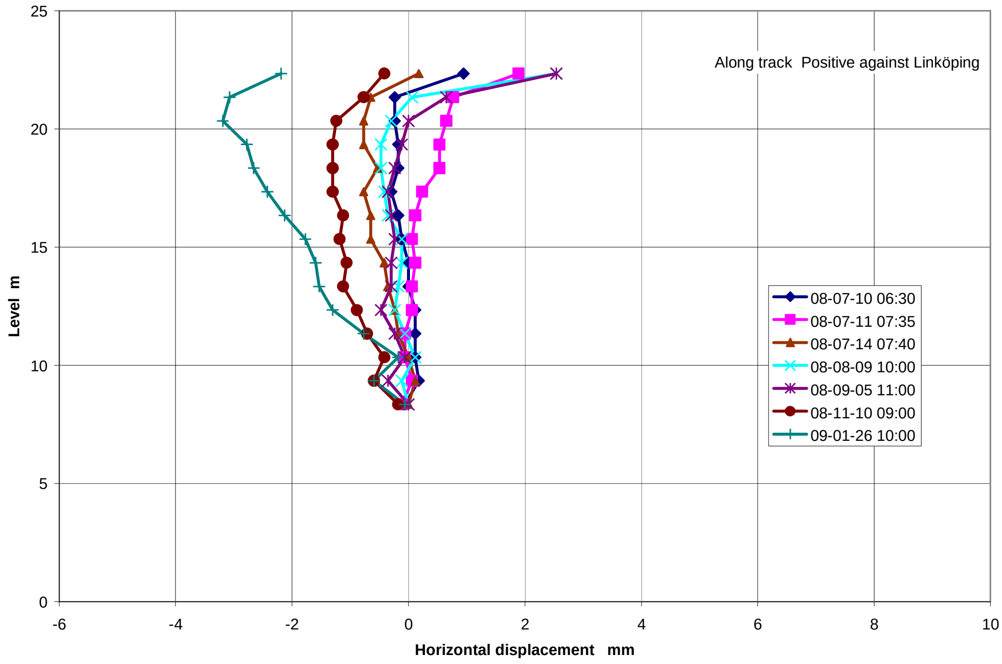


Figure 140 – Horizontal displacements along the railway line versus level for the inclinometer located at the eastern toe of the embankment at different dates.

3.7.7 Changes in pore pressure an ground water level in soil

Measurements of the pore pressure in the soft soil at four levels and the ground water level in the frictional material below the clay have been performed both in the test section km 193+900 and at a reference station considered not to be influenced by the installation of columns.

In Figure 141 the results in the test section are presented as corresponding ground water level versus time.

The installation of the panels in July 2008 caused a substantial increase in pore pressures and at level 10 m below the original ground the corresponding ground water level was in the order +31.0. This gives a pore pressure that is in the same order as the total vertical stress in the soil. In the middle of September 2008 (two months after installation) the pore pressures had reduced to levels almost the same as before installation.

In the period September to December 2008 there was a decrease in hydraulic head for the depth 13 m and the ground water pipe in the frictional material (lower ground water basin). It should be observed that about 150 m east of the embankment a large work with a new pipeline for water and sewer had been performed and there was a station that demanded excavation down to rock and ground water lowering. This probably had some influence on the monitored ground water levels in the lower part of the soil profile.

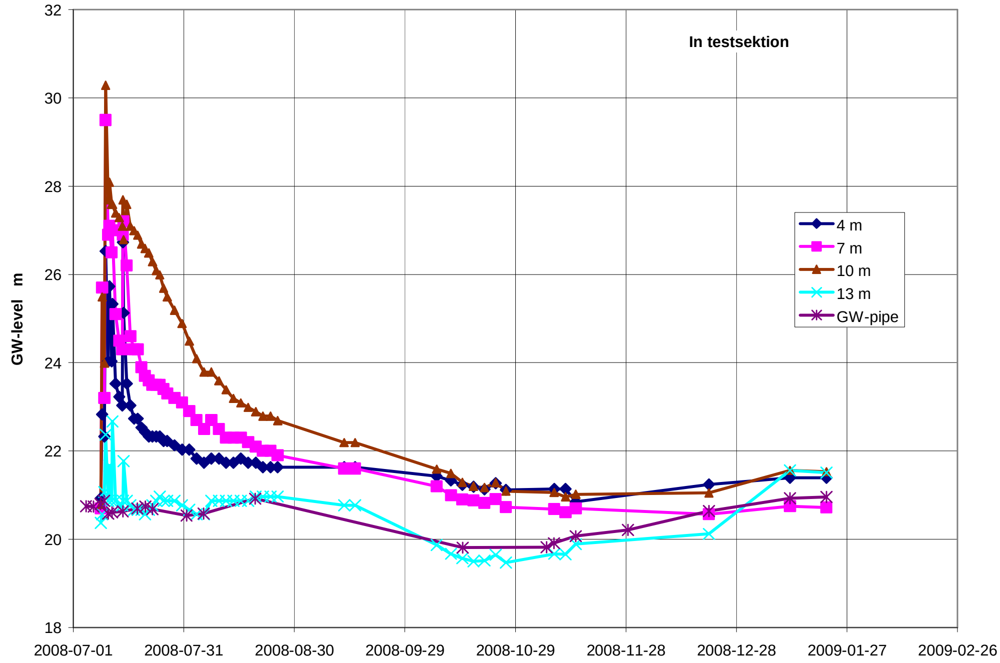


Figure 141 – Ground water level versus time in the test section km 193+900. Depth given as depth below original ground surface level +21.5.

In Figure 142 are presented the results from the measurements at the reference section. The results show the natural variation of pore pressure with time without any influence of the installation of the columns. The natural variation in the ground water level according to the measurements at the reference station is in the order of what could be expected. In the design it was assumed a ground water fluctuating from a ground water level at the ground surface (+21,5 m) in the winter and 2.0 m below the ground surface in the summer (+19.5 m).

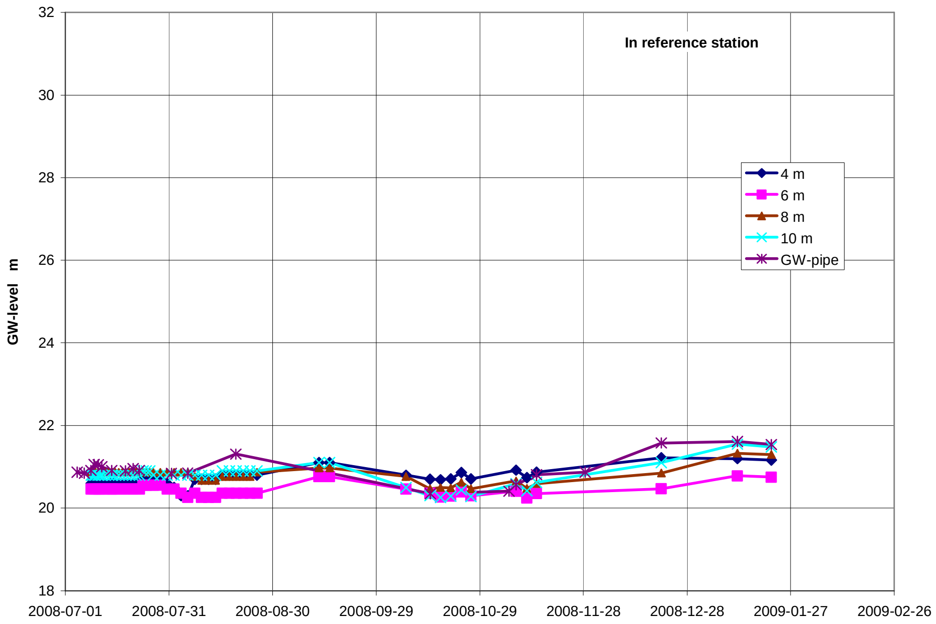


Figure 142 – Ground water level versus time in reference station. Ground level at +21.5 m. Depth given as depth below ground level.

3.8 Evaluation of results

3.8.1 General

The performed test with installation of panels of inclined overlapping columns of stabilized soil with the deep mixing method indicates that this is a possible alternative for stabilization measures for subgrade of existing railways. An uncertainty exists in how to verify that the geometry of the panels is as designed. Today there is no known method to do this when the columns are installed. In other methods for soil improvement like the jet grouting technique a device for measurement of the position of the injection rod just above the lower part of the rod is used. However this method has so far not been used in Sweden for deep mixing with the dry method. In the full scale test, an equipment mounted to the mixing tool for measurement of inclination was tested but it did not work properly.

To succeed with a satisfactory geometry of the panels there should be high demands first on the working platform being horizontal with good precision and on the positioning of the machine for installation. The angle towards the vertical and also the position in the horizontal plane should be checked to assure that when the crane is moved to produce a new column the crane is not twisted in the horizontal plane. There could be great errors in panel geometry if the position of the crane mast is not appropriate.

All the performed tests like the traditional penetration tests of columns and the geophysical test indicate that the geometry of the panels were as intended but this has not been actually proven.

The design of the strengthening measures was based on conservative assumptions of the strength of the stabilized soil based on investigation results from tests on samples that were mixed and stabilized in laboratory. The actual behaviour measured in-situ when producing the columns shows that the soil in the area of the panels has higher strength than in the area not subjected to preloading from pressure berms and embankment. There were a lot of interruptions in the work caused by problems with the too big overlap and that the amount of binder was too large for the machinery at hand. It was decided to reduce the binder quantity and to decrease the overlapping between columns to be able to produce more homogenous stabilized panels.

The tests results indicate that the properties of the stabilized soil used in the design were possible to achieve in-situ. The upper part of organic clay had undrained shear strength of 50 kPa that is higher then the one used in the design.

3.8.2 Behaviour

The experience from the excavation of the pressure berm and replacement with a penetrable fill of gravelly sand was that by using small excavation sections of at most 3 m width no measurable influence on the position of the track was caused. It is important to remove any large objects in the soil that can create problems when installing the columns. It is always important to get parameters from machinery about maximal size of such objects that machine tool can safely penetrate. In the test area there were two large boulders that had been missed and which needed to be excavated afterwards.

The installation of the panels was performed using a manual control of the position of the crane mast and this resulted in a longer production time than for normal deep mixing.

The installation of the columns caused a considerable heave of the area above and around the panels (embankment and surroundings) but a tolerable heave of the railway track. The heave was in the order of what could be expected taking into account a binder quantity of 80 to 100 kg per m^3 of soil, see Section 3.3.2. The installation also caused some horizontal movements in the order of 80 mm, which indicates that the stability of the embankment was probably a little higher than assumed in the earlier performed stability analyses. The calculated safety factor was in the order of 1.1 which ought to result in a considerably larger horizontal movement when installing the columns.

The installation caused a high increase in pore pressures in the soil and the decrease in pore pressures after installation. Dissipation of the pore pressure to the same as before the installation took about two months. There is probably some influence on the pore pressures also from the nearby construction site where a pumping in the lower friction material was performed.

After the installation was finished 14th July 2008 the pore pressure decreased and a settlement started in the area around the installation of the panels. From 14th July 2008 to March 2009 the increase in settlement is in the order of 20 mm for the eastern track, 25-30 mm for the western track and about 40-50 mm in the area of the panels. All these values represent settlement at the measurement section km 193+900. The settlement along the track occurred in an area with a width of 40-50 m. In the stabilized area with a width of about 14 m the settlement of the western track is 20 mm at distance 6.5 m from the test section compared to 30 mm for the test section km 193+900.

The reason for the continued settlements is not clear but it should be observed that there was an increase in load in the area when the material in the pressure berm were excavated and replaced by a fill of gravely sand in the area for the pressure berm and also outside the area when a horizontal platform was constructed for the crane for installation of columns. Also the fill for the new transportation road gave new loads to the area. Those extra loads from the material transported to the area were probably one of the reasons for this ongoing settlement with time. The first two months after installation of columns, settlement also occurred as a result of the decrease of the excess pressure developed at the time of installation.

3.9 Conclusions, recommendations and future investigations

The performed test installation of inclined columns panels has proved that this method of soil improvement is a possible alternative as measure for increasing the stability of an existing railway embankment on originally soft subgrade.

Installation of subgrade strengthening can be performed under an operated railway embankment.

There is always need to take safety precautions and consider restriction for train speed or axle loads at the time of strengthening work.

The positioning of the installation equipment is a vital issue and for the future this ought to be further developed so there is no question about what geometry of the panel that has been created in the soil.

There was a substantial heave directly at installation followed by settlements in the area where the panels were installed which to a certain extent affected the embankment and the track. For application of this method in full scale, measurements of track geometry at the time of strengthening installation and afterwards are important and give information about necessary track levelling.

All works with stabilized soil should include test in situ to verify that the properties assumed in design actually are achieved in the field. It is not enough with results only from laboratory tests on samples stabilized in the laboratory.

Sometimes a combination of inclined and vertical columns could be favourable.

Complementary investigations will be performed and evaluated in 2009. The existing information on the quality and homogeneity of the columns need to be improved because we have only performed penetration tests in the stabilized soils. The aim of the future excavations is to give more information on this issue for the upper 2-3 m of the stabilized material. Excavation is intended to be performed in the area with the ring of twelve vertical columns and between two of the panels (probably panel P6 and P7. Core sampling is intended to be performed to 10-14 m depth in one column in the ring and in one of the panels P6 or P7. Those excavations will be performed down to 2-3 m length in stabilized soil (means depth of 3-4 m if possible). Core sampling is planned from stabilized soil. Laboratory work will include unconfined compression tests.

Since the result and evaluation of this complementary investigations will only be available after delivery of this report. The additional results are going to be presented separately in form of a short Banverket's report.

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5. Annex : bibliographic study on deep mixing methods

This annex presents a synthesis/state of the art of the different soil improvement works based on lime-cement columns or soil-cement columns undertaken a road or railway environment. These works are taken from the literature review. In the second part, guidelines and recommendations are proposed to implement this soil improvement technique in a railway environment. These recommendations are taken from the existing recommendations and from the two experimental works carried out within the INNOTRACK project by LCPC/SNCF and Banverket (see other sections of this report and D2.2.8 GL).

5.1 Some elements on the Deep-Soil mixing method

The hitherto development of different technologies and equipment used in Soil Mixing is difficult to follow without a certain generic classification system. Several similar systems have already been developed for this purpose, e.g. (FHWA-RD-99-138, 2000), (CDIT, 2002) and (European-Standard, 2005)). The classification format adopted herein is based on three fundamental operational characteristics also identified in the FHWA report. The distinction between wet and dry technologies with respect to the form of binder introduced into the soil is the most straightforward, and hence the most widely used format. In the dry mixing methods the medium for binder transportation is typically compressed air, while the wet mixing method works with a slurry of binders. Lime, cement or lime/cement mix are usually used as a binder. The second characteristic is related to the method used to mix the binder, i.e. by mechanical action of the mixing tool with the binder injected at relatively low velocity, by hydraulic action of the fluid grout injected at high velocity (jet grouting), or by a combination of both techniques (so-called hybrid mixing). The third basic characteristic reflects the location, or vertical distance of the drilling shaft over which mixing occurs in the soil.

A possible classification has been proposed by Topolnicki (2003), see table I.

Table I – Classification of Deep Mixing Method (Topolnicki, 2003)

Binder	Condition	How to feed	Method
Lime	Dry	Screw feeding (mechanical)	DLM - Deep Lime Mixing Method
		Compressed air (pneumatic)	DJM – Dry Jet Mixing Method
			Nordic method (Europe)
Cement			DJM – Dry Jet Mixing Method
	Wet	Pumping (liquid feeding)	CDM – Cement Deep Mixing
			European method – Flight auger type

The construction of deep in situ soil mixing can be carried out either in columns or by mass mixing volumes of soil. The mixing can use either the dry method (used in Sweden for instance) or the wet method (used in Europe, USA and South East Asia). Mass stabilisation uses dry mixing and is currently applied in Finland and Sweden.

When comparing technical features of recently used DMM (Deep Mixing Method) and SMM (Soil Mixing Method) machines and operational systems it should be kept in mind that the aforementioned methods have been developed while taking into account various demands and constraints of regional markets, as well as soil conditions prevailing in areas of potential application. Moreover, various operational systems also reflect different objectives of ground improvement and design approaches. Consequently, not all SM methods can be regarded as equivalents although all are based on the same overall concept of in situ soil stabilisation. Despite these variations, the main technical goal of any SM method is to ensure a uniform distribution of binder throughout the treated soil volume, with uniform moisture content, and without significant pockets of native soil or binder.

Using lime/cement mixture instead of lime will increase the stiffness of the treated soil and cause stress concentration in the columns, while at the same time the hydraulic conductivity is decreased.

There is a European standard for the execution of deep mixing, (European-Standard, 2005). This is a standardisation of the execution procedures for geotechnical works (including testing and control methods) and the required material properties. Guidance on practical aspects of deep mixing, such as execution procedures and equipment, is given in an annex of the standard.

Methods of testing, specification and assessment of design parameters, which are affected by installation, are also given in an annex.

The deep mixing method is a technology that mixes in-situ soils with cementitious materials to form a vertical stiff inclusion in the ground. A mixing tool is rotated down to the design depth. Once the design depth is reached, the direction of rotation of the mixing tool is reversed and the tool is withdrawn at a constant rate.

During advancement and/or withdrawal of the mixing tool, agents such as quicklime, slaked lime, cement and fly ash are forced into the ground. The agents, widely referred to as binders, may be introduced in the form of either a dry powder or wet slurry. The wet methods of deep mixing are usually designed to provide higher unconfined compressive strengths values than the dry method. The MDM (Modified Dry Mixing) switches seamless from wet to dry during each individual installation. This technique facilitates penetration of stiff soils, fluidises low plastic clays as well as ensures the complete hydration of the added binder. A schematic of the deep mixing installation process is shown in Figure 1. Figure 2 shows a photo of installation equipment.

The deep mixing method produces columns in the ground that can be installed singularly, or in rows, grids, or blocks. The design spacing, diameter and length of columns depends on such factors as the allowable total and differential settlements, and the required capacity to prevent stability failures. Single dry-mixed columns are typically spaced at 1.0 to 1.6 m, center-to-center, and diameters range from 0.4 to 1.0 m. Single wet-mixed columns are typically installed with diameters ranging from 0.9 to 2.4 m. The maximum depths of treatment for the dry and wet methods of deep mixing are about 35 m and 40 m, respectively.

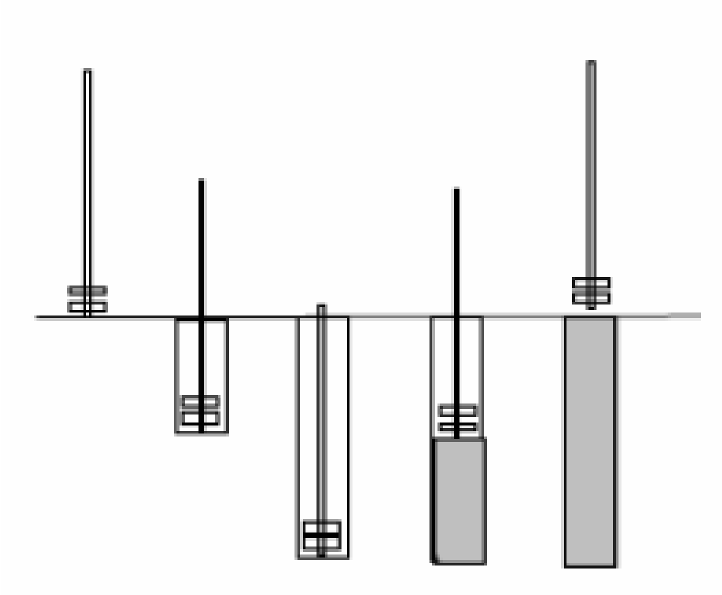


Figure 1 – Schematic of soil mixing installation (SGF, 1997).

Figure 2 – Dry deep mixing equipment (Holm and Smith, 2006)

Grouting as well as soil mixing methods have been used for embankment stabilisation with varying success. Their main advantages are that they are relatively low-cost compared with structural measures and are generally less disruptive to the embankment as only light equipment is deployed. However, there is always more uncertainty as to their performance, the extent of ground that is treated and their durability. They may, however, prolong or maintain a given level of serviceability until a more extensive repair can be conducted. As with all other remedial or preventative measures, knowledge of the ground conditions and failure mechanisms is essential to ensure effective and successful treatment.

For Wet soil mixing method, a slurry of binder and water is generally used. Mostly cement is used as the binder. Prior to the stabilisation process the binder is mixed with water to achieve a slurry.

As far as the stabilisation process is concerned, in the deep stabilisation process the soil is mixed in columns. For both wet and dry processes the binder is injected into the soil through a hollow pipe to a nozzle in the mixing tool. With dry mixing, binder is fed to the mixing tool only as it is withdrawn from the target depth of mixing whereas with the wet mixing the binder is supplied during both penetration to and withdrawal from the target mixing depth. By rotating the mixing tool and injecting the binder, the soil is mixed with the binder and a soil mixed column is formed as the pipe is lifted.

In the wet mixing method the addition of the wet binder slurry can cause heave or production of spoil at the surface. In practice the spoil fills the space left by the withdrawal of the mixing tool. The degree of spoil production appears to be related to the ratio of area to be treated to the sum of the areas of the sections of the columns. As this ratio raises so does the volume of spoil produced.

5.2 Deep mixing method for railway track subgrade improvement

In 2004, Moseley and Kirsch have published the second edition of Ground Improvement by Blackie Academic and Professional.

Applications of wet and dry DM in Southeast Asia include several important projects completed in Taiwan, Singapore, Hong Kong and Thailand, generally in cooperation with Japanese contractors. In 2001, Hercules Grundlaggning AB carried out ground improvement works for oil tank foundations in Vietnam (Forsberg, 2002), and LCM-Keller stabilised the soil beneath the realignment of a railway line in Malaysia (Raju et al., 2003).

5.2.1 Sweden

In the railway context, several experimental test sites were used in order to test the feasibility of the Deep Mixing method.

In the western part of Sweden 80 km of a new motorway and a new high speed railway have been constructed on soft, high-plastic clay. In order to reduce the settlements and improve the stability conditions of the embankments, lime-cement columns have been used extensively (about 9 million linear meters of columns). Already during the feasibility studies, in year 2000, a large amount of test-columns were installed in order to study their geo-mechanical properties (strength, stiffness, permeability).

In order to get a better base for design, it was decided to create three full-scale test embankments, founded on lime-cement columns (LCC) not reaching firm bottom (i.e. “floating columns”). These were constructed in year 2001 and are located in Nödinge, Stora Viken North and Surte (Olsson et al., 2008) and (Alen et al., 2005b).

The test embankments were considered necessary due to lack of reliable design methods for estimating consolidation settlements of embankments founded on floating columns. The embankments were heavily instrumented in order to study their settlements. Measurements were made of the variation of the settlements with depth in the columns, in the clay between the columns and in the clay below the columns respectively. Additional measurements included pore pressure changes in the clay below the embankments as well as horizontal displacements in the clay just outside the embankments. In order to evaluate the results of the measurements, engineering judgement and 1D analytical analyses were combined with 1D, 2D and 3D FEM analyses. Based on such analyses, simplified design methods for calculating consolidation settlements were developed to be used within the upcoming construction. Furthermore a comprehensive, yet relatively simple, design method was developed in order to be used for more general conditions of lime-cement columns not reaching firm bottom.

Nödinge

For the Nödinge test embankment, the water content was about 100%, the clay content above 50% and the hydraulic conductivity was about 5-10 10^{-10} m/s.

The Nödinge test embankment is located about 25 km north of Gothenbourg (Göteborg). The test embankment had a length at the crest of 25 m and width of 13 m. At Nödinge the columns where installed in May 2001 about 7 months before the first load increment. A total of 153 columns where installed in a quadratic pattern with every other column being 12 m long and the rest 20 m long. The lime cement columns were constructed with the dry mix method and consisted of 50% Lime and 50% cement with 90 kg/m3. The columns had a diameter of 0.6 m, see Figure 3. The total height of the embankment was about 2.8 m.

There were a total of 7 bellow hoses, 14 piezometers, 4 settlement hoses and 1 inclinometer.

Two load increments were applied to the embankment, each approximately 25 kPa. The first load increment was chosen so that no critical stresses would be exceeded in the columns. This load increment was allowed to act for a year and a half and was approximately 25 kPa (about 1.5 m high).

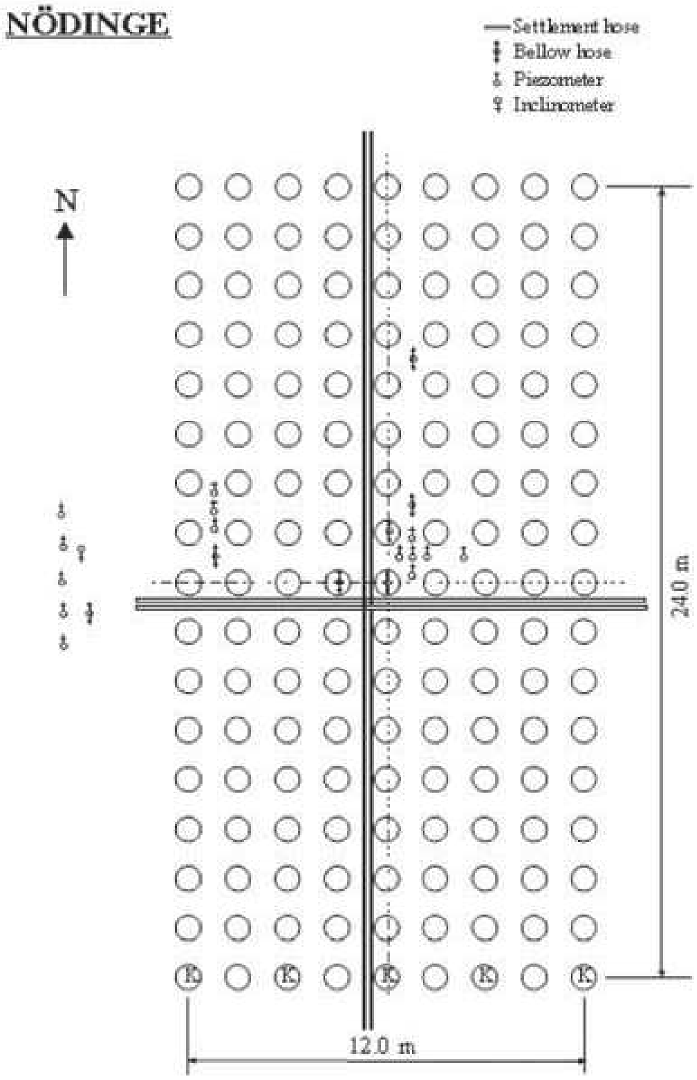


Figure 3 – Installation pattern for lime/cement columns and placement of gauges at the Nödinge test embankment (Olsson et al., 2008)

Figure 4 shows the results from the bellow hose measurements in the centre of the test embankment.

As can be seen in Figure 4, the settlement in the clay and in the lime cement columns, from a depth 3–4 m, are almost identical. This implies that equal strain is valid for a column and its surrounding clay, i.e. plane sections remains plane. It could also be seen in Figure 4 and Figure 5 that above 12 m, apart from the uppermost 2–4 m, the ground settles comparatively little. While between the depths 12 to 20 m, settlements are larger. In the deeper part there is only one column per 4.5 m^2 compared to one column per 2.25 m^2 in the upper 12 m.

When looking at the compression in the middle of the test embankment for the upper 12 m (3-12 m) the compression is about 30 mm after about 6 years. The corresponding compression for the lower 8 m is about 60 mm. The compression of the clay layer beneath the stabilized soil is about 45 mm.

In Figure 5, settlement with time is shown for different depths of the soil profile. One can see that the settlement is continuing with an almost constant rate. In addition, we also notice that between the depths 16 m to 20 m, 20 m to 22 m and for 26 m to 30 m there is still an increasing difference in the last measurements. This implies that settlement is still in progress between these layers.

In Figure 6, we can notice that the installation of the columns affects the excess pore-pressure to a very large amount, at least in the vicinity of the installation. For each of the load increments, about 25 kPa each, there is an increase in the excess pore-pressure. However, it is far from what would be expected according to the Swedish practice. Partly this could be explained by stress redistribution normally not taken into account.

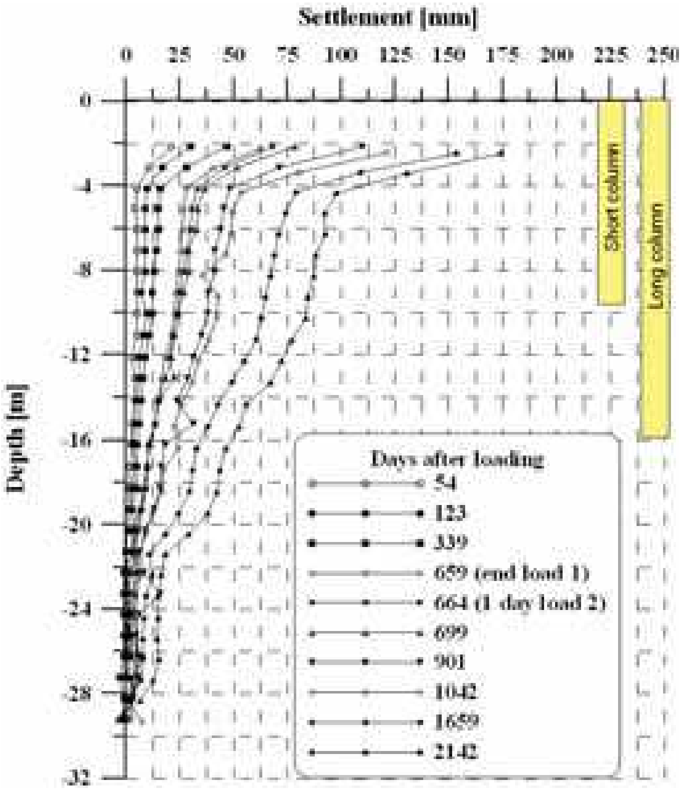


Figure 4 – Measurements from bellow hose in the center of the test embankment. Both in the clay between the LCC and in the 20m long LCC (Olsson et al., 2008)

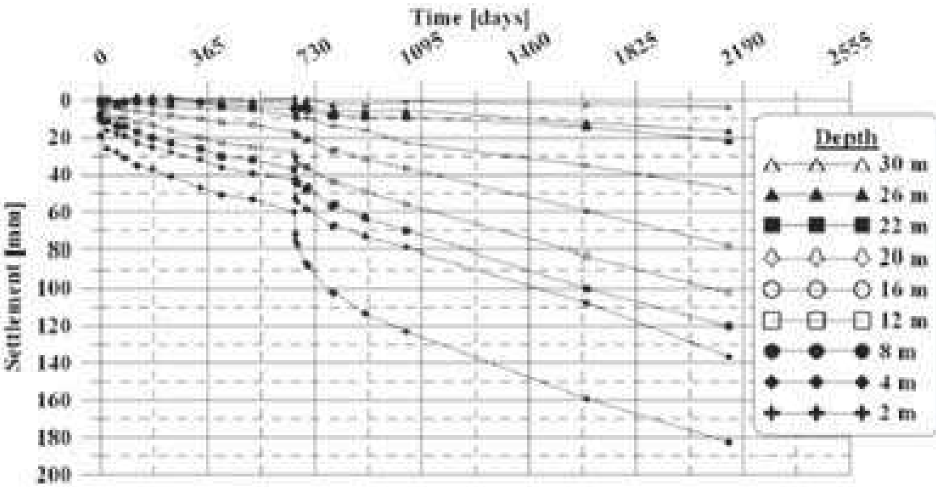


Figure 5 – Settlement with time for different depths from the bellow hose in the center of the embankment in the clay (Olsson et al., 2008).

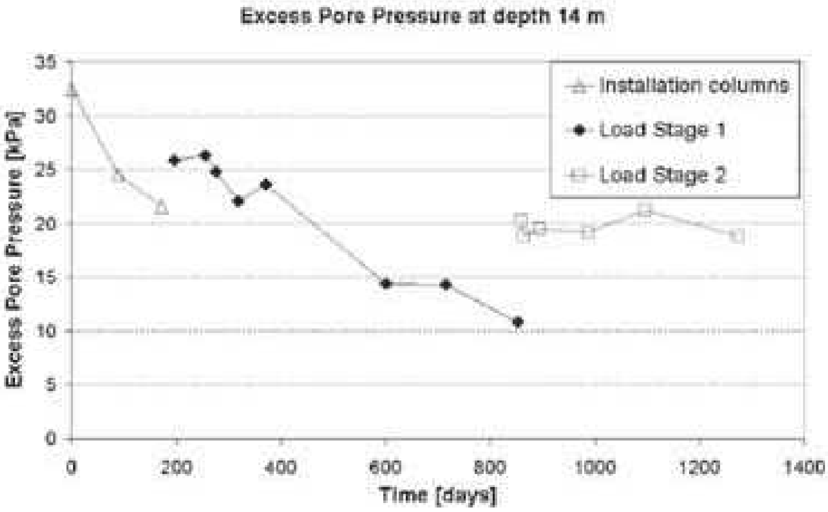


Figure 6 – Excess pore pressure with time at the depth 14m in the center of the test embankment (Olsson et al., 2008)

It is obvious that the relatively large settlement in the upper 2–4 meters of the soil profile indicate that that the quality of the columns in this layer is inferior to the rest of the columns. This was also confirmed when load tests were made on single columns and the columns were examined by Baker et al. (2005). The columns from 2–4 m and down are however of very good quality. This indicates that about half of the total settlement comes from the upper 2-4 m. The settlement has no tendency to slowing down and it will probably go on for quite some time. This indicates that the settlement magnitudes in the clay layer beneath the stabilized soil is strongly dependent on the clays creep properties (Baker et al., 2005).

Load tests on single columns indicated that the modulus of the columns is much higher than Swedish practice, (Baker et al., 2005). The settlement measurements justify the assumption of equal strain in the clay and in the columns. As from about 2–4 m and downwards the bellow hose settlement gives almost identical settlements. The measurements indicate that the critical stress was not exceeded in the columns, more than perhaps for the upper 2-4 m. They show that the installation of the LCC contributes to a quite large increase of the excess pore pressure. One can also see the effect of the two load increments.

Stora Viken North

Another test site to study the feasibility of soil reinforcement method in a railway context is Stora Viken North in Sweden ((Baker et al., 2005)). The soil profile consists of 15 m layer of slightly overconsolidated soft clay followed by a layer of about 2 m thick cohesionless material, which is fairly stiff. Under the cohesionless material another clay layer of about 20 m can be found. Lime/cement columns were installed down through the first clay layer, and thus had a length of about 16 m with the tip resting on the cohesionless material. The column diameter was 0.6 m and they were placed in a quadratic pattern at a distance of 1.5 m. The instrumentation consisted of settlement gauges, bellow hose settlement device for measurement of settlements at different depths, settlement hose for monitoring of settlement across the fill and piezometers. The bellow hoses were installed in the clay between the columns as well as within the columns The embankment, 12 m wide and 25 m long, was constructed in two lifts, where the first lift, applied in September 2001, corresponded to the working load for the future embankment, about 18 kPa. A second load was added almost two years later, resulting in a total load of 40 kPa, close to the estimated creep load capacity of the columns. Typical results are given in Figure 7 and Figure 8.

The pore pressures were measured from September 2001 until the Spring 2005. Piezometers were placed in the clay between the columns, in the columns and in the clay layer below the columns. A reference station was also installed. Apart from the seasonal variation, comparatively small pore pressure increases were measured. The pore pressure increase in the clay between the columns was less than 5 kPa during the first lift and up to about 10 kPa in the second lift.

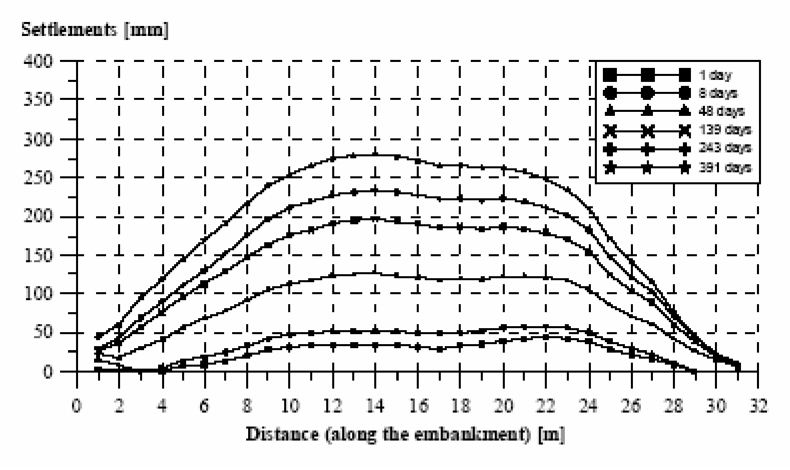


Figure 7 – Results from measurements of settlements of the ground surface under the embankment, during the second lift at Stora Viken North (Alen et al., 2005b)

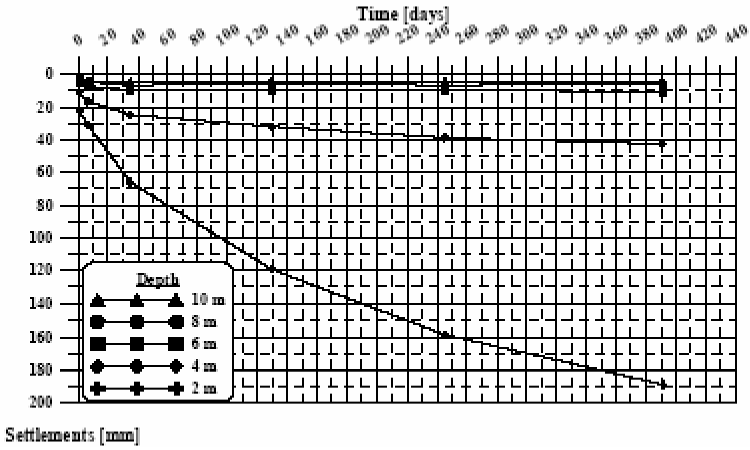


Figure 8 – Results from measurements of settlements in the clay, under the embankment, plotted versus time, during the second lift at Stora Viken North (Alen et al., 2005b).

Again, it could be observed that the upper 2 m settle a lot and at least 2/3 of the settlement occur in the upper 3 m. During the second increment, it is also obvious that the settlement is substantial down to about 6 m but for greater depths they are less than 1 to 2 cm. The larger settlements in the upper 6 m could be caused either by creep deformation, primarily in the clay, or by the fact that the creep load in the column is reached, so that the clay in between gradually consolidates. Measurements with an inclinometer confirm that the horizontal displacements are comparatively small, maximum about 6 cm for the two load increments, which roughly corresponds to an average settlement for the embankment of about 4 cm. During the first increment, practically no settlement occur in the clay below the layer of cohesionless material, while one can see an indication that some settlement is starting to develop during the second increment. Although these settlements do not affect the lime/cement stabilized clay, they are of great interest for the final design of the railroad.

Load test results on columns are shown on Figure 9. Two procedures were tested with bottom plate and without bottom plate.

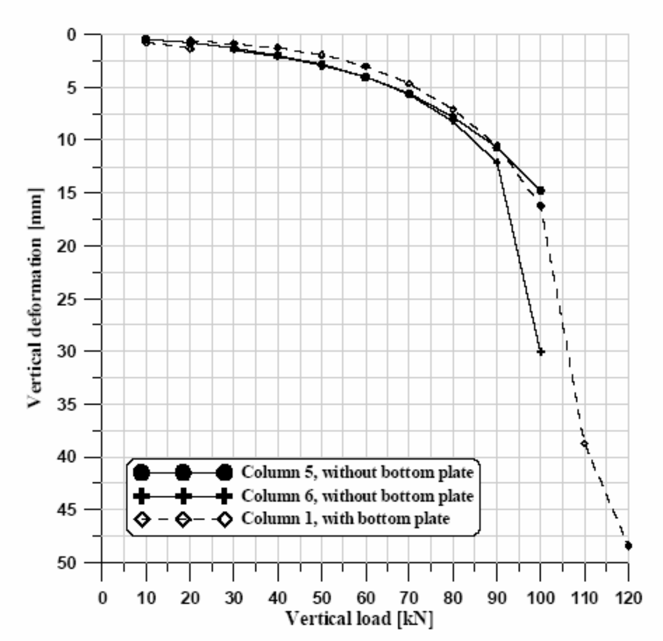


Figure 9 – Vertical deformation at the top of the columns a function of load (Baker et al., 2005)

From the mechanical test results, the modulus for the columns can be evaluated. These modulus represent boundary conditions which are somewhere in between the odometer and the triaxial case, as an expansion in horizontal direction will result in an increase in horizontal stresses. But the difference between the triaxial and oedometer modulus is probably fairly small.

For the centre 3 m of the column the evaluated modulus is very high for the first load steps, in the order of 200 MPa, while it decreases somewhat for stresses around 300 kPa. Then, when the column starts to crack, close to the creep load a significant drop in the modulus is obtained. In all of the analysis, the mobilized skin friction at the periphery of the column has been accounted for. It might be reasonable to assume that the column behaves as a linear elastic – perfectly plastic material.

As far as the permeability tests ar concerned, the hydraulic conductivity value for column 7 was about 0.5 to 1 10^{-8} m/s while, for the other column it is closer to 5.10^{-8} m/s. This indicates that the hydraulic conductivity of the columns is 5 to 50 times larger than that of the intact clay.

Surte

Surte is situated about 20 km north of Gothenbourg (Göteborg) and a few kilometres south of Stora Viken and the soil at this test site location consists of a similar clay, down to about 30 m. However, there is no frictional material in the center of the layer, as was the case for Stora Viken North. Lime/cement columns were installed in a quadratic pattern covering an area of 25 by 12 m^2. Half of the columns were 20 m long and the other half 12 m long. Measurements of settlements and pore pressures during and after construction of the embankment were measured in the columns, as well as in the clay in between. The embankment was also constructed in two lifts. Typical results are given in Figure 10. At this site the settlements are larger in the clay compared to the columns down to about 6 m both load increments. The settlements are comparatively small even if, also here, the upper two meters seems to be of a somewhat poorer quality than the rest.

The generated pore pressures were very small and the rate of settlement was of about 5 mm/year for the whole clay layer at the end of the measuring period.

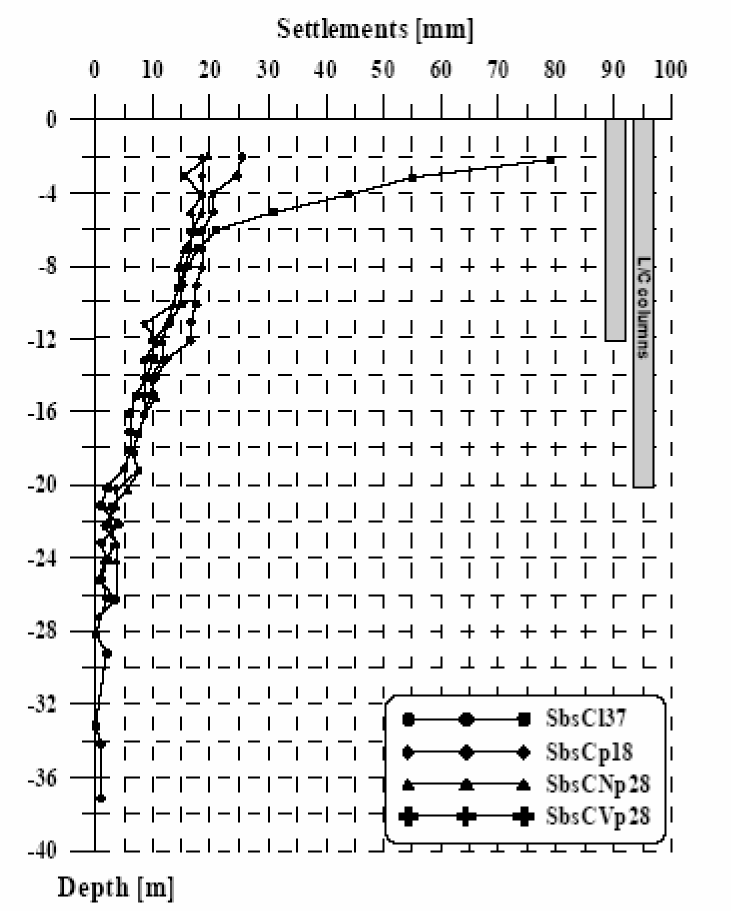


Figure 10 – Results from measurements of settlements in both the clay under the embankment (SbsCI37) and the lime/cement columns (SbsCp18, SbsCNp28 and SbsCVp28), during the first lift at Surte (540 days) (Alen et al., 2005b).

Discussions on the three test embankments

It is obvious that the comparatively large settlement in the upper two meters of the profile indicate that the quality of the columns in this layer is inferior to the rest of the columns. This was also confirmed when load tests were made on single columns and the columns were examined closely, as reported in (Baker et al., 2005). The bulk of the columns, from 2 m and down, are however of a very good quality. Load tests and hydraulic conductivity tests on single columns indicated very high modulii for the columns, much higher than recommended by Swedish practice and these modulii are basically confirmed by the embankment tests. A more thorough analysis will be made as the new method for settlement calculations (Alen et al., 2005b) will be tested and evaluated. It is obvious that the method for settlement calculations, according to Swedish practice, would result in far too large settlements, compared to what was obtained for the test embankments at Stora Viken North, Surte and Nödinge. The reason for this is primarily due to the assumption concerning load distribution, but also that much lower modulus for the column is prescribed. The settlement measurements justify the assumption of equal strain in the clay and the columns, as from 2 m and downwards the bellow hose settlement gives almost identical settlements. For Surte equal strain theory seems to be valid from 6 m and down. Furthermore, the measurements indicate that the creep load was not exceeded at Nödinge except perhaps for the upper 2m. If the large settlements in the upper 2 m are caused by the fact that the creep load in the columns is exceeded, or simply by poor quality of the columns, will be further investigated.

The test embankments were designed and performed in order to improve the accuracy of the final design of the new highway and high speed railroad track. It was a part of a contract between the Swedish National Road Administration, the Swedish National Railway Administration and the Consultant Group. The information rendered by the measurements and the analysis of settlements will be of great value for the future work, and no doubt, it will greatly influence the design. Furthermore, the instrumentation was expanded as part of a research project and the results form an important base for the development of a new method for settlement calculations of lime/cement stabilized soil. It can be concluded that the columns have a very high stiffness after a couple of years. The clay and the columns interact as expected and the assumption of equal strain is justified, compare Broms (1984) and Carlsten (2000) mentioned by (Alen et al., 2005b). However, the upper 2 m the columns are of an inferior quality and this layer settle substantially more compared to the rest of the stabilized soil. This phenomenon needs further attention and modifications in the production, such as the use of a protection layer of up to half a meter, should be considered.

Ledsgård Site

The Ledsgård site is situated north of Kungsbacka about 25 kilometres south of Göteborg on the West Coast Line in Sweden. The site is near a river, the Kungsbacka ån, in the centre of a large plain. The ground is lowest close to the river and rises towards the north. At the Ledsgård site there is a very soft organic soil (gyttja) below the dry crust. Below the gyttja there is soft clay. The river is probably the origin of the gyttja layer described below. At some time, the river has probably been re-routed so that today it passes outside the gyttja area where the railway line passes.

From a geotechnical point of view, the Ledsgård site is rather special, with a pocket of very soft organic soil (called gyttja) of maximum 3 m thickness below the dry crust. This gyttja extends approximately 200 m along the track. The layer has its maximum thickness in the measurement section 24+265. The maximum cone liquid limit is 250 % and the maximum organic content 20 %. The gyttja is underlain by soft clay and the depth to bedrock is more than 60 m. Geotechnical investigations have been carried out in phases. Detailed information on the Ledsgård site, including the results of extensive field and laboratory investigations, is presented in Banverket (1998) and Banverket (1999) both mentioned by (Holm et al., 2002). In connection with the R&D project of 1997/98, the test section and its immediate surroundings were investigated with penetration tests, sampling, shear wave velocity measurements and laboratory investigations, including dynamic triaxial testing. The results were presented in Banverket (1999), mentioned by (Holm et al., 2002)). Additional soil investigations were carried out along a 400 m section with a potential need for countermeasures to reduce the vibrations, thus identifying the extent of the gyttja pocket. In the measurement section (24+265), the railway structure is approximately 1.4 m thick and consists of crushed rock (macadam). Figure 11 shows the soil conditions in the test/measurement section. Special soil investigations were performed. Cross-hole and down-hole measurements were made to determine the shear wave velocity in the different soil layers. The shear wave velocity is in the order of 40 m/s in the gyttja layer. In the underlying clay, the shear wave velocity is approximately 60 m/s, increasing to 90 m/s at 14 m depth. Figure 12 shows the measured shear wave velocity.

The main result of the additional soil investigation carried out in connection with the design of countermeasures is presented in Figure 13. The figure shows a section along the line with the gyttja pocket sketched in (based on 10 sampling points). The gyttja is subdivided into two groups with the natural water content above and below 150 %. The maximum water content observed is slightly in excess of 200 %. (200 % water content corresponds to approximately 17 volume-% solid material and 83 volume-% water, i.e. a 1:5 ratio of solid material/ water).

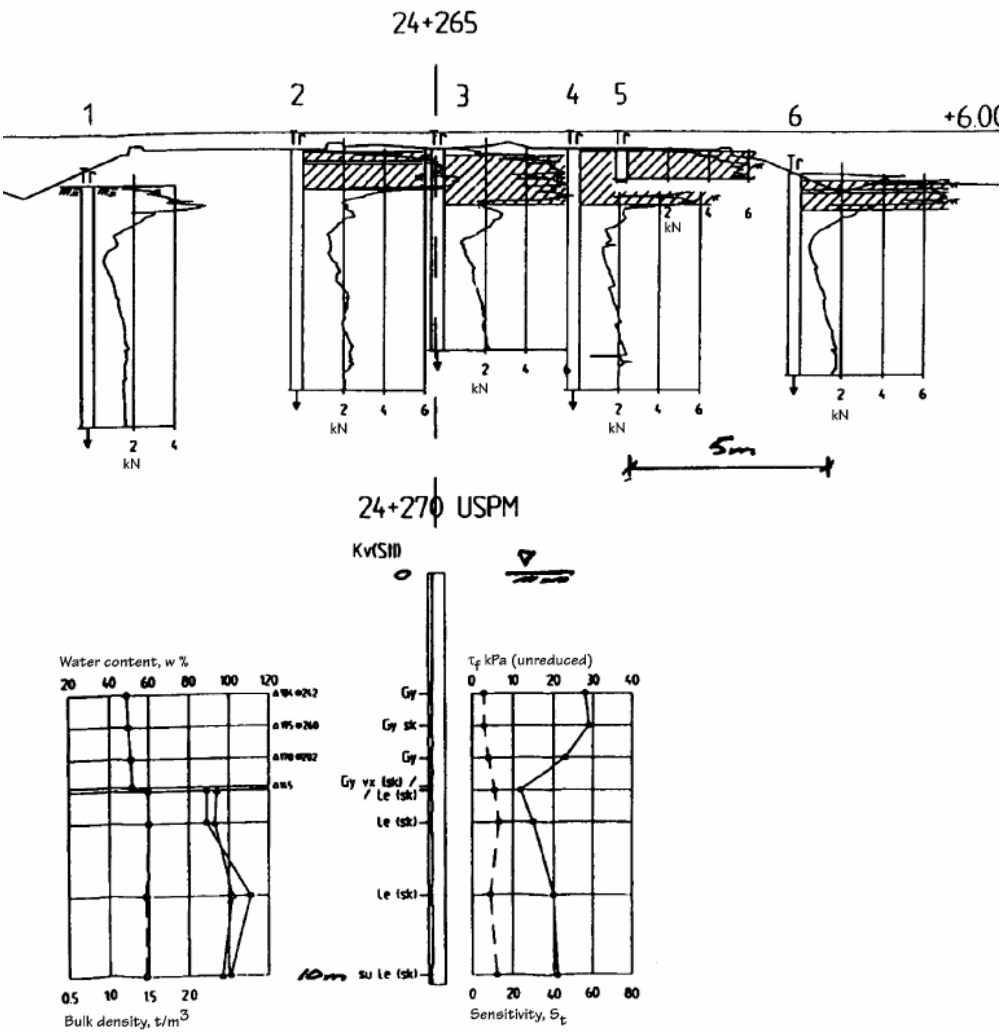


Figure 11 – Soil conditions in the Ledsgard test section (Holm et al., 2002)

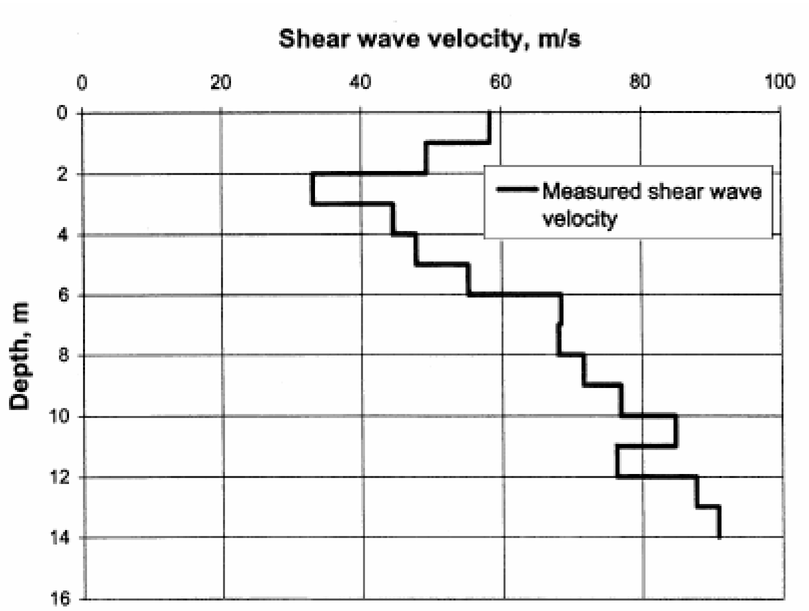


Figure 12 – Shear wave velocity at Ledsgard (Holm et al., 2002)

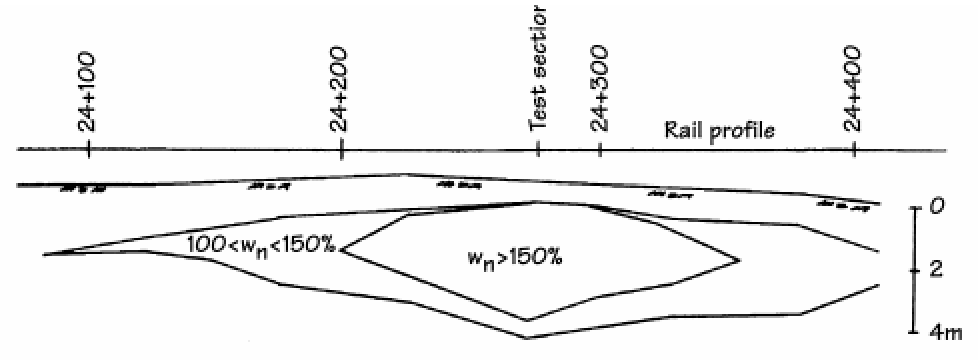


Figure 13 – Extent of the gyttia pocket along the track based on sampling at ten points. Natural water content indicated (Holm et al., 2002)

To follow the behaviour of the railway line at Ledsgård, Banverket carried out settlement measurements on the section by levelling the western track at regular intervals. The result of such measurements for an eight-month period in 1998/ 99 are shown in Figure 14. The figure clearly shows that the settlements start in Section 24+000. The figure also shows the result, in terms of relative level, of vibration measurements carried out under regular traffic at 160 km/h. The measurements were made at 50 m intervals by accelerometers mounted on 30 cm diameter concrete blocks cast between sleeper ends. These measurements clearly show the extremely soft behaviour in the test section and the local extent of high vibration levels.

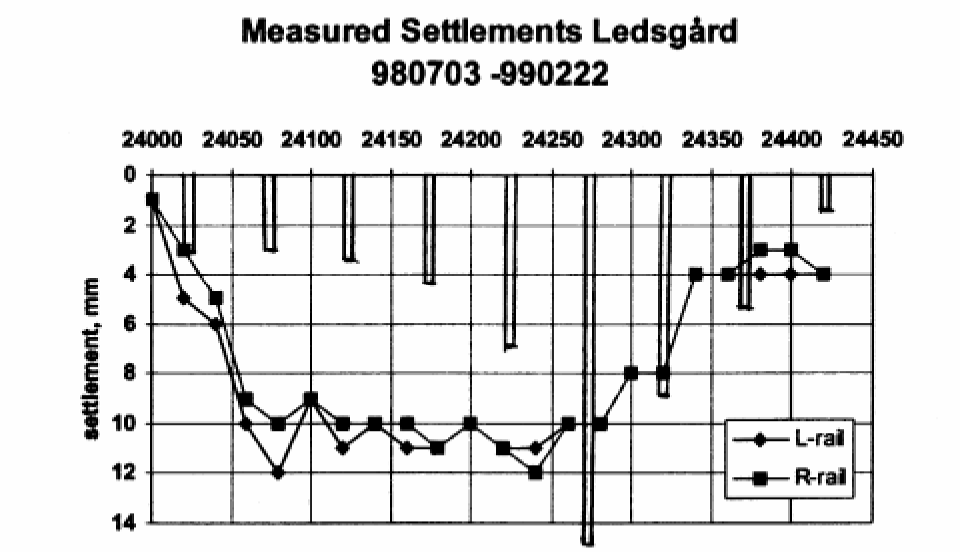


Figure 14 – Settlement observations of the western track at Ledsgard (both rails) (Holm et al., 2002)

Based on the good results of soil improvement with the dry deep mixing method (dry DMM) achieved in the area, also in gyttja type material, and the recommendation in the Technical Note mentioned above, the dry DMM (normally lime/cement columns) was a prime alternative from the beginning of discussions on countermeasures at Ledsgård. However, laboratory investigations had to be carried out to verify the effect of the improvement, especially in the gyttja. Other methods to limit vibrations at high speeds include a stiff, piled concrete deck and a stiffening beam in the railway structure itself. Certain calculations were carried out for the latter method of a beam structure corresponding in bending stiffness to a plate with a thickness of approximately 0.45 m.

The most important parameter is the stiffness of the various components of the railway structure and the ground. The stiffness, together with the geometry and density, governs the wave velocity, which in turn governs deflections and the critical speed of the whole system. The dynamic properties of the soil material at Ledsgård have been thoroughly investigated in the Banverket studies, (Banverket, 1998) and (Banverket, 1999). In regard to the material in the railway structure, the stiffness properties are deduced with empirical methods presented in the literature. The stiffness of the structural components such as the rail and concrete (beam solution) is quite well known. However, of prime importance in the design of the dry DMM improvement is the stiffness of the improved soil. To estimate this property, laboratory tests on samples of improved soil were carried out. The stiffness, together with the undrained shear (compressive) strength, was determined through a series of unconfined compression tests. The test results in a stress/strain curve from which the undrained shear strength and the elasticity modulus can be deduced.

Important topics in designing effective soil improvement with the dry DMM are to determine the amount and proportion of binder components as well as the required mixing energy. An initial recommendation, which proved so successful that it was kept throughout the project, was to use 150 kg binder per treated cubic meter of gyttja with the unslaked lime/cement in a proportion of 25/75. Outside the gyttja pocket, the amount was reduced to 120 kg/m^3 in a proportion of 50/50. The unconfined compression tests showed a very good effect of the treatment. Figure 70 shows the results in terms of undrained shear strength and elasticity modulus versus time for three prepared batches (two of gyttja and one of clay). Corresponding tests on untreated samples resulted in an undrained shear strength of 20 kPa and a Young's modulus of 1.1 MPa. (The modulus is deduced from a straight portion of the compression test curve corresponding normally to a compression in the order of 0.5 %). The compression at failure is approximately 1 % with a brittle failure mode.

The value of stiffness of the treated soil to use in the dynamic design of the railway structure/foundation is a complex and important question. In situ, the stabilised soil is confined and the stress/strain levels are very low compared to the laboratory tests. On the other hand, the mixing process is very different in the field compared to laboratory mixing.

Vibrations at high speed are primarily caused by the soft gyttja pocket. The gyttja pocket starts in approximately Section 24+150 and extends down to the river, see Figure 13. Due to observed settlements all the way back to Section 24+000, improvement was carried out from Section 24+000 to 24+372, where the existing dry DMM improvement commences. The first 150 m, from Section 24+000, is stabilised by dry DMM in a single pattern. From Section 24+150 down to the existing lime/cement columns close to the bridge, the columns are installed in a ladder-type configuration with two longitudinal walls extending (in principle) to 7 m and transverse walls extending to 6 m depth below the rail. To accommodate the existing reinforcement at the bridge and to take settlements into account every second column in the longitudinal walls extends to 13 m depth, as indicated by the figures within the "column" circles in Figure 15.

The demand for proper transmission to the stabilised soil required a good connection between the columns and the railway structure. This was achieved by installing the columns from a "working platform" to be removed after completed installation to expose the stiff column heads.

As mentioned above, the amount of binder was totally 150 kg/m^3 with the components unslaked lime and cement in a ratio of 25/75 in the "ladder" part from 24+150 on and 120 kg/m^3 with a binder ratio of 50/50 in the single column part

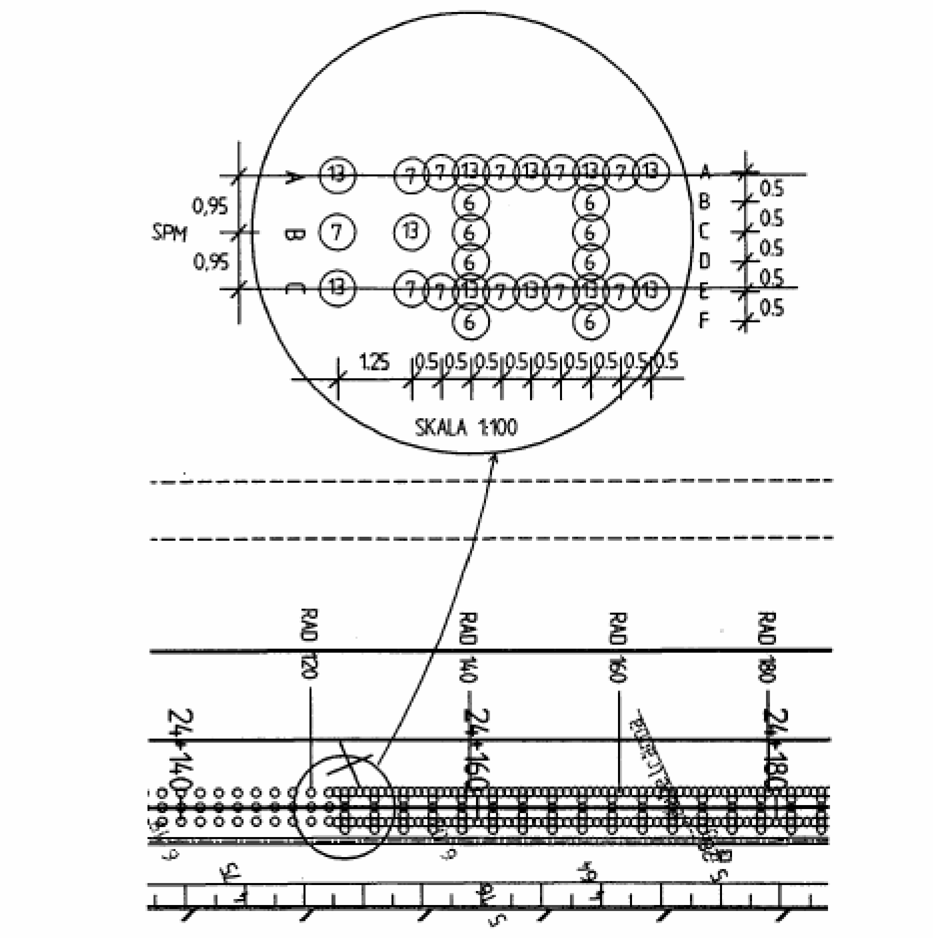


Figure 15 – Layout of soil improvement with the dry DMM columns (lime/cement columns). Also showing the transition at 24+150 from “settlement” stabilisation (from 24+100 to 24+150) and “high speed“ stabilisation. Column length below rail level given by figures within circles.

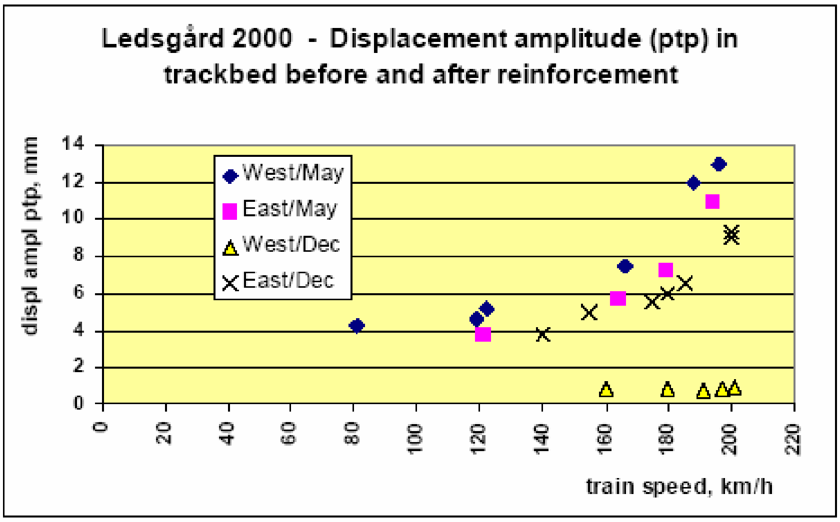


Figure 16 – Summary of measured displacements in the trackbed before (May) and after (December) reinforcement of the western track. Central part of X2 trains,, J&W (2001) cited by (Holm et al., 2002).

Vibration measurements in the tracks were carried out on both tracks before and at different times after the countermeasures. Measurements were performed on rails and sleepers as well as on deflections in the trackbed at sleeper ends. Vertical deflections were the prime object. The excitation source was primarily X2 trains and the Track Loading Vehicle (TLV). A summary of measured deflections in the trackbed on both tracks before and after the countermeasures is presented in Figure 16 (traffic on the respective/ measured track). The figure shows peak-to-peak deflections in the middle part of the X2 train travelling at various speeds from May (before) and December (after) measurements. "West" indicates the track where dry DMM improvement (lime/cement columns) was performed, whereas the “East” track is not reinforced. The results of the countermeasures are very clear in regard to the dry DMM improved track. The vertical deflections at low speed have been reduced from 4 mm to approximately 0.8 mm (practically constant deflections with speed after countermeasures), i.e. a reduction by a factor of 5. At higher train speeds, the reduction is even more pronounced, since the dynamic amplification of the trackbed movements has disappeared, resulting in a 15 to 20-fold reduction in vertical deflection amplitude. The vertical deflections in the eastern track were initially somewhat lower (about 15 %) than in the western track and have decreased further (15 – 20 %) as a consequence of the countermeasures in the western track. Thus, the vertical deflections in the untreated track after countermeasures in the western track are in the order of two-thirds of those of the untreated western track before the countermeasures. A comparison of early measurements of the reinforced track six weeks (September 2000) after dry DMM column installation with the December 2000 measurements (4.5 months after installation of columns) indicates that the vertical deflections decrease with time. The measured vertical deflection amplitude (peak-to-peak in the middle part of the train) was 1.0 mm in September 2000 and decreased to 0.8 mm in December 2000. The ratio between the lateral displacement amplitude along the track and the corresponding vertical deflections was essentially constant with train speed, 0.2 – 0.3, (J&W, 2001). The measurements were carried out on both tracks only before the countermeasures.

Measurements with the Track Loading Vehicle, (Banverket, 2001), showed that the track receptance at 1 Hz was reduced by a factor of 3.4 and 1.2 for the western and the eastern track, respectively, due to the countermeasures in the western track. The excitation and response were measured on the rail head mainly by means of accelerometers. The receptance was extremely high before soil improvement. There was a resonance of 2 – 4 Hz. The track secant stiffness showed similar results with an increase corresponding to a factor of 3.3 for the western track and a factor of 1.1 for the eastern track. The displacement of one sleeper end during train passages, before DMM improvement, is shown in Figure 17. The passage of each wheel is indicated. At the low speed train passage (80 km/h), a normal, although high, displacement pattern arose. Maximum displacement occurred when a wheel passed the sleeper.

For the high-speed train passage (192 km/h), extremely high displacement occurred with a different pattern. The maximum displacement occurred after the bogie passages. After the improvement with dry DMM, the displacement pattern was normal for all velocities tested (max 200 km/h).

When designing the countermeasures the extent of the gyttja pocket was an important boundary for the different layout of the dry DMM soil improvement. To study the varying vibration conditions along the reinforced track section, vibration measurements were carried out in three sections along the track using geophones placed 3 m from the western track, (J&W, 2001). Measurements were performed in Section 24+265 (the test section where essentially all measurements were carried out), section 24+175 (also with dry DMM wall stabilisation) and section 24+075 (with dry DMM columns in single pattern). Figure 18 shows the measured vertical particle velocity amplitude in the middle of the X2 train versus train speed before and after countermeasures. From Figure 18, it is clear that in the two southern sections there is a sharp increase in vibration level with increasing speed. In the northernmost section, however, there is no trace of such behaviour. This was expected, since the soft gyttja pocket does not extend to this section.

After the dry DMM soil improvement, the vibration amplitude decreases to approximately the same low level in all three sections.

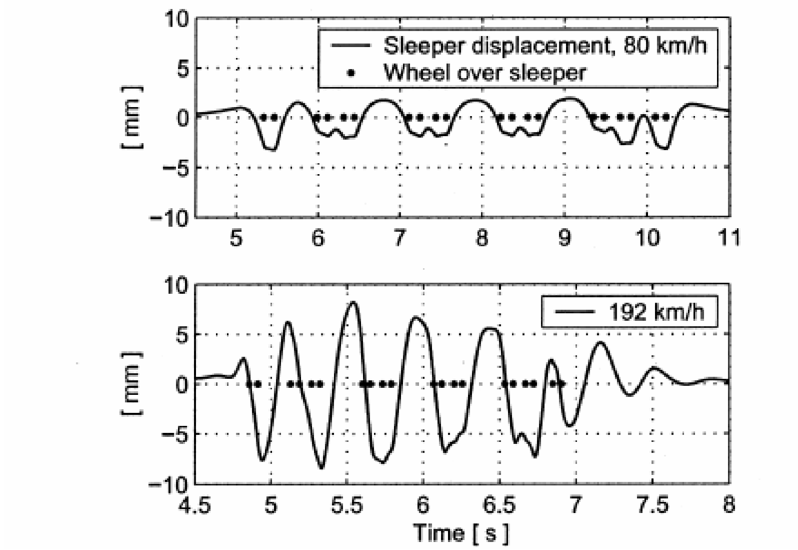


Figure 17 – Displacement of sleeper end, train passing at speeds of 80 and 192 km/h (Holm et al., 2002).

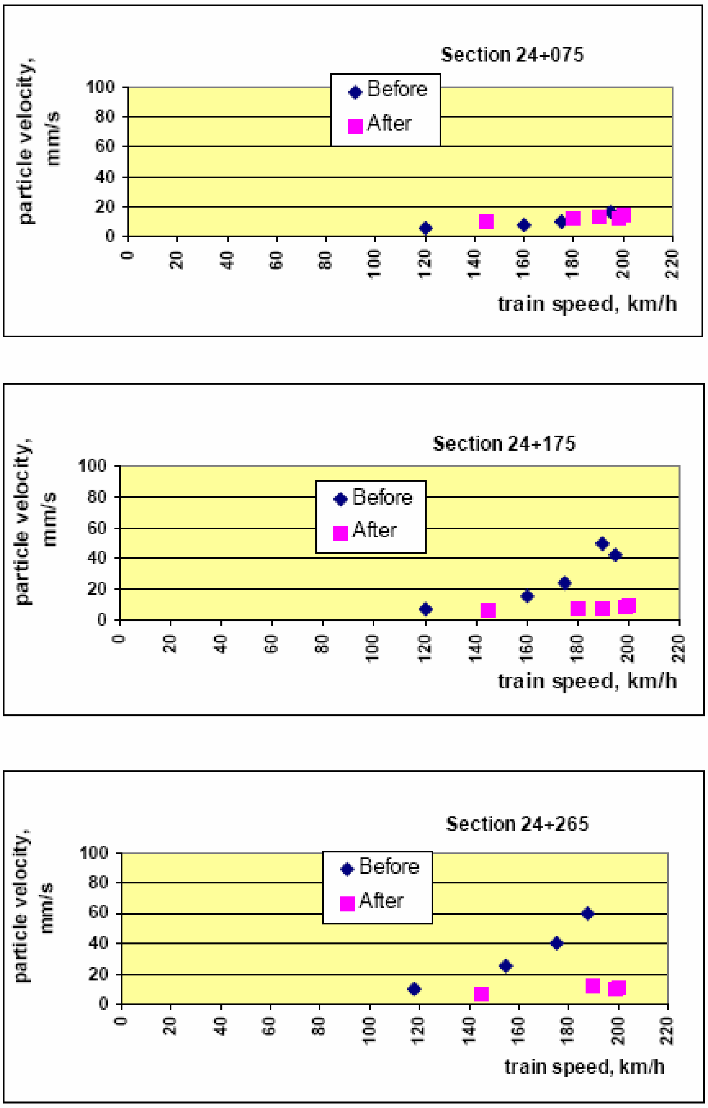


Figure 18 – Particle velocity vs. train speed in Sections 24+075, 24+175 and 24+265 before (May 2000) and after (December 2000) dry DMM improvement of the railway structure, J&W (2001) mentioned by (Holm et al., 2002)

Other test sites

This section describes the location, soil properties and the installation of the lime-cement columns at three test sites. These tests sites were selected in connection with the West Coast railway project, a project aiming at constructing a new double-track railway line along the Swedish west coast between Goteborg and Malmö, making it possible for trains to travel at a speed up to 250 km/h. All the three sites are located in Halland, a province south of Gothenbourg (Göteborg). Deep stabilization using lime/cement columns has been applied in some sections under the railway embankments (Baker, 2000).

At the Varberg test site, large scale compression tests were performed to determine the modulus of elasticity of the lime/cement columns. To prepare columns for the tests, a steel plate connected to a wire running throughout the length of the column was installed at the bottom of the column simultaneously with the mixing of the soil. At the Fjärås test site, large scale permeability tests at different levels were performed to measure the permeability of the columns in situ. At the Löftån test site, both in situ load tests and permeability tests were performed and large samples of the columns were taken and transported to the laboratory.

Varberg test site

The geotechnical properties are as follows:

The undrained shear strength determined by the field vane test is about 14 kPa at 2m depth and increase to about 30 kPa at 8m depth. The bulk density is between 1.6 t/m^3 and 1.68 t/m^3.

The natural water content varies between 62 and 72 % which is higher than the liquid limit.

The installed lime/cement columns about 5 to 6 m in length were constructed by using 26 kg/m (92 kg/m^3) and the proportion by weight was 50% cement to 50% lime. The diameter of the columns is about 0.6m.

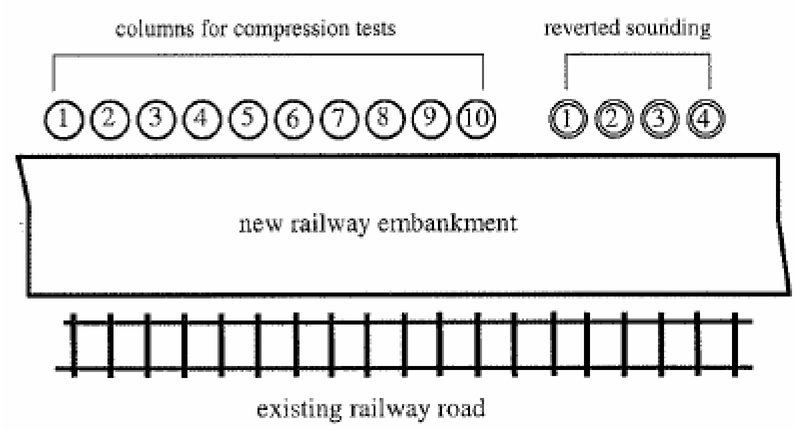


Figure 19 – Plane of installation of lime/cement columns at the Varberg site (Baker, 2000)

Ten columns were to be used for large scale in situ compression tests and four other columns were used to determine the shear strength of the columns (Figure 19).

The results showed very low resistance at the upper 0.5m of the lime/cement column and this was the reason for the excavation of the column top by about 0.5m. Load test results are given as axial vertical deformations versus depth below the top of the column for different load steps (Figure 20).

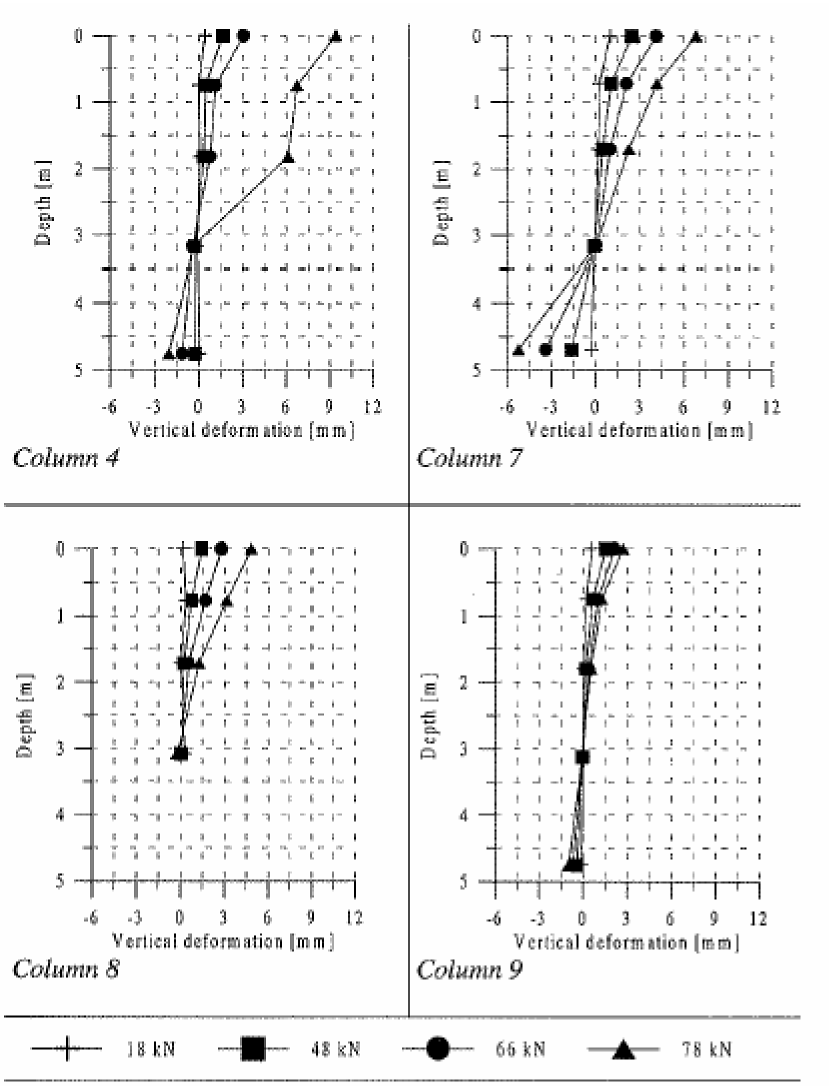


Figure 20 – Load test results at the Varberg test site (Baker, 2000)

The Fjärås test site

The geotechnical properties are as follows:

The undrained shear strength determined by the field vane test is about 8 kPa at 2m depth and increase to about 20 kPa at 10m depth. The bulk density is between 1.55 t/m^3 and 1.65 t/m^3.

The natural water content is about 70% at 2m depth and increases to 85% at 4m depth and then decreases to 55% at 9m depth. The liquid limit is less than the natural limit content.

The installed lime/cement columns about 15 m in length were constructed by using 30 kg/m (106 kg/m^3) and the proportion by weight was 50% cement to 50% lime. The diameter of the columns is about 0.6m.

Three columns were used for large scale in situ compression tests; two single columns and one column within a row. Samples were taken from other columns to carry out permeability test in the laboratory.

For column CTH1, four tests were conducted and the levels selected were 2.7 m, 5 m, 7.4 m and 9.7 m. The hydraulic conductivity values were roughly between 10-7 m/s to 10-8 m/s, with a minimum value at 7.4m depth (Figure 21).

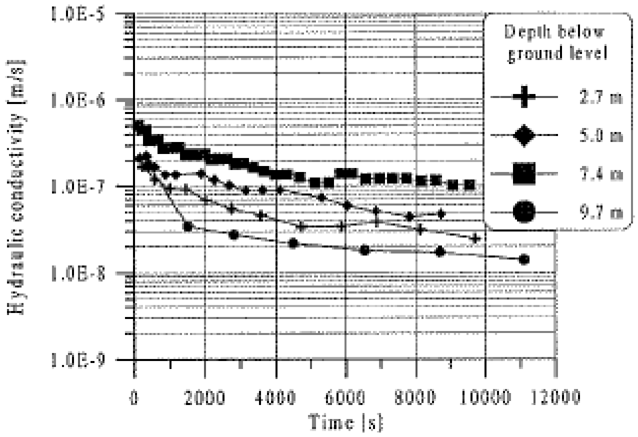


Figure 21 – In situ hydraulic conductivity test results for columns CTH1 (Baker, 2000)

The Löftån test site

The geotechnical properties are as follows:

The undrained shear strength determined by the field vane test is about 20 kPa at 2m depth and increase to about 20 kPa at 7m depth. The natural water content varies between 81 and 86 % which is higher than the liquid limit (74-79%). Thirty lime/cement columns were installed to a depth of 7m by using 38 kg/m (135 kg/m^3) and the proportion by weight was 50% cement to 50% lime. The diameter of the columns is about 0.6m. The columns layout is shown on Figure 22.

Twenty columns were intended for field permeability tests and laboratory tests. The remaining ten columns were used to perform field compression tests.

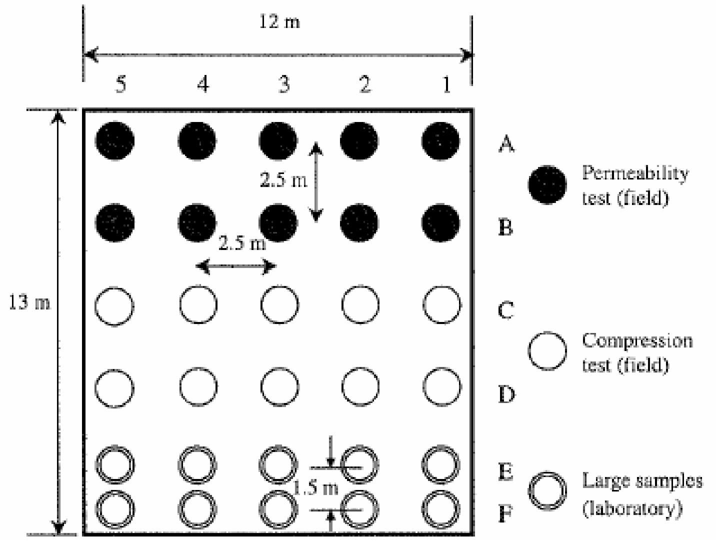


Figure 22 – Plans of columns installation at the Löftån test site (Baker, 2000)

The predominant behaviour of all the load test results from Löftån was that the relative deformation of the upper part of the column was larger than for the remaining parts of the column. In column C4 and D3 (Figure 23), large deformations at the bottom of the column were observed as well, which may be due to insufficient pull-up of the bottom plate directly after the installation of the column. The observations made during the load test provided clear evidence that the lime/cement columns do not consist of fully saturated material. In columns D3, air bubbles started to come from the columns through the drilled holes when the applied load exceeded 100 kN. At this load level, deformation became larger and part of the air included in the column started to flow through small cracks, possibly caused by the applied load.

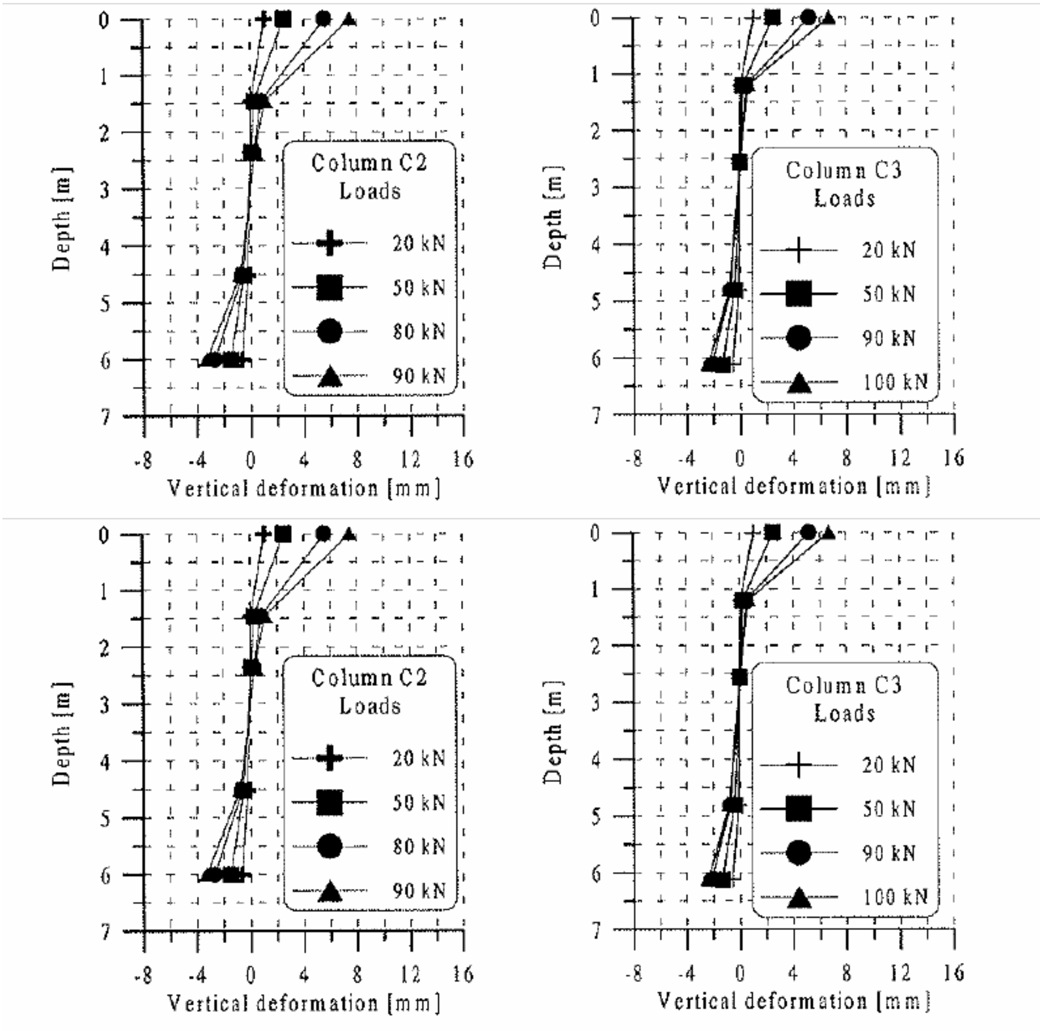


Figure 23 – Load test results at the Löftån test site (Baker, 2000)

The hydraulic conductivity of the clay was measured at two depths using constant-head piezometers. The measured hydraulic conductivity was about 8 10^{-10} m/s and 7 10^{-10} m/s at 2.1 m and 6.1 m, below the original ground level respectively (Figure 24).

For lime/cement columns, column A1 was excavated to a depth of only 0.4m below the original ground level. Three tests were conducted at 2.4m, 4.2m and 5.6m below the top of the column (Figure 25). At 2.4m, the measured hydraulic conductivity was about 3 10^{-8} m/s or about 40 times the hydraulic conductivity of the clay. At 4.2m depth the measured hydraulic conductivity was about 1 10^{-8} m/s. At 5.6m depth, the measured hydraulic conductivity was about 1.5 10^{-8} m/s.

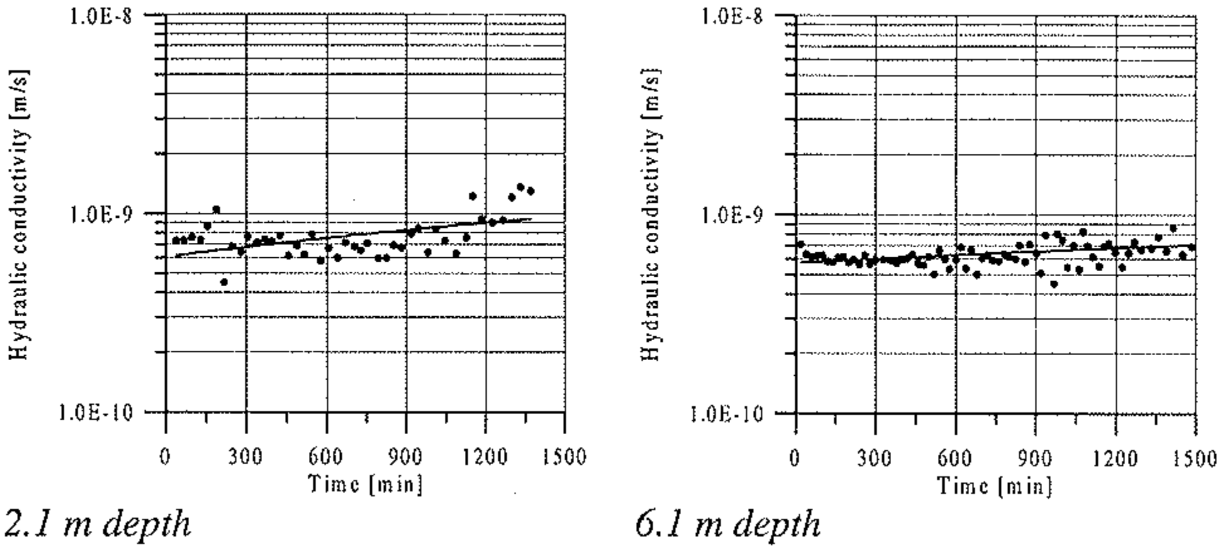


Figure 24 – The in situ hydraulic conductivity of the clay at two levels (Baker, 2000)

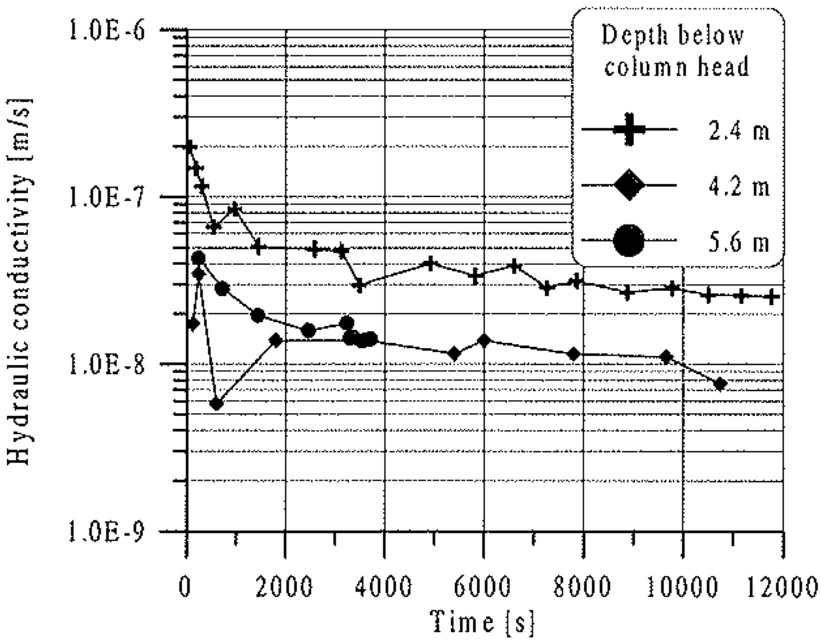


Figure 25 – In situ hydraulic conductivity test result for column A1 (Baker, 2000)

5.2.2 France

In France, Bachy Soletanche developed the COLMIX method in the mid-1980s, in conjunction with the French Railway Authority (SNCF) and the French Public Works Research Laboratory (LCPC). The COLMIX method appears to be the first development outside Scandinavia. The method features twin, triple or quadruple contra-rotating and interlocking augers, generally 3 to 4 m long and driven via hollow stem rods coupled to a single rotary drive. Blended soil moves from bottom to the top of the hole during penetration, and reverses on withdrawal ensuring very efficient soil mixing and recompaction. Several road and rail embankment stabilisation projects have been completed with this method in France, UK and Italy, as summarised by Lebon (2002, mentioned by (Moseley and Kirsch, 2004)).

The COLMIX technique of SOLETANCHE-BACHY has been tested by SNCF-LCPC for the INNOTRACK project as well as a similar technique provided by KELLER FOUNDATIONS. The results of these tests are given in (Rocher-Lacoste and Le Kouby, 2008) and some elements are presented in paragraph 5.3.

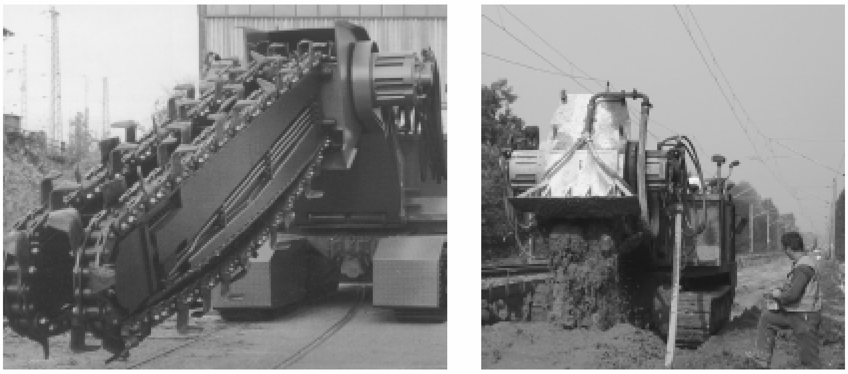
An alternative soil reinforcement technique is currently used by SOLETANCHE-BACHY; the Trenchmix technique (Figure 26).

Figure 26 – Trenchmix by SOLETANCHE BACHY

5.2.3 Germany

Cut-mix-injection

Another high-capacity specialised wet mixing system developed in Germany in 1994 is the FMI method (Fras-Misch-Injektionsverfahren cut-mix-injection). It was applied for the first time in 1996 in Giessen (Pampel and Polloczek, 1999, mentioned by (Moseley and Kirsch, 2004)). The FMI machine is made of a special cutting tree, along which cutting blades are rotated by two chain systems (Figure 27 a). The cutting tree can be inclined up to 80°, and is dragged thro ugh the soil behind the power unit. Due to special blade configuration, the soil is not excavated, but mixed with a binder which is supplied in slurry form through injection pipes and outlets mounted along the cutting tree. With this method it is possible to treat the soil in deep strips, with a mean capacity of 70–100m^3/h. The width of treatment is 1m down to a depth of 6m or 0.5 m down to a depth of 9 m. The applications mainly covered ground improvement works along railways (Figure 27 b).



(a) (b)

Figure 27 – The FMI (cut-mix-injection) machine, Germany (courtesy of Siedla & Schönberger)((Moseley and Kirsch, 2004)

Soil-cement columns

To analyse the dynamic soil-structure interaction of railway lines on soft soil experimental and numerical investigations have been carried out. The goal was to get information about the influence of different soil improvement layouts and further on to establish a design tool for railway lines on soft soil based on the additional results of a numerically supported parametric study.

To investigate the influence of the soil improvement layout under a railway line on soft soil experimental investigations were done in a 300 m long testing area in northern Germany (Katzenbach and Ittershagen, 2005).

The layout of the test tracks TS0–TS4 and the column arrangement installed in the five different test tracks TS0-TS4 is given in Figure 28. The ground improvement was constructed by Lime-Cement Columns with a column diameter of 0.6 m. Based on the Design Guide for Soft Soil Stabilisation (BRE, 2002 mentioned by (Katzenbach and Ittershagen, 2005)) a dry mixture of 90 % cement and 10 % lime was used, the amount of binder mixed in was about 110 kg/m³.

The ground below the track system in the testing area consists of a soft soil layer with a depth of about 10 m overlaying a good bearing middle dense to dense sand stratum.

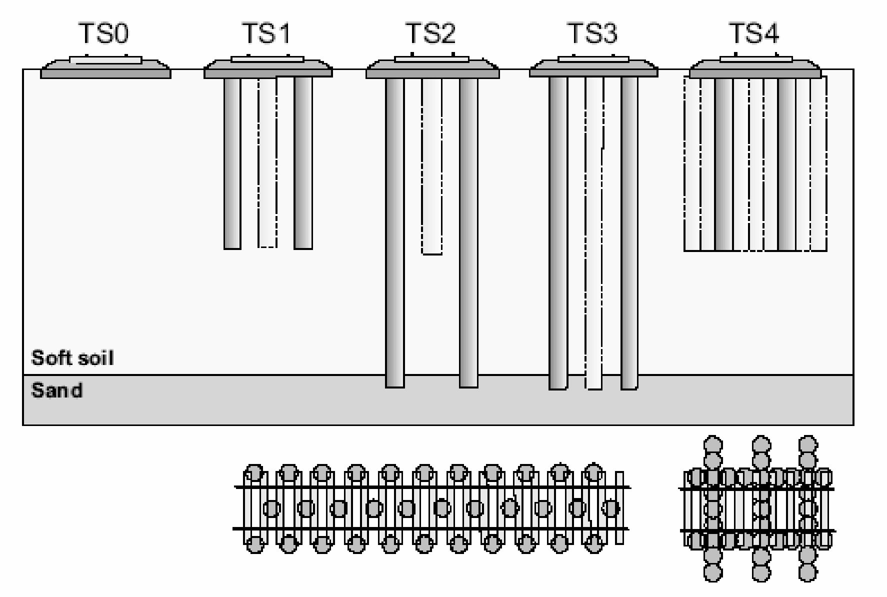


Figure 28 – Column arrangement installed in the test field (Katzenbach and Ittershagen, 2005)

Table XIV – Mean mechanical parameters of the subsoil (Katzenbach and Ittershagen, 2005)

Parameter	Symbol	Unit	Soft soil	Sand
Elastic module	E	MN/m^2	≤ 3	65
Friction angle	φ' / φ_u	°	25 / 0	30 / 0
Cohesion	c' / c_u	kN/m^2	5 / 40	0 / 0
Density	γ	kN/m^3	14–17	18–19

Each test track was equipped with multiple measurement devices pictured in Figure 29 to observe the dynamic response of the subsoil and the railway structure during operation. During the tests the area was subjected to train traffic for three months with different train speeds of V1 = 30 km/h, V2 = 50 km/h, V3 = 70 km/h and V4 = 90 km/h (V4 only for the passenger trains). Altogether 1.200 train passages were observed and recorded.

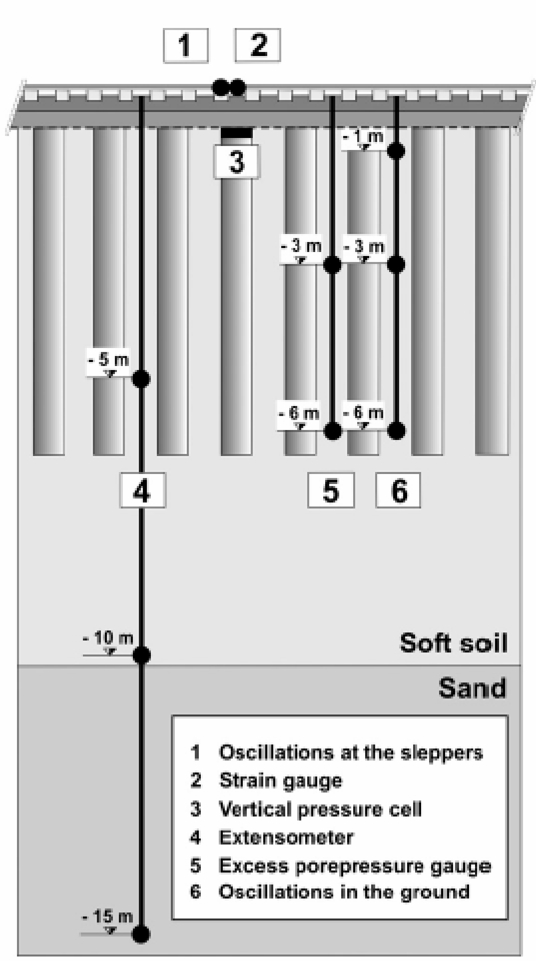


Figure 29 – Layout of measurement devices (example TS1) (Katzenbach and Ittershagen, 2005)

In Figure 30, the excess porewater pressure in a depth of 3 m for a freight train with a speed of about 30 km/h is represented for test track TS0, TS1 and TS4. The measurements reveal that only a small amount of the dynamic loading is transferred in the porewater. For the test track configuration TS0 without any soil improvement the additional porewater pressure is $p \leq 30$ kN/m².

To get information about the oscillations in the ground during the train passage triaxial acceleration measurements were performed in a depth of 1 m, 3 m and 6 m below the surface. The vertical accelerations are integrated into velocities and converted into the Root Mean square Values, abbreviated RMS. The results for the freight trains for different train speeds are presented in test track TS0 (Figure 31), TS1 (Figure 32) and TS4 (Figure 33) are presented. To compare the results best fitting curves with an exponential function are added.

In Figure 31, the RMS magnitude decreases rapidly with increasing depth, the greatest values are measured close to the surface. Beyond this it can be recognized especially for the measurement point at a depth of 1 m that the RMS in test track TS0 increases more than linear with every train speed increment. Compared to test track TS1 and TS4 the RMS-Values close to the surface are obviously smaller due to the higher dynamic stiffness of the soil improvement under the railway track and thus connected with a smaller rate of long term deformation.

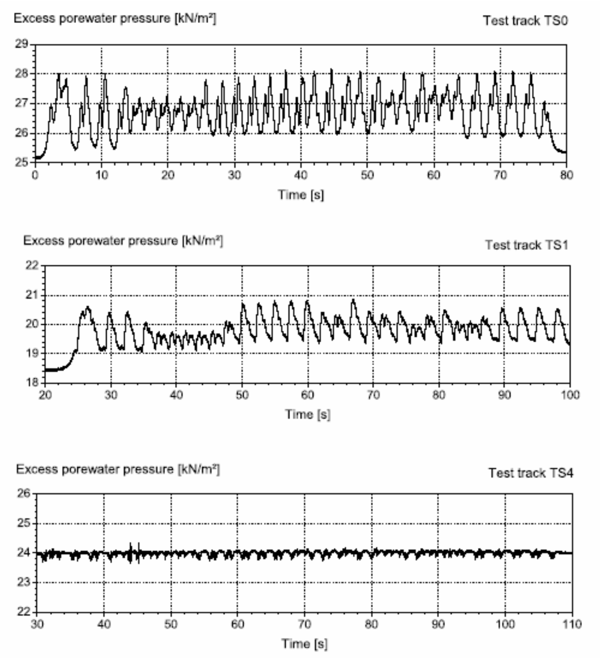


Figure 30 – Excess porewater pressure measurements in TS0, TS1 and TS4 (Katzenbach and Ittershagen, 2005)

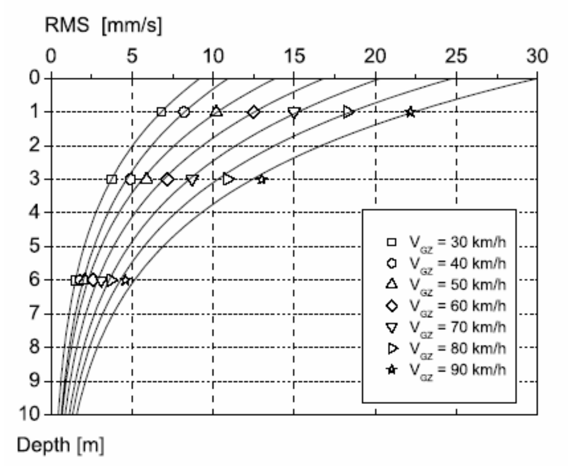


Figure 31 – RMS of vertical accelerations vs. depth for test track TS0 (Katzenbach and Ittershagen, 2005)

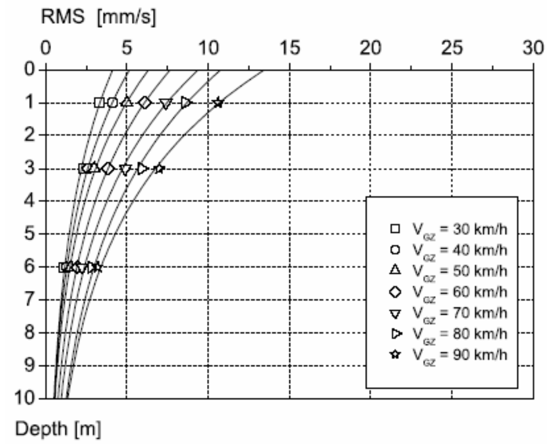


Figure 32 – RMS vs. Depth for test track TS1 (Katzenbach and Ittershagen, 2005)

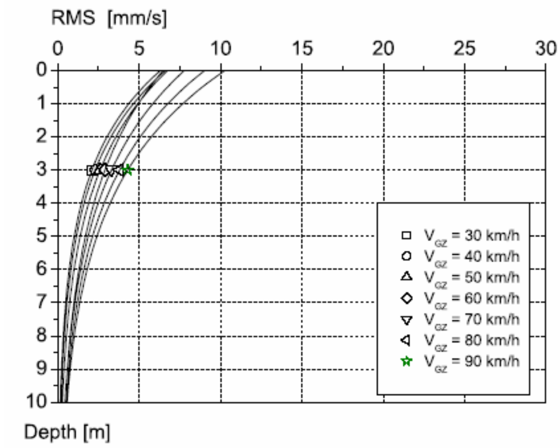


Figure 33 – RMS vs. Depth for test track TS2 (Katzenbach and Ittershagen, 2005)

Hamburg–Berlin railway line: reinforced embankment on pile like elements

General

As part of upgrading the railway line Hamburg-Berlin by the German Rail company, the Buechen-Hamburg and the Paulinenaue-Friesack part of the railway line were upgraded in 2003 to allow a train speed of 230 km/h. Because of the very soft organic soil (peat and mud) layer with insufficient bearing capacity, stabilization of the embankment foundation was necessary at these two parts.

Buechen-Hamburg line

The part of this line with a total length of 625 m was near the railway station Buechen. The soft underground was stabilised with columns installed using Mixed-in-Place Method (MIP) and the embankment was reinforced with geogrids at its base on the top of the columns (Figure 34). The MIP belongs to the wet deep mixing methods.

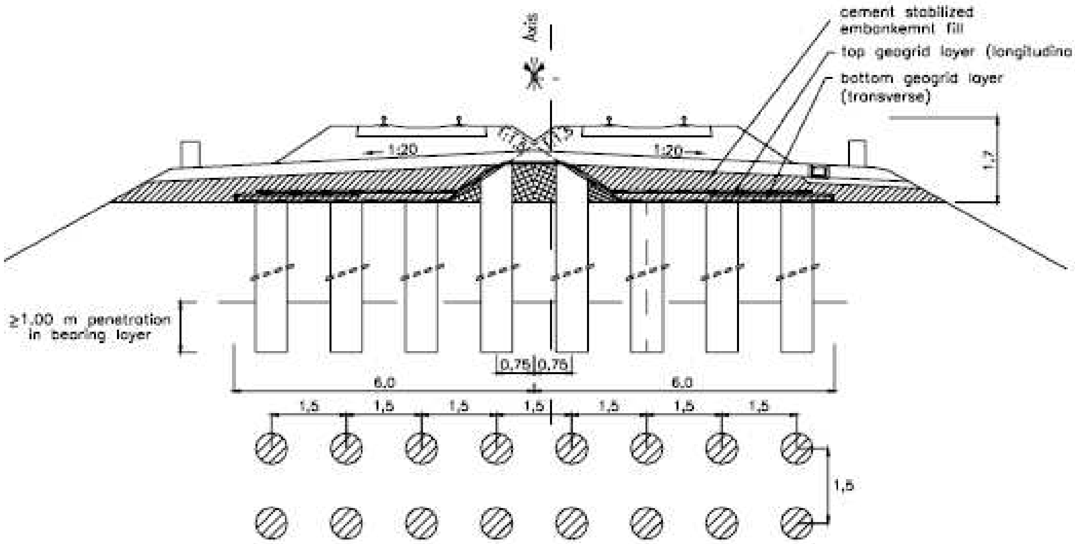


Figure 34 – Foundation system at a typical section in the Buechen-Hamburg line (Topolnicki, 2003)

Underground condition

The underground consists of a 3 to 5 m fill of silty and gravely medium dense sand with slag and organic mixtures underlain by 0.5 to 2 m thick layer of a very soft peat and mud. The peat soil has a void ratio of 80 to 330% and an organic content between 25 and 80%. Beneath the soft layer a medium dense and slightly silty sand layer with a thickness up to 8 m is encountered followed by a boulder clay with soft to stiff consistency and a water content of 10 to 20%.

Construction

Throughout the upgrading work, a single track operation at a speed of 90 km/h was maintained. The stability of the track during operation was secured by extending the slope of the ballast layer to the base of the embankment (Figure 34). This had made the construction of the geogrid reinforcement across the total embankment width possible. The MIP-columns were installed after excavation of the subbase layer (Figure 35 b). Prior to the setting of the MIP material, the columns generally were cut to a level of 1.7 m below top of rail line during the next excavation stage (Figure 35 b). The columns adjacent to the embankment symmetrical axis were however left uncut and resulted in a cover less than 1.5 m on top of the columns. Nevertheless, this construction option was favoured over a temporary sheet pile wall, since the retracting of sheet piles could lead to unexpected settlements.

The MIP-columns were installed using a single axis auger (Figure 35 a). A cement slurry was injected continuously into the soil during the penetration as well as during the retrieval of the auger. Due to the rotation of the auger, the cement slurry is mixed with the soil. The MIP-technique is free of vibrations and displacements and therefore had no effect on the ongoing railway traffic on the other track. The cement columns (diameter 0.63 m) were installed in a square grid of 1.5 x 1.5 m.


a)


b)

Figure 35 – a) Installation of MIP-columns; b) cutting of the MIP-columns (Topolnicki, 2003)

The proportion of the slurry mix (water, cement and bentonite) and the water/ binder ratio (approx. 1.0) was determined in laboratory on trial mixed samples.

During the first soil stabilisation stage (track Hamburg-Berlin), about 800 l/m³ binder were mixed into the soil. During the second stage (track Berlin-Hamburg), the binder was optimised and mixed with the soil to the extent up to a homogenous soil/binder mixture was obtained. This resulted in a variable, soil dependant binder quantity. The depth of the columns was determined on the basis of cone penetration tests prior to column installation. All in all 3260 MIP-columns with a length between 5 and 8 m were installed (total length ~ 21000 m).

On top of the MIP-columns two layers of Fortrac® PVA geogrid type M 400/30-30 had been placed. Since the geogrids are subjected to a load in longitudinal direction only, the short-term tensile strength in transverse direction was taken to be 30 kN/m, whereas the required short-term tensile strength in longitudinal direction was set at 400 kN/m.

The first geogrid layer was placed in transverse direction directly on top of the MIP-columns. This geogrid was rolled up near the embankment axis during the 1st construction stage, and later laid across the whole embankment in the second stage. The second geogrid layer was placed in longitudinal direction.

To obtain a uniform bearing platform for the ballast bed, 2.5 to 3% cement was added to the filling material. The top of this cement stabilisation was made rough to ensure a sufficient friction with the upper protective layer. To avoid an influence of hydrolysis of the cement, Polyvinylalcohol was used as geogrid material.

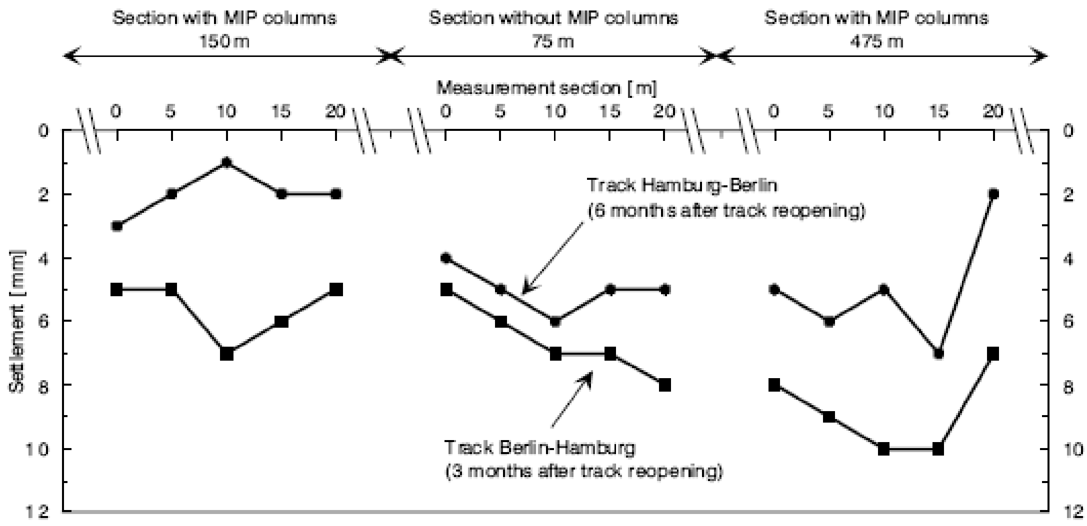
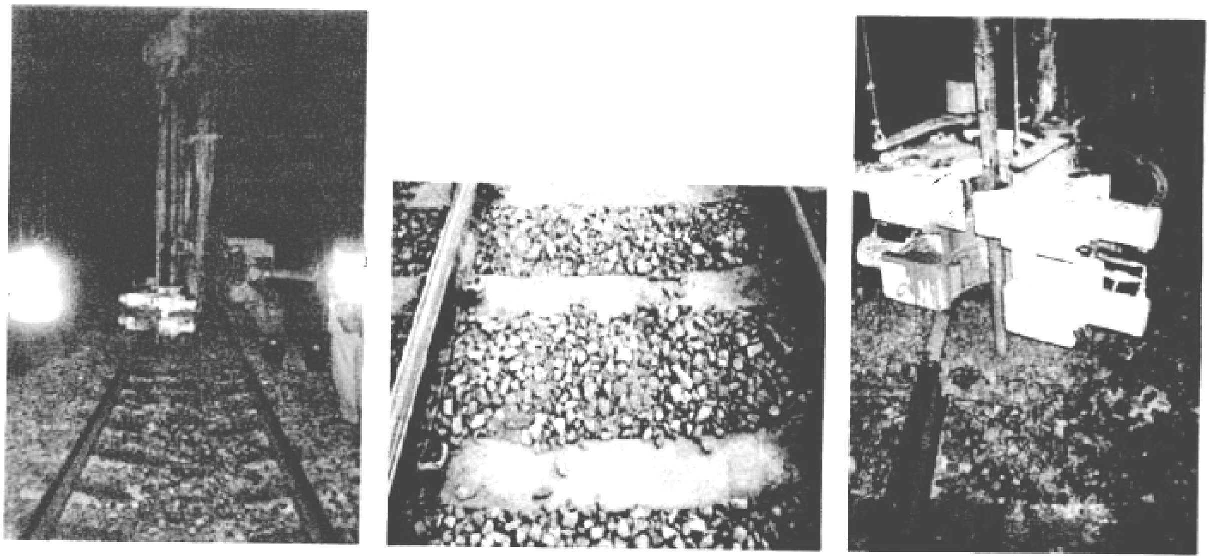


Figure 36 – Measured settlement (Topolnicki, 2003)

Monitoring. The settlement behaviour of the tracks was monitored by means of geodetic measurements of the position of the outer rail of both tracks. The measurements were carried out at three measurement sections each 20 m in length and consists five measuring points with a spacing of 5 m. These measurement sections were selected at locations with unfavourable soil conditions. The results of the settlement measurements over a period of 6 months train operation are presented in Figure 36. On both tracks the train was operated up to a speed of 160 km/h. Figure 36 shows a maximum settlement of 7 mm in a period of 6 months after reopening of the track Hamburg-Berlin. This settlement can be considered as small since a settlement of 10 mm to 15 mm can usually occur due to compaction of the ballast bed, the subbase layer and the embankment, even if the soil conditions could be favourable. Furthermore, the geogrids require a small amount of deflection to function effectively.

5.2.4 UK

Within the ISERT project (Improvement of Stiffness of Existing Rail Track), an experimental work was undertaken in a soft clay site. The issue was the dynamic forces due to railway traffic (Konstantelias et al., 2002). The subgrade layer needed to be reinforced. The research was focused on implementation of a ground improvement technique that can be applied in the cess (area either side of the track, usually up to the fence), through the ballast shoulder or between sleepers, without removal of track components. Any technique, therefore, is subjected to strong working restrictions, that will dictate plant and rig dimensions. The improvement technique should not affect ballast functions relating to load transfer, drainage and maintenance facilitation. In order to determine an optimum grout mix for the reinforcement/modification of the formation layer beneath the track bed, numbers of laboratory-based experiments were carried out and a mixture based on cement was chosen. The density chosen was 122 kg/m^3, the water/cement ratio was 1 and 15 litres per meter depth grout was used. The vertical withdrawal rate was about 200 mm/min at a speed of 250 revolutions per minute.



(a) Overnight work on the track (b) Close view of contaminated ballast 24 hours after completion of the trial (c) Ballast contamination during column construction

Figure 37 – Issues encountered during the execution phase (Konstantelias et al., 2002)

Some ballast contamination occurred during the process of the works. In particular, during major system blockages water/cement slurry was washed onto the ballast which in turn solidified. The formation of these columns is shown in Figure 37 (a) and (c), and a typical crib after completion of the works is shown in Figure 37 (b). The final layout of the completed works is given below and it is shown in Figure 38.

Four metres sections of the track were treated. The construction of the grout columns had caused surprisingly little disturbance, as the final levels were ±2.5mm and ±0.5mm in both the vertical and horizontal sections. Figure 39 shows the improvement in stiffness of the subgrade after the treatment.

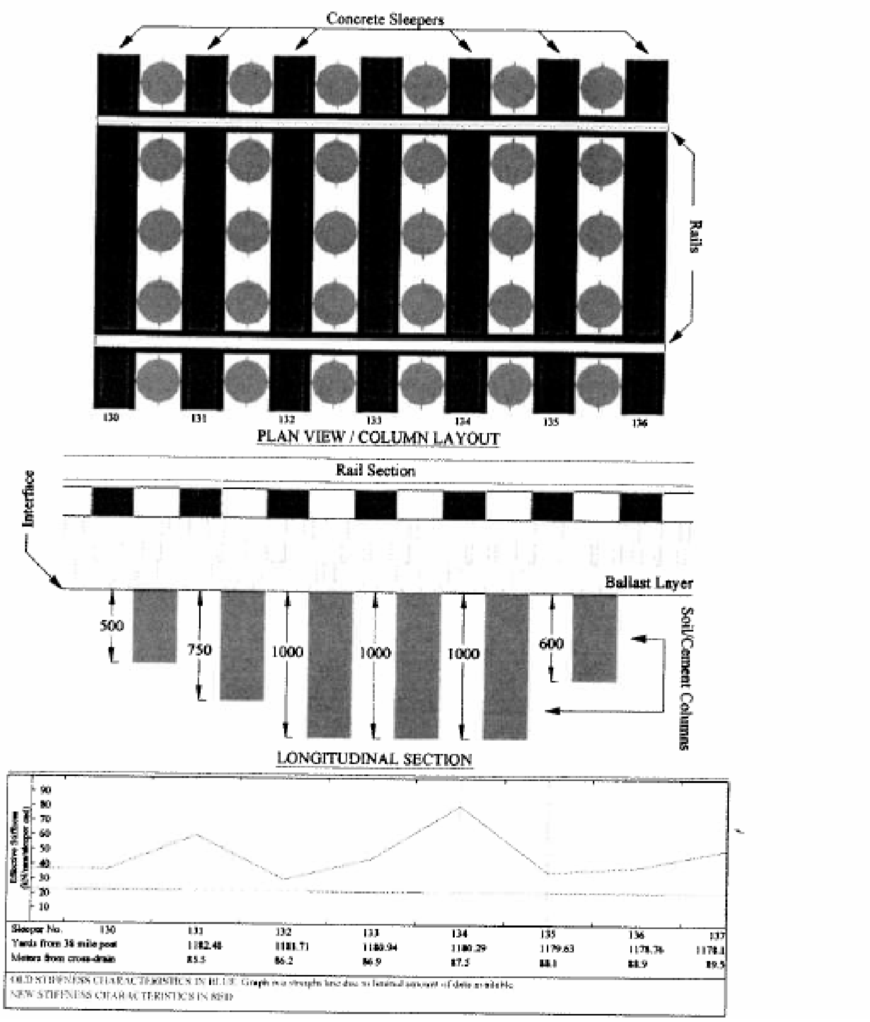


Figure 38 – Final layout of the soil/grout columns (1 bed of five 0.5-meter deep columns, 1 bed of five 0.75-meter deep columns, 3 beds of five 1.0-meter deep columns and 1 bed of five 0.6-meter deep columns)

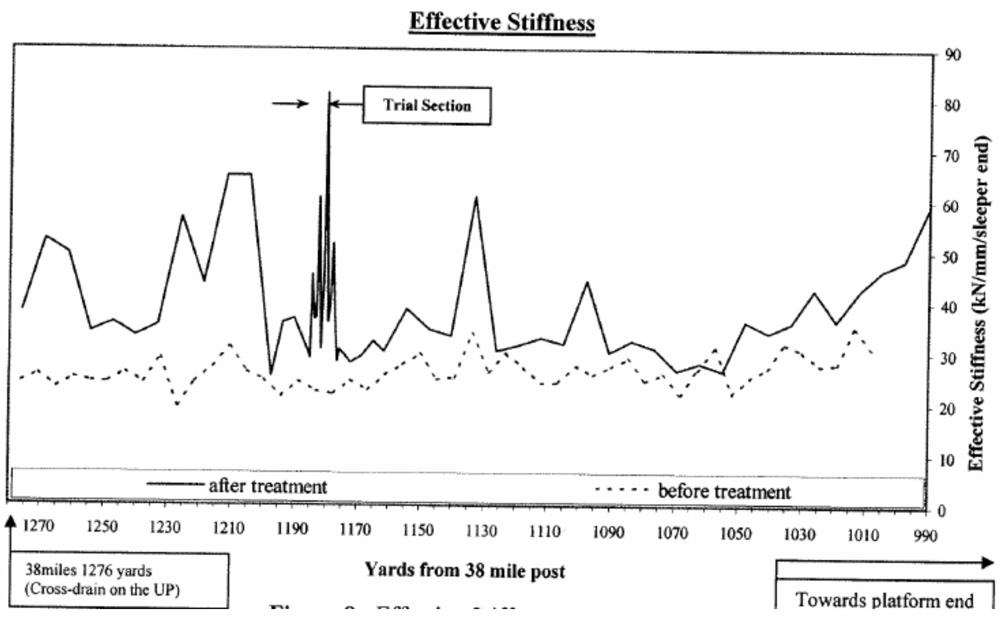


Figure 39 – Effective stiffness (FWD results) (Konstantelias et al., 2002)

5.2.5 Malaysia (dry DM)

In this paragraph, a case history is presented in a synthetic manner to illustrate current applications of the DMM, focusing mainly on embankment and foundation support. This is a recently completed project conducted in Malaysia.

Source	Raju, Abdullah and Arulrajah (2003) mentioned by (Moseley and Kirsch, 2004)
Location	Railway line between Rawang and Ipoh, Malaysia
Construction site	800m long, 20–25m wide (Figure 40)
Soils	Very soft silty clay or clayey silt to loose silty clayey sand, typical CPT log see Figure 41, moisture content w¼ 50–70 per cent, groundwater c. 1m below ground surface
Embankment height and load	1.5–3 m, equivalent traffic load 30 kPa
Design requirements	Train speed 160 km/h, max. settlement 25mm in 6 month of operation, max. differential settlement 0.1 per cent along the centreline, safety factor for slope failure 1.5 (long-term)
Applied DM method	dry method single shaft (Figure 41 b) Column data Diam. 0.6 m, length 7–14 m, overall 50 000 linear m
Column pattern	Detached columns, square/rectangle, 1–1.3m c/c under the rails (ap¼28–17 per cent), 1.4–1.5m c/c remaining area (ap ¼14–13 per cent)
Design shear strength	250 kPa under the rails, 150 kPa remaining area Binder type and factor Portland cement 100 per cent, 100–150 kg/m3
Embankment reinforcement	Geotextile 100/50 kN/m (longitudinal/transverse direction)
Observed performance	Settlement below 10mm for embankment 1–1.5m, lateral displacement below 15 mm, Loading test (see Figure 42)

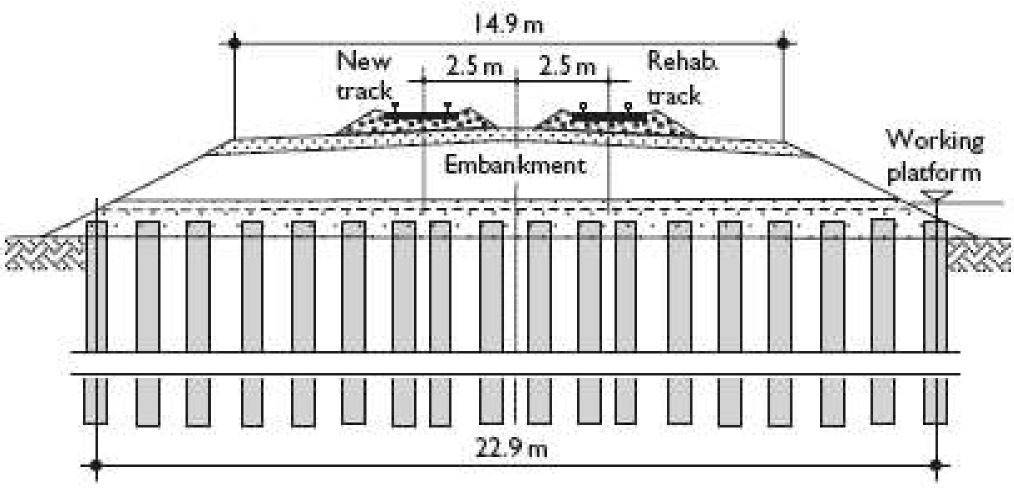


Figure 40 – Typical cross-section of the railway embankment and treated zone (Raju et al., 2003, mentioned by (Moseley and Kirsch, 2004).

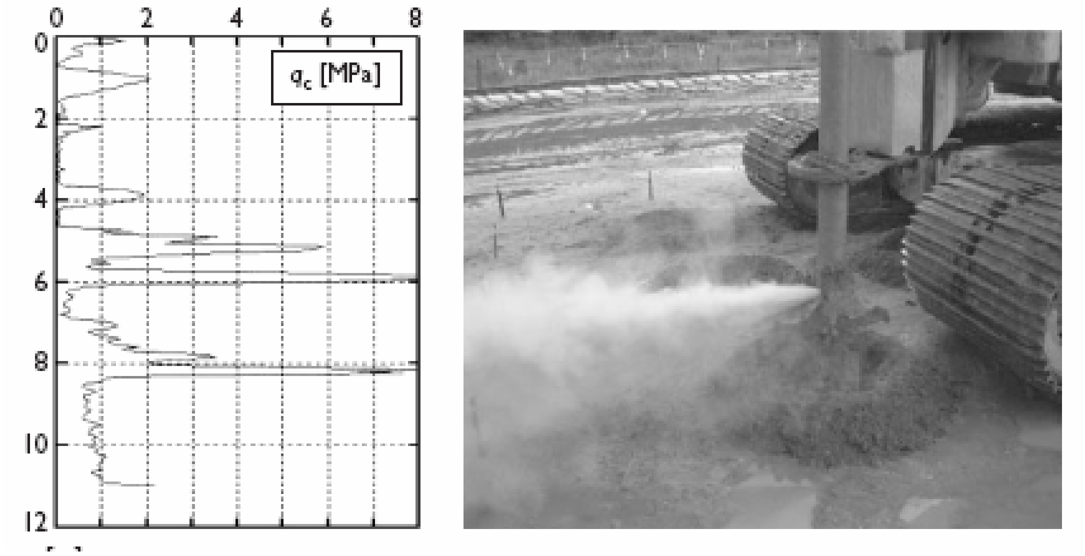


Figure 41 – (a) Typical CPT log (b) The LCM machine at work (Raju et al. (2003) mentioned by (Moseley and Kirsch, 2004))

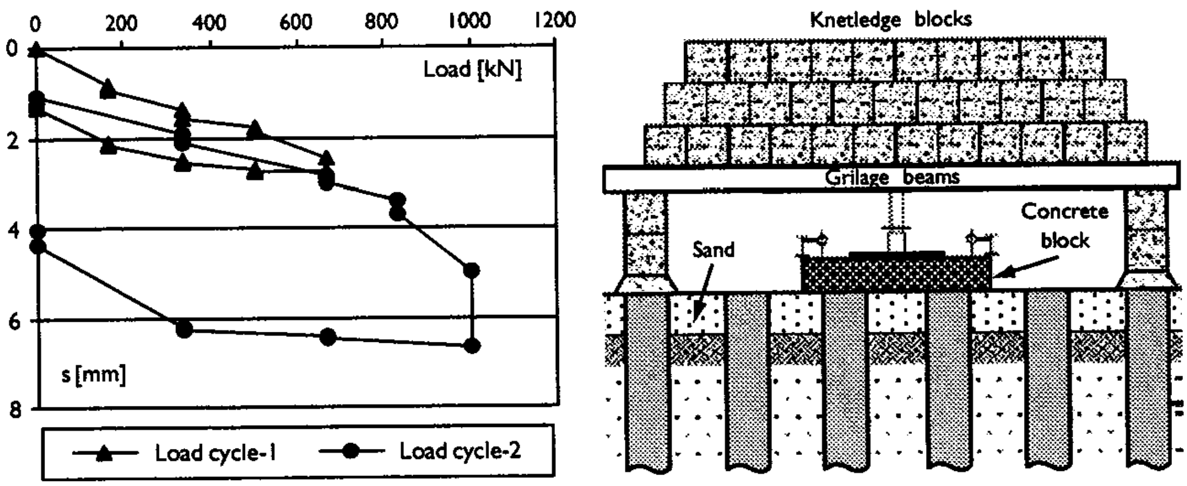


Figure 42 – Control static loading test over an area of 3x3m, 4 columns : vertical displacement vs. applied load (after Raju et al.(2003) mentioned by (Moseley and Kirsch, 2004))

5.3 Prescriptions on the dry-wet deep mixing method

The stabilisation of soft clay, organic soil, loose sand and silt by the addition of dry binders is a well-known technique in the Nordic countries. The dry deep mixing method has been used frequently during the last three decades for soil improvement of road and railway structures. It is also used as a foundation for buildings such as shopping malls, oil tanks and industrial buildings. The volume of soil stabilised in Sweden and Finland during recent years has been approximately 600,000 m^3/year, corresponding to approximately 2 million metres of columns.

Mixing dry binder with soft soil results in an increase in strength and stiffness. These improvements are immediate but are also followed by a long-term increase. A mixing tool, mounted on a leader, is rotated down into the ground to the required depth. During the slow withdrawal of the mixing tool, a dry mixture of binders is injected into the soil, thus creating an improved soil column. The columns interact with the non-stabilised surrounding soil, resulting in an improved composite soil volume.

5.3.1 Purpose of soil improvement

Deep stabilisation is a method to stabilise soft soils by adding dry or wet binders in order to reduce settlements and/or to improve the stability. The soil can be stabilised either by forming columns of stabilised soil (so-called column stabilisation) or by stabilising the entire soil volume (so-called mass stabilisation). However, the two methods may well be combined.

With existing equipment the soil can be stabilised to a depth of about 25 meters when using column stabilisation whereas mass stabilisation can be used to a depth of about 5 meters.

The main purposes of deep soil stabilisation are:

a) To increase the strength of the soft soil in order to:

- increase the stability of an embankment
- increase the bearing capacity
- reduce the active loads on retaining walls
- prevent liquefaction

b) To improve the deformation properties of the soft soil in order to:

- reduce the settlements
- reduce the time for settlements
- reduce the horizontal displacements

c) To increase dynamic stiffness of the soft soil in order to:

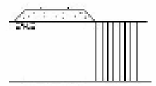
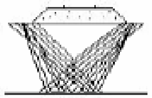
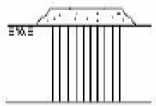
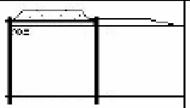
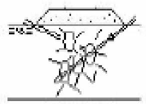
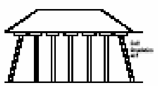
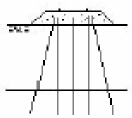
- reduce the vibrations to the surroundings
- improve the dynamic performance

d) To remediate contaminated ground (soil) by:

- creating an environmental barrier (solidification)
- stabilisation of the contaminated ground
- creating a geohydrological barrier

At the Ledsgård site, for example, deep mixing was performed to increase the stiffness of the soft soil layers. By installing the columns in an appropriate configuration and assuring a good connection between the columns and the railway structure, an effective soil improvement system was to be achieved.

Table XV – Overview of existing strengthening methods in transition zones (Holm and Smith, 2006)

	Method	Principle	Can be performed without affecting traffic	Applicable soils	Increases Stability	Reduces Settlements	Approximate Costs	Experience with existing railways
	Deep Mixing, beside railway embankment	Mixes in-situ soils with cementitious materials to form a vertical stiff inclusion in the ground	Yes	Wet method: most soft soil types; Dry method: soft fine-grained soils	X		10 Euro/m column at an amount of binder of 90-120 kg/m^3	Some
	Deep Mixing, installed inclined under embankment	Mixes in-situ soils with cementitious materials to form an inclined stiff inclusion in the ground	Yes	Wet method: most soft soil types; Dry method: soft fine-grained soils	X	X	15 Euro/m column at an amount of binder of 90-120 kg/m^3	None (some for road embankment)
	Deep Mixing, installed through embankment	Mixes in-situ soils with cementitious materials to form a vertical stiff inclusion in the ground	No (Yes, if performed during periods with no traffic)	Wet method: most soft soil types; Dry method: soft fine-grained soils	X	X	40-100 Euro/m column Depending on access	Some
(Same three configurations as for Deep Mixing)	Jet grouting	Erodes soil in situ and mixes with cementitious materials to form stiff inclusion in the ground	Yes (unless installed beneath embankment)	Most soil types	X	X	Mob/Demob 30-50,000 Euro 250-350 Euro/m column	Some
	Stabilizing berms, alone or in combination with anchored sheet pile walls	Compacted material constructed adjacent to embankment. Driven steel sections provide resistance against horizontal movements.	Yes	Clay	X		150-200 Euro /m^2 sheet pile excl. anchoring	Extensive
	Compaction grouting	Low slump grout is pumped into the ground to form grout bulbs, which displace and densify the soil	Yes	Granular soils	X	X		Some
	Pile deck or piles with pile caps and possibly, geosynthetic reinforcement	Piles transfer loads to more competent strata through friction or end-bearing; piles caps and geosynthetic transfer load to piles	No	All soil types	X	X	For driven concrete piles: 45-55 Euro/m + Pile caps	Extensive
	Embankment piles, without pile caps	Driven or grouted piles transfer loads to more competent strata through friction or end-bearing	No (Yes, if performed during periods with no traffic)	All soil types	X	X	For driven concrete piles: 45-55 Euro/m + Pile caps	Some

Three ways that deep mixed columns can be installed in railway transition zones include:

1. Columns installed vertically through the railway embankment,
2. Columns installed vertically beside the railway embankment, or
3. Columns installed inclined under the railway embankment.

In this section, we only deal with 1. and 2. Part 2 of the deliverable deals with 3.

Deep mixed columns are most effective when they are subjected to axial loads, such as when they are installed vertically directly under the embankment (Figure 43).

There is extensive experience with the use of deep mixed columns to support new railway embankments ; however there is less experience with the installation of deep mixed columns through existing railway embankments ((Holm and Smith, 2006)).

There are some limitations with installing the columns beneath the railway embankment. First, columns must be installed with equipment resting on the tracks, and as a result, the installation will disturb railway traffic unless it can be performed during limited time periods without traffic. Second, some materials such as compacted, well-graded crushed rock as well as fill with blasted stone, in the railway embankment, may not be possible to penetrate. Third, it is possible that the ballast material is contaminated with the subsoils during installation. A certain remoulding of the ballast material and the underlying fine-grained material is difficult to eliminate completely. To what extent soft clay is pushed up around the drill rod depends on how sensitive the clay is, as well as on the thickness and properties of the embankment and on the mixing process used

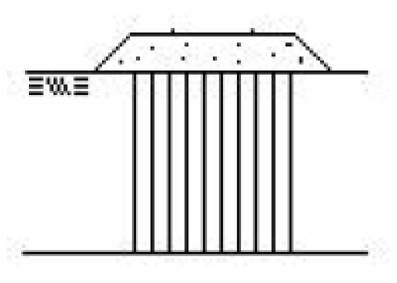


Figure 43 – Deep mixed columns installed beneath a railway embankment

The columns are installed either in blocks or in a grid pattern. Columns that are installed vertically beside the embankment will be subjected to axial and horizontal loads (Figure 44). It is typically recommended that columns be installed overlapping in slabs or wall patterns perpendicular to the railway alignment as questions remain regarding the behaviour of the columns when subjected to horizontal loads. Installation is carried out from a temporary road constructed along the railway embankment using conventional installation machines. There will remain a zone of unstabilised soil between the embankment and the deep mixed columns. The effect of this unstabilised zone will vary from case to case. In some cases, the embankment itself has settled to such an extent that this zone mainly consists of embankment material.

The advantages and disadvantages of installing deep mixed columns (1) vertically beneath the railway embankment and (2) beside the railway embankment were discussed in the preceding paragraphs. Advantages of the deep mixing method in general include that it can be performed with no or only minimal disruption to train traffic. Furthermore, there is a wide range of experienced contractors and equipment available in Europe to perform deep mixing. The dry method of deep mixing has been used extensively in Scandinavia to support new railway embankments, and for the widening of existing embankment. Both the wet and dry methods of deep mixing require a high level of field control. The high cost of mobilization/demobilization, plus the cost of the accompanying auxiliary batch plants, makes the wet method of deep mixing uneconomical for small projects. Furthermore, the wet method of deep mixing requires considerable headroom for the equipment and the disposal of large quantities of spoil volumes.

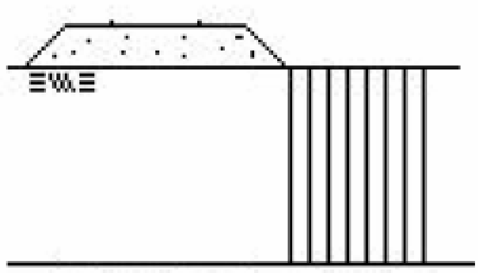


Figure 44 – Deep mixed columns installed vertically beside the railway embankment

In view of the widespread usage of dry deep mixing for soil improvement and the reduction of permanent deformations in soft soils in Sweden, it is a natural development to use the technology in vibration reduction purposes where possible. Dry deep mixing is often used to improve the soil beneath railway structures and thereby increase the stiffness of the soil-railway structure system. The principle is shown in Figure 45. Another important effect is that the improved soil increases the critical speed for the system so that extreme vibration levels due to train passages at this speed will be avoided. The dry deep mixing method (DMM) has been described in Bredenberg, Holm, Broms (1999, mentioned by (Holm et al., 2002)).

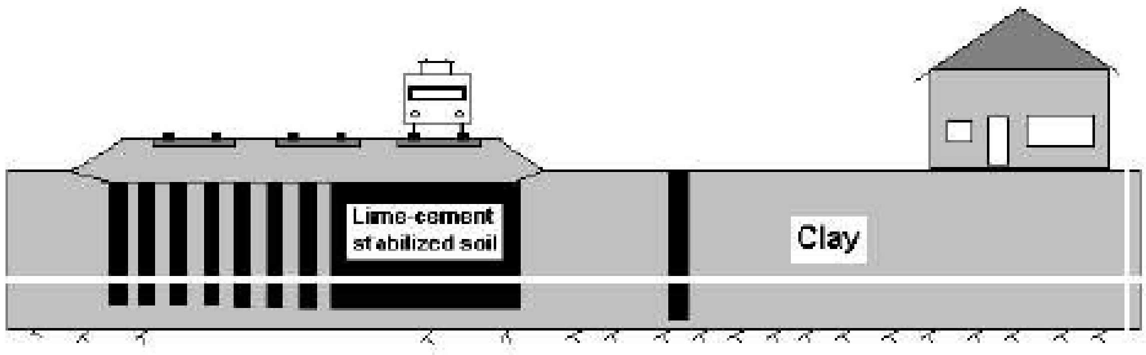


Figure 45 – Principle of improvement of the soil beneath the track with the dry deep mixing method (Holm et al., 2002)

There is a European standard for the execution of deep mixing, “Execution of special geotechnical works – Deep mixing”, EN 14679:2005. This is a standardisation of the execution procedures for geotechnical works (including testing and control methods) and the required material properties. Guidance on practical aspects of deep mixing, such as execution procedures and equipment is given in an annex to the standard. Methods of testing, specification and assessment of design parameters are also presented in the standard.

5.3.2 Geotechnical investigation

Before treatment

Field and laboratory investigations shall provide information regarding:

- sequence of soil layers and their properties;
- groundwater conditions;
- the presence of organic soil, sulphides in the soil and pH;
- the composition, thickness, firmness of the surface stratum and any tree roots, fill, etc;
- the presence of fixed obstacles to column placing (e.g. buried pipes, cables and overhead lines);
- the properties of soil after the binder have been mixed in. Mixing trials are performed for characteristic soil strata.

The in situ and laboratory tests should follow the European standards (EN 196-1 to 8, EN 196-21, EN 197-1 and 2, EN 459-1 and 2, ENV 10080, EN12716, EN 791, EN ISO 14688-1 and EN ISO 14688-1, mentioned by NF EN 14679 (2005) (European-Standard, 2005)).

Classical in situ tests are used such as core sampling, CPT, SPT, RPT, PMT (Porbaha, 2002). Geophysical methods such as seismic tomography and electrical resistivity are also used (Porbaha, 2002).

After treatment

The treated soil is investigated following the same standards as described above (Porbaha, 2002).

The dispersion of in-situ strength is investigated.

This investigation can be associated to methods of quality assurance:

- Laboratory tests on core samples, wet grab samples and block samples from exposed or extracted columns
- In situ tests
  - Penetration tests : Static Cone Penetration Test (CPT), Standard Penetration Test (SPT), Rotary Penetration Test (RPT), Pressumeter Test (PMT), Conventional Column Penetrometer test (CCP), Reverse Column Penetrometer (RCP), Modified Column Penetrometer (MCP), Column Vane Penetrometer test (CVP), Dynamic Cone Penetrometer (DCP), Static Dynamic Penetration tests
  - Geophysical methods: Seismic tomography (Downhole, Crosshole, Inhole) and electrical resistivity.

5.3.3 Some elements on design

The design is carried out for the most unfavourable combination of load effect and bearing capacity, which is likely to occur during construction and in service.

Design encompasses two distinct aspects:

- functional behaviour describes the way in which the treated soil and the untreated soil interact to produce the required overall behaviour,
- process design describes the means by which the required performance characteristics are obtained from the treated soil by selecting and modifying the process control parameters.

The design philosophy for deep stabilisation is to produce a stabilised soil that mechanically interacts with the surrounding unstabilised soil. The applied load is partly carried by the columns and partly by the unstabilised soil between the columns. Therefore, a too stiffly stabilised material is not necessarily the best solution since such a material will behave like a pile. Instead, the increased stiffness and strength of the stabilised soil should not prevent an effective interaction and load distribution between the stabilised and natural soil.

Therefore, design models are based on the assumption of interaction between columns and unstabilised soil, which implies that the design models are valid only for semi hard columns with a maximum shear strength of a given value that will depend on the results of laboratory tests.

Design of the preloading stage is based on characteristic values. When using the observational method, for example deviations from the predicted settlements will provide a basis for the decision whether a temporary surcharge can be removed, the surcharge must be increased or the preloading period should be extended.

The design should mostly be based on column strength from field tests.

Design requirements

The design method presented in this document is based on the prestandard version of Eurocode 7, ENV 1997-1. In accordance with the Eurocode philosophy in relation to soil parameter values a distinction is made between measured values, derived values, characteristic values and design values.

The derived value is the value of a ground parameter obtained by theory, correlation or empiricism from the measured test results. A characteristic value is determined from the derived values to give a cautious estimate of the value affecting the occurrence of a limit state. This terminology will be used in the following section of the Design Guide.

The determination of the derived and characteristic values shall be in accordance with the principles of Eurocode, subject to the restrictions on the characteristic values of some parameters recommended in this document.

Service life

SLS calculations are carried out using characteristic values of parameters.

Settlement calculations should also be based on the assumption that the distribution of load between columns and unstabilised soil is on the basis that at every level the same compression occurs in columns and in the unstabilised soil.

Deep stabilisation should be combined with preloading including a temporary surcharge. The purpose of surcharge is to consolidate the soil for a load higher than the service load. The surcharge should be designed so that parts of it can be removed at the end of the preloading period. This will reduce or eliminate future creep settlements.

In design it is presupposed that a settlement calculation is performed. This calculation is a basis for a prognosis of the settlements during the construction stage and the service stage respectively.

A careful follow-up (e.g. settlements, pore pressures) during the construction stage is essential for verifying the behaviour. The deep stabilisation method shall be used together with active design (observational method).

The stipulated service life is stated in construction specifications (Cf. Eurocode 7 and National Regulations).

Limit states

The ULS mechanisms to be considered in the design of stabilised soil columns are to include failure of the column itself and overall failure through the columns and the untreated ground. The design parameters for ULS shall be based on the characteristic values divided by an appropriate partial factor. Eurocode permits the use of partial factors lower than those given in EC 7 for certain temporary conditions. This Design Guide gives recommendations on the appropriate partial factor to be used under such conditions in the design of soil stabilisation.

The design of stabilised ground must satisfy ultimate and serviceability limit states.

To satisfy ultimate limit state (ULS) requirements, the design of the stabilised ground must be such that there is a low probability of collapse of the supported structure. This includes failure due to prior excessive deformation in the ground or a risk of danger to people or severe economic loss.

A column stabilisation and a mass stabilisation are designed to give the structure or embankment and its close surroundings satisfactory overall stability, so that failure of the structure or a part of this is not caused by large deformations (Cf. Eurocode 7 and National Regulations).

As stated above, the design method presented in this document is based on the prestandard version of Eurocode 7, ENV 1997-1. This version of EC 7 requires that three design situation should be considered in ULS analysis, namely Cases A, B and C. Case A mainly refers to buoyancy problems and must be considered when this relates to the particular design situation under consideration. The general application of this Case A will not be discussed in this Design Guide.

Case B relates to the strength of structural elements and is therefore not applicable to stabilised soil itself. Case B will not be discussed further, although there may be cases involving stabilised soil/structural interaction in which this case would be applicable.

Case C governs the safety margins against failure of the soil and is relevant to limit state analysis of stabilised soil. The following discussion therefore mainly relates to Case C.

To satisfy serviceability limit state (SLS) requirements, column stabilisation and mass stabilisation, including transition zones to unstabilised embankments shall be designed in such a way that the total and the differential settlements along and across the road surface satisfy the requirements in Eurocode 7 or national regulations. The SLS must include consideration of long-term creep movements.

Durability

The choice of characteristic material values should consider the durability of the deep stabilisation.

Execution process of deep mixing

The purpose of standardised laboratory tests (laboratory mixing tests) is to provide information on binder type and dosage appropriate for the actual construction. The tests should include each representative soil layer. In most cases, there is a difference between laboratory strength and field strength. The preliminary process design is based on the laboratory test results, database and information about similar experience. Before the actual construction, deep mixed test columns are constructed on which field trials are carried out to confirm that the dosage, type of binder and mixing energy yield the required strength and uniformity. In case field trials fail to satisfy the requirements given in the design, the functional and process design have to be reconsidered.

Choice of binder

When mixing the binder with soil the chemical reactions start immediately. When cement is used a stabilising gel between the soil granules is created due to pozzolanic reactions. A very homogeneous mixing is required since cement, unlike lime, does not diffuse.

When using pulverised binders based on lime the soil reactions continue for several months:

- the water content of the soil decreases since water is consumed during the chemical reactions;

- the lime reacts with the clay minerals;

- calcium ions will diffuse from zones of high binder concentration both within the stabilised volume and to adjacent zones originally not involved in the mixing. Consequently, the homogeneity and strength of the stabilised volume is improved.

The geo-mechanical properties of the stabilised material largely depend on the type of binder. In general, the strength and brittleness of the stabilised soil increase with increasing amount of cement.

On the other hand, the ductility will increase with increasing amount of lime.

The binders used in dry mixing usually consist of cement or a mixture of lime and cement, in wet mixing of cement. The choice of binder is a critical aspect of deep mixing, which largely depends on the soil conditions and the purpose of deep mixing. Testing of binders with the soil to be treated is normally an essential requirement on any deep mixing project. A summary of the binders commonly used is given in Table XVI.

Table XVI – Binders commonly used in dry mixing (European-Standard, 2005)

Soil types	Suitable binder
Clay	Lime or Lime/cement
Quick clay	Lime or Lime/cement
Organic clay and gypttja	Lime/cement or cement/granulated blast furnace slag or lime/gypsum
Peat	Cement or cement/granulated blast furnace slag or lime/gypsum/cement
Sulphate soil	Cement or cement/granulated blast furnace slag
Silt	Lime/cement or cement

The binder used in wet mixing is in most cases cement .Specially prepared binders may be used for highly organic soils or for extremely soft soils with high water contents. Mixtures of fly ash, gypsum and cement may be used in cases where low strength of the treated soil is preferred. Bentonite is frequently used to improve rheology and stabilise the slurry mixes.

Testing

In this paragraph, we are not considering in details the different existing methods but we discuss the process that must be followed to fulfil the design requirements.

For settlement reduction, the elastic modulus value is of main interest, while for improvement of stability and elimination of the risk of failure, the strength of the columns is of main interest. As regards immobilisation and/or confinement of waste deposits or polluted soil and containment, overlapping and low permeability of the columns are the determining factors.

Laboratory testing

Laboratory testing represents one of the means used for analysing the possibilities of treating the actual soil and checking the result of deep mixing. It includes on one hand laboratory mixed soil samples and on the other hand samples taken at various depths in the columns installed.

<u>Laboratory mixed samples</u>

Laboratory mixed samples offer a possibility to study which quantity of binder, which type of binder, or combination binder/filler/admixture, which binder factor and water/binder ratio are required to stabilise the soil for the intended purpose.

For the laboratory investigation of soil/binder samples, reference is given to the procedures from (EUROSOILSTAB, 2000):

- laboratory procedure for preparation and storing of the test samples of soil stabilised by binders for deep mixing column applications;

- laboratory procedure for preparation and storing of test samples of soil (especially peat) stabilised by lime and cement-type materials for mass stabilisation applications.

The correlation between the strength properties of laboratory mixed samples and the corresponding properties under field conditions is very uncertain. If extensive experience is available of the correlation between the strength characteristics of laboratory mixed samples and the corresponding characteristics of columns installed in soil of equal geological origin as the laboratory mixed samples, a conservative correlation coefficient can be applied. The same type of mixing tool, binder and binder content should be used as in the reference object.

<u>Core samples</u>

Core samples can be taken by means of a rotary core drilling equipment. Core samples can be used to study deformation characteristics, strength and uniformity. Choice of coring technique and core diameter is highly dependent on the treated soil strength and grading. Triple tube samplers are recommended for columns in soft soils.

The strength characteristics and the elastic modulus E_{col} of the samples are normally determined by unconfined compression tests. However, the results thus obtained will be affected by the existence of cracks in the samples. If cracks can be observed, triaxial testing is preferable. The compression modulus M_{col} of the samples is determined by oedometer tests. For assessing the settlement behaviour of the stabilised soil, the elastic modulus of the column is more representative than the oedometer modulus. The use of oedometer modulus in settlement analysis instead of the elastic modulus of the column leads to an underestimation of the long-term settlement.

Hydraulic conductivity tests require special equipments built for the purpose, as no standard apparatus exists. However, the permeability can be estimated by back-calculation from the value of the coefficient of consolidation determined by oedometer tests.

Wet grab samples

Wet grab samples are samples taken prior to initial set of the treated soil. They are extracted from the columns at specific depths with a suitable sampling tool, usually one per 500 m^3 of treated soil volume or one per day. The samples are obtained by lowering an empty wet grab sampling device to the sample depth, capturing the fluid sample, closing the wet grab sampling device, and bringing the sample to ground surface where the material is processed and placed into cylinders for testing. The samples are cured at a prescribed temperature in standard size sample mould, cylinders or cubes. Testing of the samples, as described above, is normally performed after 7 days and 28 days of curing. Curing conditions of the treated soil in situ on the one hand and of the wet grab soil sample on the other, are different and influence the strength and the rate of strength increase.

Field testing

Field trial tests

Because of the uncertainty regarding the applicability of the column characteristics determined in the laboratory, in-situ tests are required. One of the most important issues, namely to investigate the uniformity of the columns, can be fulfilled by some type of sounding, or by core boring as mentioned above, and/or by lifting up whole columns. Determination of the mechanical and hydraulic conductivity properties of the columns requires special equipments. A field trial test for this purpose usually comprises two to three column installations with varied binder content. Another important aspect of field-testing is to determine the criteria for the construction control of deep mixing. The construction control values may include penetration and retrieval rate of the mixing tool, rotation speed and torque of the mixing tool, overlapping width and rate of delivery of binder/slurry. When a column has to be founded in a firm bearing stratum, the torque and/or the change of penetration resistance are measured to establish the critical construction control values.

Direct determination of mechanical properties

Pressumeter tests can serve as a basis for determination of the shear strength and the compressibility of the column. The tests require preboring of a hole in the column into which the pressumeter can be inserted. Geophysical tests serve as a basis for determination of the properties of the treated soil under dynamic action and can be used for investigation of the integrity of the columns and also for indirect determination of the deformation modulus and strength.

Investigation of uniformity and indirect determination of mechanical properties

Cone Penetration Tests (CPT) are used for determination of the strength parameters and the continuity of the column.

Column penetration test is carried out using a probe that is pressed down into the centre of the column at a speed of 20 mm/s and with continuous registration of the penetration resistance (European-Standard, 2005).

Load test and test procedure

(Baker, 2000) presented the experimental work carried out on the Stora Viken North site. Due to the mixing procedure and equipment, the uppermost part of the lime/cement column is usually weak compared to the rest of the column, and it happens that it consists of only remoulded soil. About 0.9 m of the columns top was in this case excavated to identify the real column quality, see Figure 46, and a dummy column of concrete was cast in place on top to facilitate the future load tests.

Figure 46 – Preparation of the top of the columns with and without MOPS (Baker et al., 2005)

Four of the six columns were equipped with a preinstalled wire with a bottom plate at the bottom of the column, called MOPS. The MOPS was used for loading of the columns.

(a) **(b)**

Figure 47 – Test arrangement (Baker et al., 2005)

By applying a tension force in the wire using a hydraulic jack, and gradually increasing the force, the column was loaded at both ends, Figure 47 a. This procedure is much easier, compared to a dead weight supported loading of the top, se Figure 47 b.

To be able to measure the deformation of each column at different depths during the load test, four holes were drilled shortly before the load test, and during testing, tell tales were installed and readings of deformations could be made at the surface with an accuracy of 1/100 of a mm.

Loading was done by increasing the vertical load in increments of 10 kN up to the column failure, or until the rate of deformation became so large that the loading equipment could not keep pace with it. Each load step was kept constant for 16 minutes, and readings were taken after 1,2,4,8 and 16 minutes.

For the two columns, which were not equipped with MOPS, the results (Baker et al., 2005) show a similar behaviour as that for the columns with bottom plate, especially column 1.

The only difference is that the tests were terminated when the total bearing capacity of the column, that is the sum of the shaft resistance and the point bearing capacity, was exceeded.

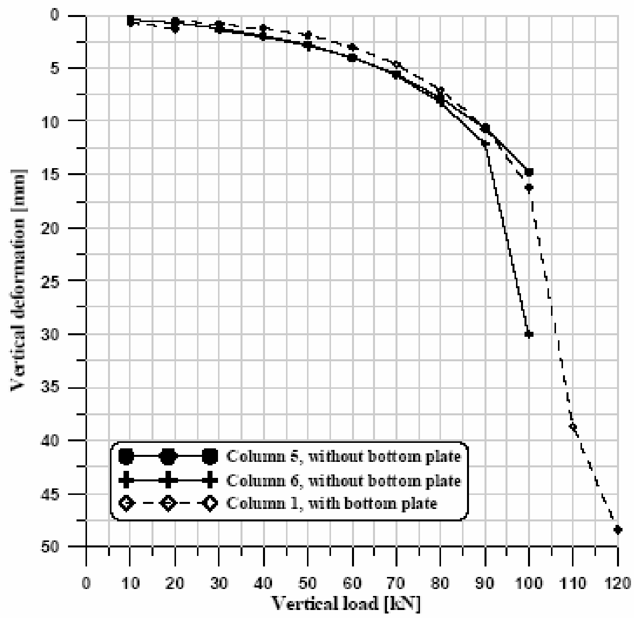


Figure 48 – Vertical deformation at the top of the columns a function of load (Baker et al., 2005)

In addition, to achieve local measurements (shaft friction), different methods can be used such as gauges installation at different depths or other methods like the French Removable extensometer.

In the experiment carried out within the INNOTRACK project ((Rocher-Lacoste and Le Kouby, 2008)), the columns were instrumented using the LCPC removable extensometer (Bustamante and Gianeselli, 1996) and (Bustamante and Gianeselli, 2001). A ϕ 52/60 mm diameter closed ended steel tube was installed for this purpose in the middle of the column just after the soil-cement column had been set up.

A column head was built for each soil-cement column on the same day as the column; 60 cm squared concrete block and 25 cm thick. During the loading test, the load was applied on the column head. The top of the steel tube reached the top of the column head.

When the soil-cement column load test came through, several segments of packers and steel ribbons (with strain gauges stuck on it) were installed in the steel tube to perform the strain measures. An example of the instrumentation of a column using the removable extensometer is shown on Figure 49. A typical result of a loaded soil-lime-cement column is given on Figure 50 from a test carried out in Goteborg (Sweden) (Bustamante and Gianeselli, 2003).

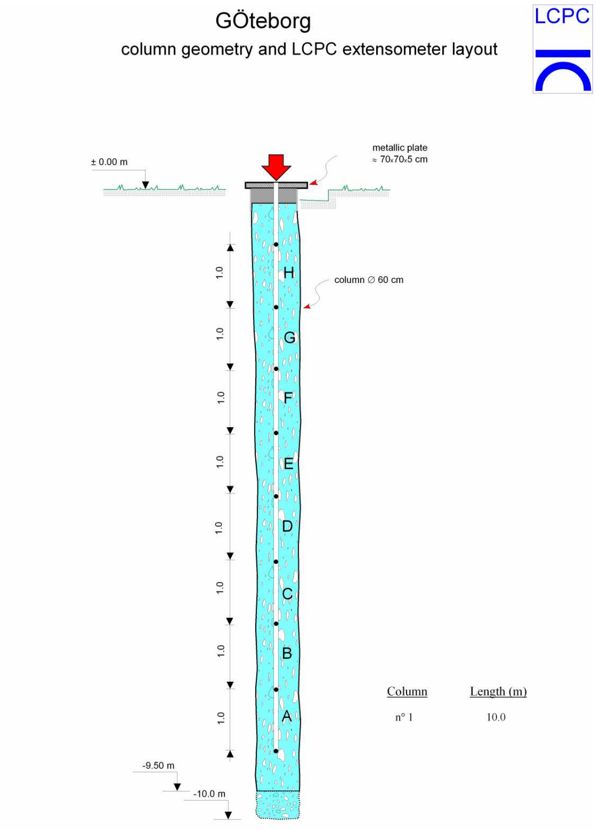


Figure 49 – Monitoring of columns with the LCPC extensometer method (Bustamante and Gianeselli, 2000)

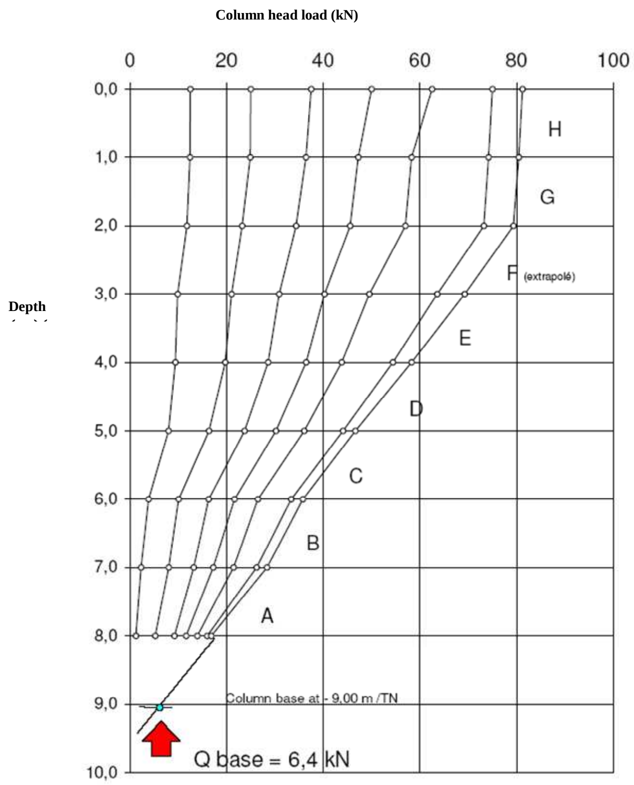


Figure 50 – Column n°1, load distribution (Bustamante and Gianeselli, 2003)

Test for Determination of in situ Hydraulic Conductivity

The test equipment is basically the same as used for flow tests in boreholes in bedrock. This requires that a hole is drilled in the centre of the column and the equipment consists of a probe, a cylinder and a digital pressure gauge (see Figure 51). Stiff plastic tubes are used to connect the different parts of the equipment.

The probe consists of a single-ended metal standpipe open at the top. Two inflatable seals 0.5 m long at each end are used to isolate that part of the standpipe perforated with small holes. The system is flushed making sure no air is entrapped, and is then gradually pressurized. Care must be taken so that the pressure does not cause cracking of the lime/cement column, yet it must be large enough to create an effective sealing. The flow of water from the cylinder and through the column is monitored as a function of time and pressure. The hydraulic conductivity can be evaluated, using a correlation factor, usually called shape factor, Brand (1980, mentioned by (Alen et al., 2005b), see the equation below :

$$K_{col} = \frac{1}{F} \cdot \frac{Q}{\Delta H}$$

With

K_{col} hydraulic conductivity of the column (m/s).

Q flow rate (m3/s).

ΔH difference in head (m)

F shape factor (m)

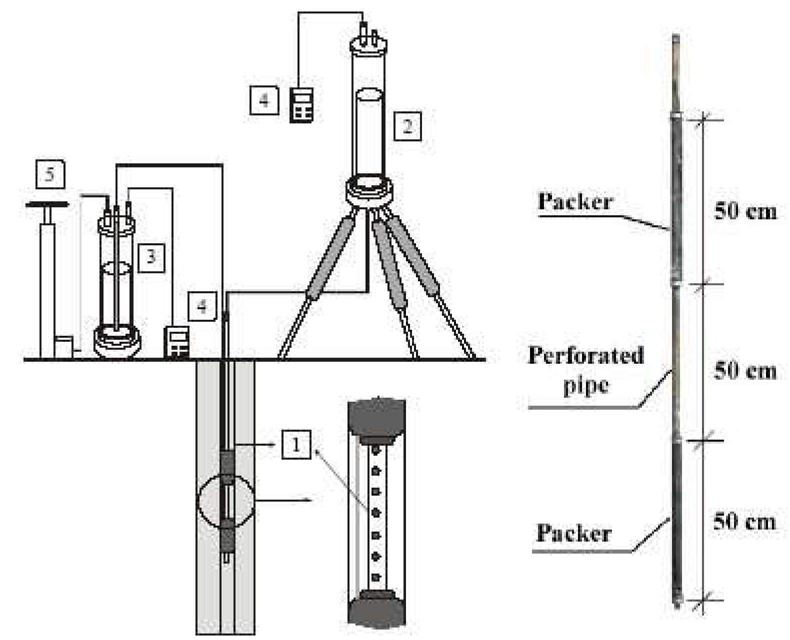


Figure 51 – Hydraulic conductivity test instrument (Baker et al., 2005)

The shape factor depends on the geometry of the columns and the test equipment and is typically in the order of 0.6 to 0.9. A finite element model was utilized for the determination of the shape factor. Two columns were tested, at 2 and 3 different depths respectively. The results are given in (Baker et al., 2005).

In the beginning of the tests somewhat higher discharges are obtained, but as the tests progress the discharges decrease. The reason for this might be the presence of air or as water becomes available the column material swells and certain existing fissures close. However after a couple of hours fairly stable values are obtained. By using the above equation, the hydraulic conductivity value for column 7 was about 0.5 to 1 10^{-8} m/s while, for the other column it is closer to 5 10^{-8} m/s. This indicates that the hydraulic conductivity of the columns is 5 to 50 times larger compared to the intact clay.

Correlation of various properties of treated soil

From the Swedish experience, the strength of a laboratory-made stabilised test sample is usually significantly higher than the strength of a corresponding material from the field. The difference is mainly due to a more efficient mixing of the binder and soil in the laboratory. Also the prevailing temperature in a laboratory is more even and differs from the temperature in the field conditions.

The former is apparent when comparing the strength of well mixed laboratory test samples with the strength of samples from similar but less homogeneously mixed columns. In laboratory test samples the attainable strength is usually from 10 to 50 times higher than the strength of the natural (not stabilised) soil. In column stabilisation the attainable strength is normally from 20 % to 50 % of the strength of the laboratory test pieces.

For the CDM method (Cement Deep mixing Method, Japan) – the most common wet mixing method in Japan and the DJM method (Dry Jet Mixing Method), the ratio is given in Figure 52.

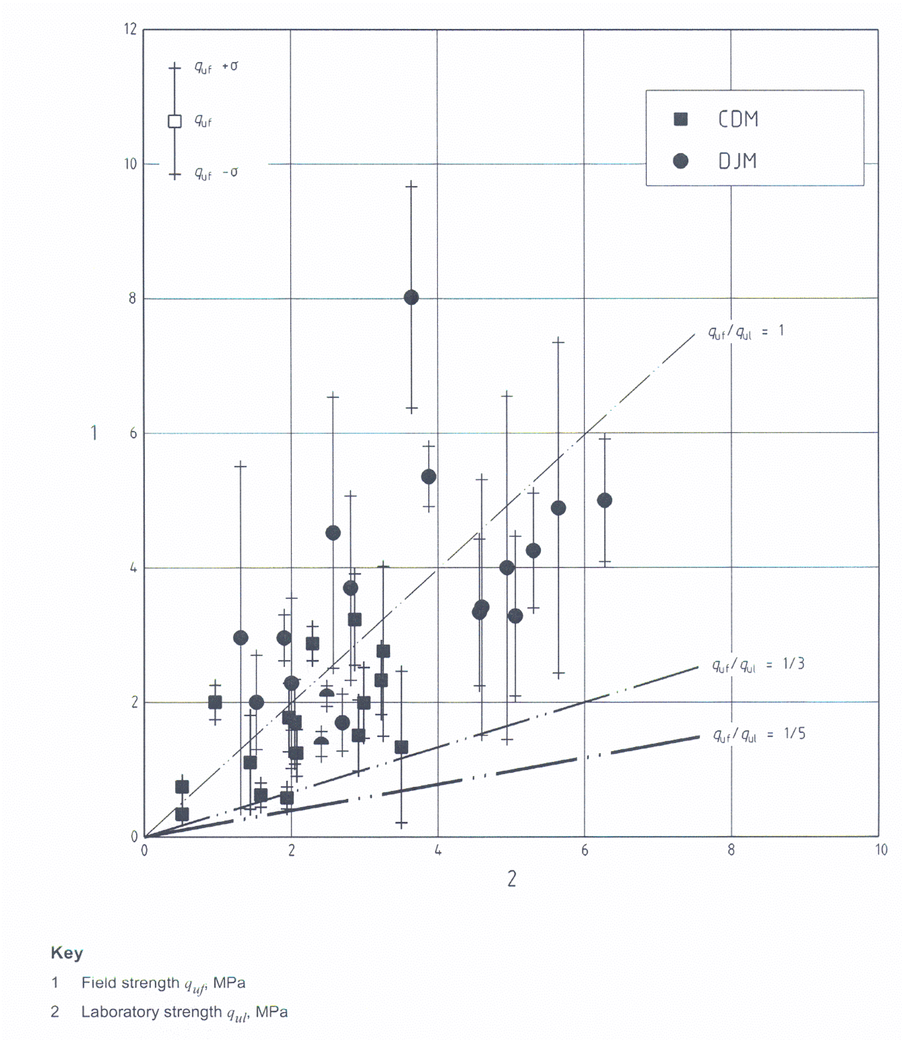


Figure 52 – Relation between strength results of field and laboratory tests for on-land constructions (European-Standard, 2005)

Aspects of design

Three domains are concerned in this section, stability, settlement and confinement.

From a literature review, we have noticed different patterns of soil-reinforcement as a response to the specific issue.

Square or triangular grid patterns of single or combined columns are usually applied when the purpose of DM is reduction of settlement and, in some cases, improvement of stability. Common examples are road and railway embankments.

Soil mixing can be done to a replacement ratio of 100 per cent wherein all the soil inside a particular block is treated, as is usually the case for shallow mixing applications, or to a selected lower ratio, which is often practised with DM. The chosen ratio reflects, of course, the mechanical capabilities and characteristics of the applied method. Depending on the purpose of DM works, specific conditions of the site, stability calculations and costs of treatment, different patterns of column installations are used to achieve the desired result by utilising spaced or overlapping and single or combined columns. Typical patterns are presented in Figure 53.

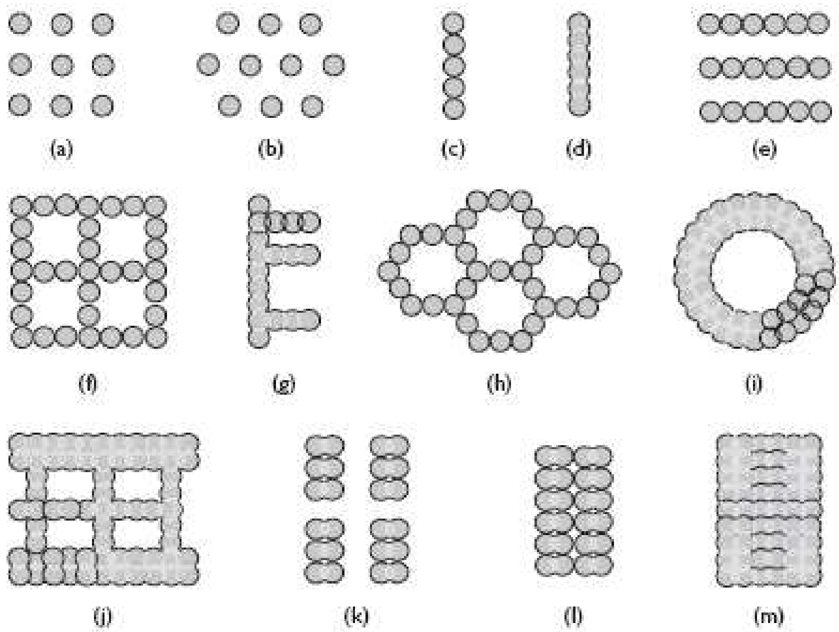


Figure 53 – Examples of deep soil mixing patterns: (a), (b) column-type (square and triangular arrangement);(c) tangent wall ; (d) overlapped wall ; (e) tangent walls; (f) tangent rigid, (g) overlapped wall with buttresses ; (h) tangent cells ; (i) ring; (j) lattice ; (k) group columns ; (l) group columns in-contact ; (m) block (Moseley and Kirsch, 2004).

The purpose of using DM is mainly the reduction of settlement and the increase of bearing capacity of weak foundation soil, as well as prevention of sliding failure (Figure 54). For on-land projects the applications usually comprise road and railway embankments, buildings, industrial halls, tanks, bridge abutments, retaining walls and underground facilities. For waterfront and marine applications they can include quay walls, wharfs, revetments and breakwaters.

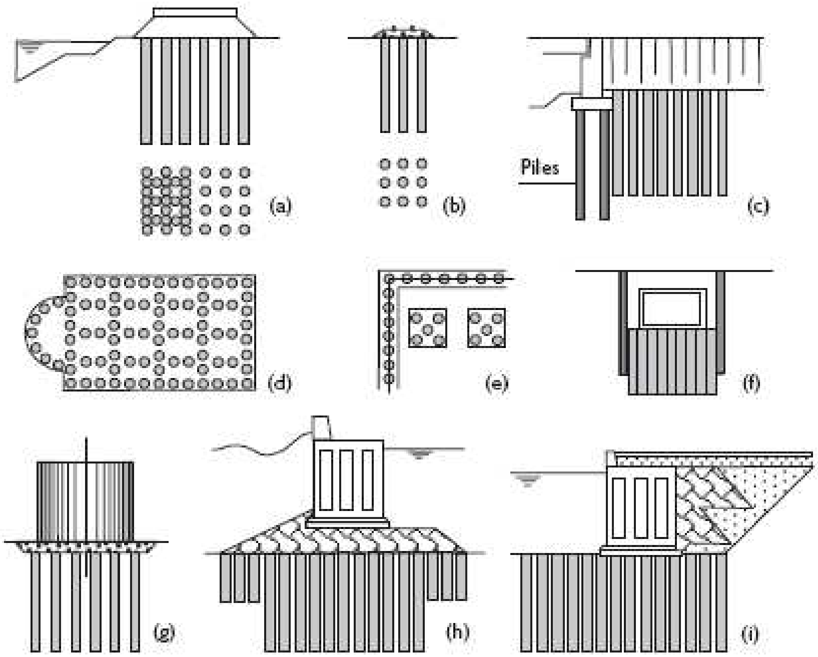


Figure 54 – Examples of DM application for foundation support (schematic): (a) road embankment; (b) railway embankment; (c) bridge approach zone; (d) slab foundation; (e) strip and pad foundations; (f) culvert; (g) tank; (h) breakwater ; (i) quay wall (Moseley and Kirsch, 2004)

General conditions for dry soil mixing

Calculation methods which have been found reliable are set out below (as far as possible, limitations known at present are indicated). According to present practice, lime and lime-cement column reinforcement is in most cases designed on the basis of total safety philosophy, i.e. without partial coefficients. Swedish and European standard, however, require partial safety factors.

Lime and lime-cement columns are inhomogeneous to varying degrees, with an irregular structure and properties varying in different directions. The columns are primarily intended to interact with clay when the columns are loaded axially. For other load combinations, the shear strength of the columns may be lower than under axial loading. Structures in which the columns are subject to tensile stresses shall be avoided.

Recommendations in (SGF, 1997) give some requirements for equivalent applied load. The slip-surface calculations allow to design the bearing capacity of columns during the construction stage.

Attention should be given to the installation stage as the construction of columns has the temporary effect of reducing the bearing capacity of soil during this stage.

Design in the ultimate limit state

Initial choice of type of geotechnical structure – calculation with characteristic values. When choosing the geotechnical structure the safety factor is calculated for characteristic values. The safety factor for the construction on unstabilised soil (i.e. the construction but without columns) shall be higher than 1.0. In some cases this means that temporary loading berms are needed. If the factor of safety with respect to failure of an unstabilised embankment (including loading berms if any) is higher than 1.0, the columns may be placed in a square or rectangular pattern.

When the factor of safety with respect to failure (unstabilised embankment) is lower than 1.0 and there is no space for loading berms, columns in the shear zone shall be placed in panels or grids. In stability calculations, the assumed shear strength of the columns should be limited to 100 kPa (lower values can of course apply when tests on columns in the field or laboratory mixed specimens give lower values). Under favourable conditions, shear strengths up to 150 kPa may be used at greater depths, e.g. under fill, with a factor of safety F > 1.2 for unstabilised soil (i.e. the same construction but without columns).

Stabilisation in the passive zone of slip surfaces should be avoided unless it is made in the form of panels or blocks. The soil strata outside the stabilised volume shall also have adequate bearing capacity to carry the loads transmitted to the unstabilised soil by column stabilisation.

The slope of the ground surface influences the design of stabilisation. If the slope of the ground surface is steeper than 1:7 and the factor of safety for the unstabilised embankment is lower than 1.2, the columns shall be placed in panels. Stabilisation in the shear zone shall be designed in the form of panels.

Design

According to EC1 the uncertainties in the calculation model can be accounted for by using γ_{Rd}. No practice has been established on how to choose γ_{Rd} when dealing with column or mass stabilisation. In the equations suggested below γ_{Rd} =1.0. Further research is needed to derive a suitable value especially when stabilisation is made in organic soils.

Design shall be performed by combined analysis and by undrained analyses. Combined analysis means that the lowest value of τ_{fd} (design shear strength) or τ_{fu} (ultimate shear strength) is selected for each section of the slip surface. When assessing pore pressures the original pore pressure conditions shall be regarded as well as the influence from column installation and loading. The approach described below assumes that stabiliser is present over the entire cross section of the columns, and that the columns are homogeneous.

The following values are recommended in stabilised columns in clay and organic clay (if no laboratory values are available):

$$c'_{k(col)} = \beta . c_{ukcol} \quad [1]$$

With:

$c'_{k(col)}$: effective cohesion of column.

c_{ukcol} : cohesion of column.

β : calculation factor for effective cohesion of column encompassed between 0 and 0.3.

β =0 : in the passive zone.

β =0.1 : in the direct shear zone.

β =0.1 : in the active zone.

The angle of shearing $\phi'_{k(col)}$ is taken equal to 30°.

The drained shear resistance:

$$c'_k = a\, c'_{k\,(col)} + (1-a)\, c'_{k\,(soil)} \quad [3]$$

$$\tau_{fdk} = c'_k + \sigma' \tan \phi'_k \quad [4]$$

with:
$a = A/c^2$, for rectangular column pattern
A = area of cross section of columns
c = distance between column centres
Undrained parameters are obtained from Equations:

$$c_{uk} = a c_{uk(col)} + (1-a) c_{uk(soil)} \quad [5]$$

$$\tau_{fuk} = c'_{uk} \quad [6]$$

The above principle of calculating the stability of embankments on stabilised soil is based on full interaction between columns and soil. When soils in which creep deformations are in progress are stabilised, full interaction between columns and unstabilised clay cannot be relied on.

Column stabilisation for embankments

Present experience of column stabilisation in soft organic soil is limited. Embankments higher than 2 m normally presuppose the use of loading berms. Note that the safety factor for the construction on unstabilised soil (i.e. the construction but without columns) shall be higher than 1.0, see previous section "Initial choice of type of geotechnical structure – calculation with characteristic values".

The bearing capacity of the stabilised soil during different stages of construction shall be determined by slip surface calculations. Installation of columns has the temporary effect of reducing the bearing capacity of soil during the construction stage. This should be taken into account. Loading on stabilised soil results in high pore pressures in soil and columns. In construction specifications recommendations are given for load application sequence, possible restrictions on excavation and restrictions on future land use in the vicinity of the stabilisation.

The following check calculation shall always be performed. Further checks may be necessary depending on the purpose of reinforcement, design, etc.

- The factor of safety for the planned embankment (without column reinforcement).
- Factor of safety during load application, with checks on maximum permissible load increments/level differences and slope gradient. Check on working sequence.
- The factor of safety for the column reinforced embankment during the construction stage with temporary surcharge, limitations concerning temporary storage sites, construction traffic, etc.
- Factor of safety during the service stage for the completed embankment with traffic load.

In slip surface calculations, the columns are made so long that the slip surfaces which pass below the reinforcement have a satisfactory factor of safety. The slip surfaces which pass substantially through the reinforcement shall have at least the same factor of safety. This presupposes that the strength of clay and column is mobilised simultaneously and that the columns together with the clay behave as a rigid body. In such cases, slip surface calculations can be based on a weighted shear strength in the active portion of the slip surface in accordance with Equation [3] and [5].

During installation of columns, mixing in some zones may be substandard and strength may therefore be lower. In those cases it is essential to impose the following limitations in design:

- a disturbed zone in the unstabilised soil below each column;
- reduced strength over the top metre length of the column.

The extent of the disturbed zone below the columns depends on the design of the mixing tool and the column diameter. For columns of 0.5-0.6 m diameter, a disturbed zone of approx. 0.5 m in length is normally obtained below the column. In the disturbed zone reconsolidation will occur in the long term.

Pressure feed of binder shall normally be stopped 0.5-1.0 m below ground level to prevent the binder being blown back along the shaft. This means that the top metre of the column may have varying properties. The strength in this section may be lower than that of the original dry crust.

Stability calculations are performed with a weighted value of shear strength; see Equation [3] and [5]. If the factor of safety with respect to failure of an unstabilised embankment (including loading berms if any) is sufficient, the columns may be placed in a square or rectangular pattern.

Design in the service state

General considerations on calculation models are:

Deep stabilisation should be combined with preloading by temporary surcharging. The purpose of surcharge is to consolidate the soil for a load higher than the service load. Removal of part of the surcharge load at the end of the preloading period reduces future creep settlements.

Requirements in the serviceability limit states are specified by the client. Requirements generally refer to settlements during the service life. Large settlements during the preloading stage can be accepted if very small settlements during the service stage can be achieved.

The load on an area stabilised with columns is carried partly by the columns and partly by the unstabilised soil between the columns. The compression modulus of the columns is considerably higher than that of the unstabilised soil. Settlements under load will therefore be significantly smaller on a stabilised surface than on an unstabilised surface.

The calculation model presented below has its origin in the model for lime columns described by Broms (1984, mentioned by (EUROSOILSTAB, 2000)). The model has also been used for soft and semi-hard lime cement columns, see Rogbeck et al (1995, mentioned by (EUROSOILSTAB, 2000)).

Settlements within the stabilised soil volume are influenced by the following factors:

- the ratio of the compression modulus of the columns to that of unstabilised soil;
- the proportion of the stabilised surface occupied by columns;
- the consolidation properties of the soil;
- the bearing capacity of the columns;
- the time of load application in relation to column installation;
- the permeability in unstabilised soil and in the columns.

The calculation model assumes that the depth of soil is uniform and that all columns penetrate to the same depth. Since there is a variation in the properties of unstabilised soil and in the effect of binder stabilisation, it may be economical to use columns of different lengths. In such a case calculations regarding the magnitude of settlements must be made for different column lengths.

Distribution of load between columns and stabilised soils and Distribution of load between columns and unstabilised soil are calculated on the assumption that the same compression occurs in columns and unstabilised soil at every level. This implies that the load on the unstabilised soil is gradually transferred to the columns and that the load is transmitted to the bases of the columns, as shown schematically in Figure 55. Settlements in the soil below the columns are calculated on the assumption that the load is transmitted to the bases of the columns. The permeability of the columns is higher than that of unstabilised soil, and the columns therefore speed up the consolidation process. This means that the stratum below the columns may be assumed to be drained by the columns.

The compression modulus of columns increases with time. Due to different methods of mixing and stress ratios, the development of compression modulus is different in the field and the laboratory. The results of settlement calculations should therefore be given as probable maximum and minimum values.

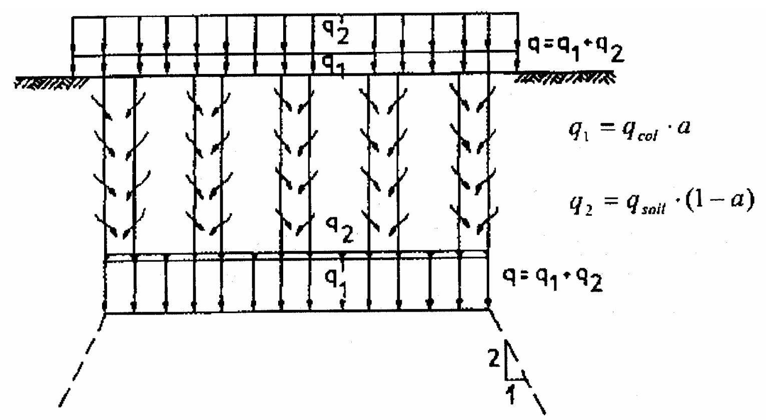


Figure 55 – Distribution of loads.

Design of the preloading stage is based on characteristic values. By using the observational method, possible deviations from the predicted settlement can be found by settlement measurements during the construction phase. This will provide a basis for deciding when a temporary surcharge can be removed, whether the surcharge must be increased or the preloading period extended.

The load-deformation curve in stabilised columns may be assumed to conform to the curve in Figure 56. The curve is linear up to the long-term strength (creep strength) of the columns, and the slope of the curve represents Young's moduli of the columns, E_{col}. Once the long-term strength has been exceeded, load on the columns is assumed to be constant. The load-deformation relationships described are used to calculate the distribution of load between the columns and unstabilised soil.

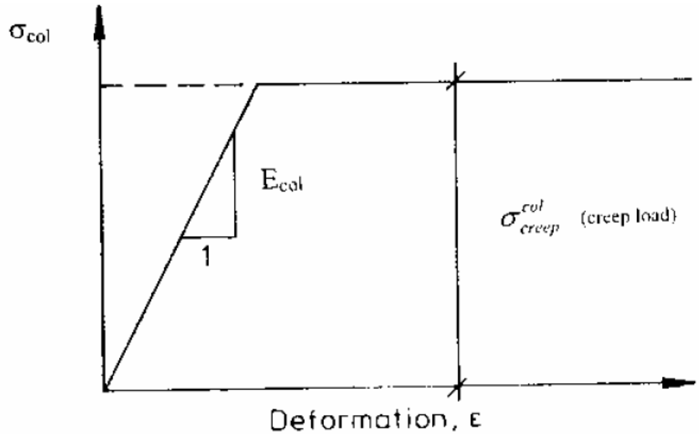


Figure 56 – Assumed load-deformation curve in column of stabilised soil (EUROSOILSTAB, 2000).

The ultimate strength σ_{ult} is a function of the shear strength c_{uk} of the columns and the effective horizontal pressure σ'_h on the columns, according to the empirical expression:

$$\sigma_{ult} = 2.c_{u,col} + 3.\sigma_h' \qquad [7]$$

σ'_h is the horizontal effective stress between the soil and the columns. It can be put equal to the original effective vertical pressure in the soil due to the deformations, which occur when the stabiliser is mixed in. Equation [7] is to some extent based on total stress analysis with $\phi = 30°$ in the column. Distribution of load between columns and unstabilised soil is calculated by an iteration process. Normally account is also taken of the fact that the horizontal pressure increases when a load is imposed on the area stabilised by columns. The increase in horizontal pressure is assumed to be 50% of the imposed load on the soil, according to Equation [8], and this means that the creep load of the column increases and the column thus takes a larger load.

$$\sigma_h' = \sigma_{v0}' + 0,5.\sigma_{v0}' \qquad [8]$$

The long-term strength of stabilised columns, σ_{creep}, can be put at 70-95% of the ultimate strength. If the long-term strength of the column is 90% of its ultimate strength, this means that the individual column is designed to carry the maximum load q_{1max}. The equations proposed by (SGF, 1997) and (EUROSOILSTAB, 2000) are :

$$q_{1\max} = 0,65.a.\sigma_{ult} \qquad [9]$$

$$q_{1\max} = 0.90 a \sigma_{ult} \qquad [9]b$$

where:

$a = A/c^2$, for rectangular column pattern

A = area of cross section of columns

c = distance between column centres

The creep load varies with the distance below ground level. The load q_1 carried by the individual column is at all times less than the total load q. The load q_2 on the unstabilised soil is calculated as the difference between the total load q and the load q1 carried by the columns.

$$q_2 = q - q_1 \qquad [10]$$

Settlement of a group of columns

Settlements within the area stabilised by columns are calculated by dividing the soil profile into characteristic strata. Settlements in the columns are calculated in accordance with Equation [11] where Δh is the stratum thickness.

The settlement reduction is generally about 25 to 80% thanks to the stabilisation method based on columns.

$$s_1 = \sum \frac{\Delta h}{a} \frac{q_1}{E_{col}} \qquad [11]$$

Where:
s_1 = settlement in the column, m
Δh = stratum thickness, m
q_1 = load on column as above, kPa
a = ratio of areas as above
E_{col} = Young's modulus of column, kPa

Settlement in the unstabilised soil is calculated in accordance with Equation [12]

$$s_2 = \sum \frac{\Delta h}{1-a} . \frac{q_2}{M_s} \qquad [12]$$

where:
s_2 = settlement in unstabilised soil, m
$q_2/(1-a)$ = load on unstabilised soil as above, kPa
M_s = compression modulus of unstabilised soil, kPa

A first calculation is made by assuming that $q_1 = q_{1max}$. The calculated settlement S_1 in the columns is compared with the calculated settlement S_2 in the unstabilised soil. If $s_1 > s_2$, a load transfer is performed by gradually reducing q_1 and correspondingly increasing q_2, so that finally $s_1 = s_2$. The calculated settlement s_m is then equal to s_1 and s_2.
If the soil is normally consolidated, s_m can be calculated from Equation [13].

$$s_m = s_1 = s_2 = \sum \frac{\Delta h . q}{a.M_c + (1-a).M_s} \qquad [13]$$

If however $s_1 < s_2$, the columns cannot take any more load, and the settlement s_m which occurs is equal to the calculated settlement s_2 in the unstabilised soil.
Settlements within the mass stabilised area are calculated by assuming the mass stabilised volume to behave as a linear elastic perfectly plastic layer. All of the load q is carried by the mass stabilised volume. The strength must be chosen at such an extent that the yield strength of the stabilised soil is not exceeded. The settlement is calculated in accordance with Equation [14]. Note that considerable settlements can be derived during the curing period (when the load only consists of the working platform) and these settlements have to be calculated separately.

$$s_m = \Sigma \, \Delta h \;\; q / M_m \qquad [14]$$

where:
s_m = settlement in the mass stabilised volume, m
Δh = stratum thickness, m
q = load on mass stabilisation, kPa
M_m = compression modulus of mass stabilised soil, kPa

In the case of floating columns, the settlement of soft soil layer beneath the column tip can be calculated by the shallow foundation approach. The total settlement will be the sum of the settlement of each layer.

When using mass stabilisation a preloading working platform should be applied soon after the stabilisation work. This compresses the stabilised volume and increases its strength. The amount of settlement is much depending on the soil to be stabilised. For peat and dredging mud quite a large settlement can occur due to the compression (compression could be up to 30-35 %). In the laboratory procedure suggested for preparation and storing of test samples for Mass Stabilisation Applications it is suggested that the compression of the test sample should be measured in the laboratory. These recordings can be used for calculation of the immediate settlements. However, these settlement develop rapidly. The settlements of the mass stabilised layer in the service time are usually small. If columns are made beneath mass stabilisation the settlement calculation for the columns stabilised volume is carried out as previously explained. Calculation of settlement as above holds only for the stabilised volume. Calculation of settlement in strata below the stabilised volume is carried out in the traditional way. No spread of load is assumed to occur in the stabilised volume.

Settlement rate

When the effective stress in the soil is less than the preconsolidation pressure, settlements will develop rapidly.

When the effective stress in the soil exceeds the preconsolidation pressure, the rate of consolidation settlement in the stabilised soil stratum is calculated in the same way as for vertically drained soil.

Experience shows that the permeability of the macrostructure of the column is 200-600 times higher than of unstabilised soil. As stated earlier it is essential to make a prognosis of the magnitude and rate of settlement during the preloading time. Today the columns are casually considered as drains and the theory does not take into account the increase of strength in column with time and loading. For calculation of the rate of settlement, the permeability of lime stabilised organic soil may be assumed to be approx. 1000 times as high as that of unstabilised clay. The permeability of soil stabilised with other binders (e.g. lime/cement) can in the calculation be assumed to be 200-600 times as high as that of unstabilised soil. The permeability of stabilised soil is difficult to estimate in advance and therefore results from calculation of rate of settlements shall not be given as exact values but in an interval. For fill on top of lime and lime cement columns with the columns spaced at 0.8 - 1.8 m between centres, the rate of settlement can be approximately calculated from an equation for radial flow (originally from Barron, (1948), and modified as presented in Åhnberg et al, (1986, mentioned by (EUROSOILSTAB, 2000)); see also Hansbo, (1979, mentioned by (EUROSOILSTAB, 2000)), mentioned by (EUROSOILSTAB, 2000)).

Note that calculation of the rate of settlement is only approximate. Monitoring shows that the calculated rate of settlement is broadly correct when 80-90% of the total settlement has developed.

$$U = 1 - \exp\left[\frac{-2.c_{vh}.t}{R^2.f(n)}\right] \qquad [15]$$

where:
U = degree of consolidation
c_{vh} = coefficient of consolidation in unstabilised soil in the horizontal direction and for vertical deformation normally assumed to be equal to 2 c_{vv}
c_{vv} = coefficient of consolidation in unstabilised soil in the vertical direction and for vertical deformation
t = period of consolidation
R = influence radius of columns

For columns installed at distances c between centres in a square grid or one made up of isosceles triangles, the influence radius is R = c/(π)$^{1/2}$ = 0.56c. If the columns are placed in a grid of equilateral triangles, R = 0.53c.

$$f(n) = \frac{n^2}{n^2-1}\left[\ln(n) - 0,75 + \frac{1}{n^2}.\left(1 - \frac{1}{4.n^2}\right)\right] + \left[\frac{n^2-1}{n^2}.\frac{1}{r^2}.\frac{k_{sol}}{k_{col}}.L_D^2\right] \qquad [16]$$

where n=R/r:
r = column radius
c = distance between column centres
L_D = column length with drainage upwards only and half column length with drainage both upwards and downwards
k_{soil} = permeability of unstabilised soil
$k_{co}l$ = permeability of columns

The rate of settlement as above holds only for the stabilised volume. Calculation of the rate of settlement below the stabilised volume is performed in the traditional way, bearing in mind that the columns drain into the top of the stratum.

Bearing capacity of columns

Single column

The ultimate bearing capacity of a single column is governed either by the shear strength of the surrounding soil (shear failure) or by the strength of the column material (column failure). In the case of soil failure, the ultimate bearing capacity of a single column depends both on the skin friction resistance along the surface of the column and on the end resistance. The short term ultimate bearing capacity of a single column can be expressed as:

$$Q^{col} = \left(\pi.d.l_c + \frac{9\pi d^2}{4} \right).c_u \qquad [17]$$

Where d : diameter of the column (usually d = 0.5 to 0.6 m)

l_c : length of the column

c_u : average undrained strength of the surrounding soft soil determined

In Equation [17], it is assumed that the skin friction is equal to the undrained shear strength c_u and the base resistance is approximated to be 9 c_u.

The mobilised resistance in the soft layer is very weak in comparison to the resistance mobilised in the substratum.

The base resistance of floating columns is generally low compared with the skin resistance, while the base resistance of columns that extend down to a bearing layer can be high. A large part of the applied load will then be transferred to the bearing layer through the bottom of the columns. Further details on the bearing capacity of single columns can be found in Broms (1991, mentioned by (Topolnicki, 2003)).

Group of columns

Similar to single columns, the ultimate bearing capacity of a column group is governed either by the shear strength of the untreated soil between the columns and the shear strength of the column material. Possible shear failure of a group of columns is shown in Figure 57. The bearing capacity of a group of columns arises from the skin resistance along the perimeter of the column group (2 c_u l_c (B + L)) with B : width of the group, L : length of the group, l_c : length of the column. And the base resistance of the block, which is 6 to 9 times the undrained strength of the soil c_u. Hence, the total bearing capacity of a group of columns can be written as:

$$Q_{ult}^{group} = 2.c_u.l_c(B+L) + (6.to.9).c_u.B.L \qquad [18]$$

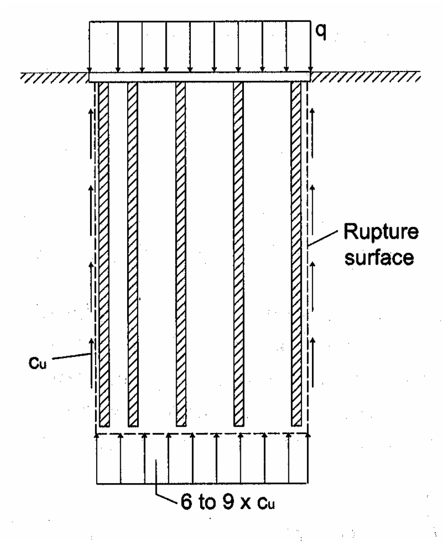


Figure 57 – Shear failure of column group as a block (Topolnicki, 2003)

The factor 6 refers to a foundation with L > B, whereas the factor 9 can be used for square foundations. However, a relatively large deformation, 5 to 10 percent of the width of the loaded area, is required to mobilise the maximum base resistance. It is therefore proposed to neglect the base resistance in the design (Broms 1991, mentioned by (Topolnicki, 2003)).
The ultimate bearing capacity of a group of columns may also be governed by local shear failure along the edge of the block. This kind of ultimate bearing capacity can be calculated in the same way as the stability of slopes, but it can also be estimated from:

$$q_{ult} = 5.5\ c_{u,av}\ (1+0.2\ b/l)$$

Where b and l are the width and length of the locally loaded area, and $c_{u,av}$ is the average shear strength along the assumed rupture surface.

General conditions for wet soil mixing

Increasing use of the wet method of deep soil mixing to improve the engineering and environmental properties of soft or contaminated ground indicates growing international interest and acceptance of this relatively new technology. In this method of ground treatment soils are mixed in-situ with cement or lime-cement mix in a slurry form. Wet deep soil mixing is currently applied for stabilisation of the soil to a maximum depth of about 50 m. The slurry is injected into the soil through hollow rotating mixing shaft with various types of cutting tools at its tip. The mixing shafts are also equipped with continuous or discontinuous auger flights, mixing blades or paddles to increase the efficiency of the mixing process. In some methods, the mechanical mixing is enhanced by simultaneously injecting fluid grout at high velocity through nozzles in the mixing or cutting tools (Topolnicki, 2003).
The wet mixing method can be applied in soft clays, silts, fine grained sands and organic soils (peat, sludge, etc.). The use of wet mixing in organic soil however requires a special binders and execution procedures. The mechanical property of the stabilised soil depends on the binder property, on the history of deposition and behaviour of the soil, mixing and curing conditions. The historical development of the wet deep soil mixing method can be found in Topolnicki 2003. According to FHWA 2000 mentioned by (Topolnicki, 2003), the wet deep mixing method is classified into two major groups based on the mix operation. These are mechanical mixing and a combined jet and mechanical mixing. Within these groups, there is the possibility of injecting the slurry through the holes along the length of the shaft or the end of the shaft or a combination of both of them.

Strength of wet deep soil mixing columns

The strength of a single column is governed by the shear strength of the soil (soil failure) or by the shear strength of the column material (column failure). The strength of the column material can be determined in laboratory, for example, using the unconfined compression tests. Field and model investigations and even load tests are often conducted to assist the design procedure. The strength of the column mainly depends on the type and amount of binder and working specifications; such as the rate of penetration and withdrawal, rotation speed, injection method, mixing tool, water/binder ratio, etc. This requires a good understanding of the complexity of soil-binder physics, chemistry and mechanical property.

The strength and deformation parameters can reasonably correlated with the compressive strength of the column material, in other terms the unconfined strength. In practice, the unconfined compressive strength is the key parameter for the current design of stabilized columns due to its simplicity in testing and its cost effectiveness. At present the unconfined compressive strength of the column material cannot reliably be predicted based on the properties of the native soil and the type and amount of the binder. Therefore, it is generally recommended to conduct in advance appropriate trial tests on stabilized soils to obtain more adequate data. At this stage the relationship between the unconfined strength and the binder factor α can be determined. The binder factor α is expressed as the ratio of weight of injected dry binder to the volume of the ground to be treated ((Topolnicki, 2003)).

(Topolnicki, 2003) compiled the field strength and permeability of wet deep mixing columns from literature for ranges of cement factors and different type of soils as shown in Table XVII.

The volume ratio defined as the ratio of the volume of slurry injected to the volume of ground to be treated vary greatly and reflect the type of mixing technique used, but it generally lies in the range of 15 to 50% and in most cases between 25 and 40%. The lower volume ratio, the higher will be the efficiency of the mixing mainly due to higher rotational speed or jet assistance. The relationship between the unconfined compressive strength and other parameters are also compiled in Table XVIII.

Table XVII – Typical field strength and permeability (Topolnicki, 2003)

Soil type	Cement factor α [kg/m³]	28-days unconfined compressive strength q_u [MPa]	Permeability k [m/s]
Sludge	250 - 400	0.1 – 0.4	1×10^{-8}
Peat, organic silts/clays	200 - 350	0.3 – 1.2	5×10^{-9}
Soft clays	150 - 300	0.5 – 1.7	5×10^{-9}
Medium/ hard clays	120 - 300	0.7 – 2.5	5×10^{-9}
Silts and silty sands	120 - 300	1.0 – 3.0	1×10^{-8}
Fine-medium sands	120 - 300	1.5 – 5.0	5×10^{-8}
Coarse sands and gravels	120 - 250	3.0 – 7.0	1×10^{-7}

Table XVIII – Typical correlations for cement-treated soils (Topolnicki, 2003)

Selected parameters	Expected values/ratios or relationships
Gain in the unconfined compressive strength with time	$q_{u,28\,day} = ca.\ 2 \cdot q_{u,4\,day}$ $q_{u,28\,day} = 1.4\ to\ 1.5 \cdot q_{u,7\,day}$ (silts, clays) $q_{u,28\,day} = 1.5\ to\ 2 \cdot q_{u,7\,day}$ (sands) $q_{u,60\,day} = 1.4\ to\ 1.5 \cdot q_{u,28\,day}$ (clays, silts)
coefficient of variation in the unconfined compressive strength	*0.2 to 0.6* (typically *0.35 to 0.5*), It is lower for laboratory mixed samples than for field samples
Relative strength ratio:	
core samples to laboratory mixed samples, λ,	0.5 to 1, lower values for clays higher for sands (1.0 for offshore works in Japan)
core samples to wet grab samples	1 to 1.5
Shear strength (direct shear with no normal stress)	$0.4\ to\ 0.5 \cdot q_{u}$., *for* $q_u < 1$ *Mpa* $0.3\ to\ 0.35 \cdot q_{u}$., *for 1 Mpa* $< q_u < 1$ *Mpa* $0.2 \cdot q_{u}$., *for* $q_u > 1$ *Mpa*
Tensile strength	$0.08\ to\ 0.15 \cdot q_u$, but not higher than *200 kPa*. Indirect splitting tests yield lower values than direct uniaxial tests
Secant stiffness modulus E_{50}, at 50 % peak strength	$50\ to\ 300 \cdot q_{u}$., *for* $q_u < 2$ *Mpa* $300\ to\ 1000 \cdot q_{u}$., *for* $q_u > 2$ *Mpa* (ratio increases with increasing the unconfined strength.)
Axial strain at failure, ε_f :	
unconfined compression test (crushing failure)	*0.5 to 1.0 % for* $q_u > 1$ *MPa* *1 to 3 % for* $q_u < 1$ *Mpa,*
confined compression tests (plastic shear)	*2 to 5 %* (undrained triaxial test)
Poisson's ratio	0.25 to 0.45, typically 0.3 to 0.4

<u>The bearing capacity of the stabilised foundation</u>

If a stabilized soil is likely to behave as a rigid structural member embedded in the ground, its external stability can be evaluated under modes of failure typical for gravity-type structures, including horizontal sliding, overturning, bearing capacity and rotational sliding. Related DM patterns which can be analyzed with this approach comprise mainly block-type improvement and, with certain simplifications, also “blocks” composed of long and short walls, as it has been practiced in Japan for various port facilities. In the latter case, however, is also necessary to examine the extrusion failure mode of untreated soil remaining between the long walls of stabilized soil when subjected to unbalanced active and passive earth pressure, generated for instance by an earthquake (Terashi, Tananka and Kitazume, 1983, mentioned by (Topolnicki, 2003)).

The principles of calculation of the bearing capacity and settlements of both single columns and column groups from wet deep mixing are similar to that for dry deep mixing columns above.

5.3.4 Installation procedure

General

An example of installation procedure is given in the following table.

Table XIX – Installation procedure (EUROSOILSTAB, 2000)

Installation procedure
Column stabilisation
Pre-installation procedure (dry and wet column methods)

Before installation of column, the following conditions shall be checked and documented:

- Data for binders
  - production date and delivery date
  - storage conditions
  - transportation
  - storage temperature
  - test to confirm the binder quality
  - binder components
  - water type/quality
- Machinery equipment
  - type of equipment
  - design of mixing tool
  - all other relevant data
- Site description
  - location and site elevation level
  - geotechnical conditions
  - weather conditions during installation
  - photos from site
  - state of eventual soil contamination
- Column data
  - diameter, m (usually 500-800mm)
  - amount of binder, kg/m or litre/m
  - mixing energy, j/m^3
  - lifting speed, mm/s and mm per revolution
  - rotation speed, rpm
  - length, m (up to 25 m)
  - column top level (elevation)
  - column tip level (elevation)
  - feeding pressure, max MPa (applies to both wet and dry method)
  - exhaust pressure (inside Kellybar) at mixing tool level
  - water to cement weight ratio (for wet method)
  - ratio of grout and additives

Plant and equipment

General

The equipment used consisted of an installer, a carrier and a materials supply station. The installer was a crawler-based rig, with a rotary engine mounted on a leader. A purpose-designed mixing tool was fitted to the leader and the binder was distributed into the ground by compressed air. The machine generates a very low ground pressure and the working range is up to 7 meters. The machine can reach ± 3 m in height, the total weight of the installer was 48 tons, generating a ground pressure of approximately 35 kPa. The binder is fed from the carrier to the installation machine. The carrier consists of two silos, one with cement and one with lime. A load cell system is placed under each silo and supplies the on-board computer with information on the amount of material fed each second through the system from each silo. Every silo has a maximum capacity of 12 tons. When the silos are empty, it is refilled at the materials supply station. The total weight of the carrier is 30 tons (54 tons when filled with binder, generating a ground pressure of 55 kPa).

The supply station consists of two bulk tankers, each with a capacity of about 40 m^3. The Hercules Limix system is based on a completely computer controlled installation process. Initially, site-specific installation parameters, such as rotational speed, withdrawal speed and binder amount, were loaded into the onboard PC. The operator enters the column number, adjusts the mixing tool into position and drills down to the required depth. From there onwards, the computer controls the installation process. The rotation speed and the withdrawal speed of the mixing tool are governed by the rate at which the binder leaves the silos. Due to the frequent variations in distributed binder, the withdrawal speed is constantly re-calculated and varies throughout the column. This ensures that the binder is correctly distributed at each level in the soil. The operator overviews the process on his screen and can abort it if necessary. All relevant data for each column are stored for quality control. One advantage of a computer controlled system is that every column is manufactured in the same way, regardless of the operator’s skill or attentiveness.

Preparation of stabilizer

When stabilizer is used which consists of two or more materials, mix these components together in the required proportions and in a quantity sufficient to perform the required tests. For wet mixing, form a slurry by mixing the stabilizer with water to obtain the required water -stabilizer ratio (m/m).
Use the bulk unit weight as determined under ”Homogenization of soil” and the required dosage of stabilizer to calculate the necessary amount of stabilizer or stabilizer slurry. Dry stabilizer in the case of dry mixing, and stabilizer slurry in the case of wet mixing, is added to the soil in the mixer. Soil and stabilizer are mixed until the mass is visually homogeneous. In the case of fibrous peat, limit the mixing time to prevent destruction of fibres. If necessary, manually move soil stuck to the mixing bowl to the centre. Note the time used for mixing. Take out two small samples and determine their water content. Protect the mixed soil from drying out before it is applied to form a sample.

Dry method

There is extensive experience in Scandinavia with installing vertical columns beside the railway embankment to increase the stability of the embankment, as shown in Figure 58. We can take the example of the Ledsgard site (see (Holm et al., 2002)).

The dry deep mixing was carried out over a period of two weeks in summer 2000 by the Swedish contractor Hercules Grundläggning. Two photos from the works are shown in Figure 59 and Figure 60. The Limix rig for dry deep mixing soil improvement is shown in Figure 60. The works were carried out according to plans. Execution of the soil improvement was a routine type of job. However, it is important to achieve an accurate overlap of the columns as well as relatively homogenous columns versus depth so that the requirements on static and dynamic stiffness are fulfilled. A relatively stiff drilling rod, an accurate inclinometer on the leader and precise positioning of the mixing tool make it easier to obtain column overlap. Driving rain and gusts during installation of the trial columns reduced neither the installation capacity nor the quality of the columns. However, it is important to have efficient equipment for dehumidification of the compressed air so that the binder remains dry until reacting with the water in the soil.

Substantial heave was observed during column installation, which lifted the neighbouring track to some extent and necessitated its re-levelling. The columns were installed from a working bed (sub-ballast material), which was later removed to expose the tops of the columns. Had the stiff, treated soil not been reached with this procedure, the plan was to dig a trench to ensure good contact with the treated soil.

Figure 58 – Installation of deep mixed columns through embankment at field test (LCM, 2002)

Figure 59 – The dry DMM column installation works with a commuter train passing on the neighbouring track (Holm et al., 2002)

Figure 60 – Limix rig for dry DMM soil improvement (Holm et al., 2002)

Deep soil mixing – Wet method

The deep wet mixing equipment is shown in Figure 61 including the separate mixing and holding tanks and pump which is connected to the deep mixing rig by flexible pipeline. The mixing is by high shear colloidal mixers to ensure each binder particle is dispersed into the slurry. The holding tanks have paddle agitators to keep the binders from settling out of the slurry. The deep mixing plant has similar dimensions to those used for dry mixing.

Figure 62 shows typical tools for wet mixing, having one or more mixing blades with teeth fitted and one or more nozzles for the binder delivery. The wet mixing tools tend to be of a larger diameter with consequently thicker blades with binder delivery nozzles along the blades. The wet soil mixing tools also vary in size but can be made to make columns up to 2.4 m diameter.

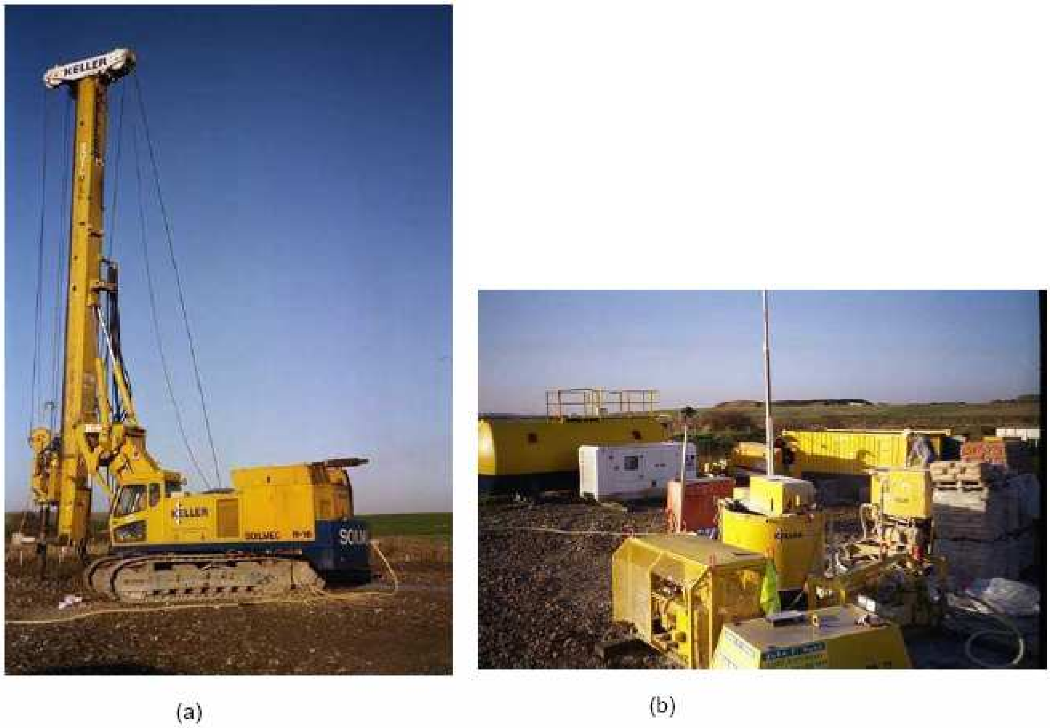


Figure 61 – (a) Deep wet mixing plant with (b) separate mixing and holding tanks and pumps ((EUROSOILSTAB, 2000)).

Figure 62 – Mixing tools for deep wet mixing (Holm and Smith, 2006)

Procedure and method

Dry method

Typical mixing tools used in the deep dry mixing usually consist of single nozzle for the binder delivery, a horizontal and curved or angled cutting blade. These tools vary in size but are usually made to produce mixed columns in the 500 mm to 800 mm diameter range.

1. The mixing tool is pushed vertically into the soil down to the prescribed depth. If rotation is used, the rate of rotation speed shall be recorded. The time for pushing as well as the depth of penetration shall be documented.

2. The mixing tool is lifted and simultaneously rotated. During lifting, the binder, usually dry cement and lime, is injected to soil. The injection is made from the centre of the mixing tool by rotation for the mixing tool wings. The amount of binder per cubic meter and also the supplied energy per cubic meter shall be prescribed. Continuous monitoring shall be carried out automatically for:

Typical values

- binder output, kg/m and kg/m^3 16-50 kg/m
- mixing energy, J/m^3
- lifting speed, mm/s and mm per revolution 20-50 mm/s
- rotation speed rpm 100-200 rpm
- feeding pressure at rig 0.2-0.7 kPa
- exhaust pressure (inside Kellybar) at mixing tool level 0.2-0.6 kPa

3. The feeding pressure shall be released after installation, surface heave shall be monitored both around one single column (see Figure 63 a) and around the whole stabilised area (see Figure 63 b).

The accuracy shall be ± 1 mm. One point each 200 m^2 will be sufficient.

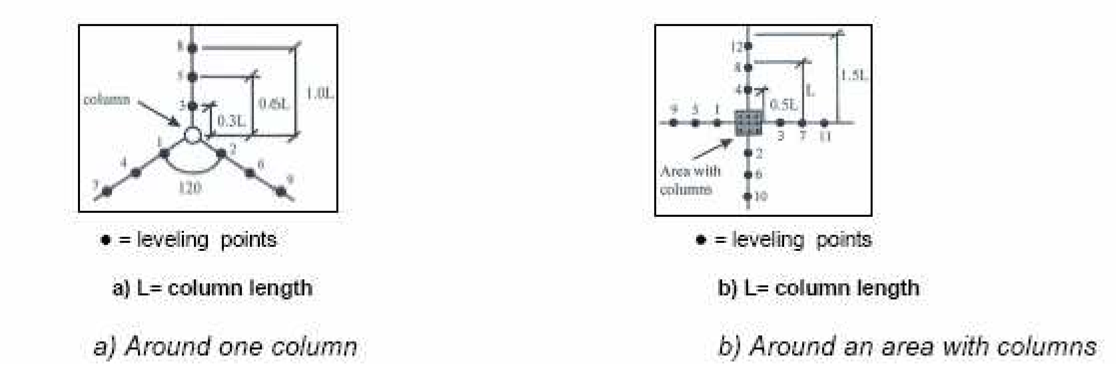


Figure 63 – Monitoring of surface heave before and after installation (EUROSOILSTAB, 2000)

4. For some columns, it is recommended that the temperature be measured in a number of columns.

As an example, the soil improvement work using dry deep mixing at Ledsgård consisted of upgrading 370 m of the western track. The following principal procedure was followed for the Ledsgård site (Holm et al., 2002):

1. Disassembly of tracks and removal of cable groover
2. Excavation of top ballast and disposal to depot
3. Excavation of sub-ballast to 800 mm below base of rail and disposal to depot

4. Setting out of dry DMM columns

5. Installation of dry DMM columns

6. Removal of non-mixed soil above columns and exposure of the top of the

columns

7. Back filling and compaction of ballast

The time schedule for the total project was approximately three months (starting in May 2000), where the deep mixing occupied two weeks (excluding trial columns).

Since the binders used to form the columns are soil-dependent, each individual site has to be evaluated separately. The first step consisted of mixing trials in the laboratory on soil samples from the site. These trials provide the initial design parameters. Trial columns were then installed on site in order to:

- Verify the design parameters
- Optimise the installation parameters
- Detect possible layers/areas with poor stabilisation effect.

The trial columns were installed within the construction area under conditions similar to those during construction. A total of 12 trial columns were installed with a length of approximately 8 metres.

The final binder recipe was established. Soil improvement was executed with two different patterns, grid pattern and single columns according to Chapter 4. The following summarises a number of parameters:

- Column diameter 600 mm
- Column length 6, 7 or 13 m (average length 7.6 m)
- Total length 13,000 m
- Binder quantity 120 and 150 kg/m3
- Binder type: unslaked lime and standard Portland cement with 50/50 and 25/75 blend
- Area coverage: about 54 % in the grid part and about 21 % in the single column part
- Column overlap for wall pattern: 0.1 m
- Installation capacity per 16 hours: 1,000 m/rig

Table XX – Typical execution values of the Nordic and Japanese dry mixing techniques (European-Standard, 2005)

Mixing machine	Nordic technique	Japanese technique
Penetration speed of mixing shaft	2,0 m/min to 6,0 m/min	1,0 m/min to 2,0 m/min
Retrieval speed of mixing shaft	1,5 m/min to 6,0 m/min	0,7 m/min to 0,9 m/min
Rotation speed of mixing blades	100 revolutions/min to 200 revolutions/min	24 revolutions/min to 64 revolutions/min
Blade rotation number[1)]	150 per m to 500 per m	≥ 274 per m
Amount of binder injected	100 kg/m^3 to 250 kg/m^3	100 kg/m^3 to 300 kg/m^3
Retrieval (penetration) rate	10 mm/rev to 30 mm/rev.	10 mm/rev to 35 mm/rev.
Injection phase	Typically during retrieval	Penetration and/or retrieval

<u>Wet method</u>

In wet mixing, the binder is usually cement slurry. Filler (sands and additives) may be added to the slurry when necessary. The specific quantity of slurry added can vary with depth. For machines with the outlet below the mixing tool, the slurry need not be added during the retrieval phase.

1. The mixing tool is inserted (bored or pushed) vertically into the soil down to prescribed depth. While the mixing tool is pushed down, injection of slurry could be made. If rotation is used, the rate of rotation speed shall be recorded. The time for pushing as well as the depth of penetration shall be noted. If injection of slurry is made, the amount shall be noted as well as the mixing energy (J/m^3) and the rate of penetration. If the mixing tool is rotated with injection at the bottom level for some time, this has to be documented (see time-output and time-lifting speed curves).

2. During the injection of slurry, continuously monitoring shall be carried out automatically for:

Typical values

- output of slurry, litres/minute 100 to 250 litre/min
- input pressure at machine, kPa max. 20 bar
- output pressure, kPa (if possible) 0 to 10 bar
- lifting speed, m/s 0.1 to 0.5 m/s
- rate of rotation, rpm 10 to 20 rpm

3. The amount of slurry overflowing at the top of the column shall be measured during installation until the slurry flow is cut off.

4. Starting before and up to 48 hours after installation, surface heave shall be monitored both around one single column (see fig. A.1a) and around the whole-stabilised area (see fig. A.1b). The accuracy shall be ±1 mm. One point each 200 m2 will be sufficient.

5. If cement is used the water to cement weight ratio shall be documented (typically in the range 0.5 to 2.0). However other binders such as lime and/or bentonite are also used depending on the application.

For the wet mix method the columns can be up to 1.2 m diameter and so the torque of the mixing pipe and blades can be up to 160 kN.m with rotations at 15 to 20 revs/min and feed rates of 50 cm/min. The binder water slurry flow rates are generally around 35 to 70 litres/min.

The amount of binder is typically in the range of 300 to 400 kg/m^3 in soft organic soil to give a field strength (c_u) of 100 to 150 kPa.

The most likely effect on nearby structures is from heave during the deep mixing. In the case of deep dry mixed column a 5 to 10 cm heave is not uncommon within 0.5 m of the edge of a column during stabilisation work in soft clay. For deep wet mixing with high dosages and high slurry pressures heaves of up to 0.75 m have been measured. However these heaves are local to the columns and would only be a problem if the stabilisation was within one column diameter of a building foundation.

For the dry mix method the processes from the blending of materials to the use of the binder may be contained in one or two items of plant. For the wet method there will need to be separate items of plant.

The wet mix process blends the materials with water in a high shear mixer to form a slurry at the design water to solids ratio. The binder slurry is then transferred to reservoirs that continually agitate the slurry to ensure that the constituents of the mix do not separate. The binder slurry is then pumped at the required flow rate to the deep stabilisation machine.

The proportion of binders used in the wet mixing is controlled by quantities of materials added to the high shear mixer. In the case of the dry mix method the binders are stored in separate silos and the feed rate into the air stream adjusted until the rate of loss of the material from the silos is as previously calculated to give the correct mix proportions.

An example of the experiment carried out in France, TGV station (Rocher-Lacoste and Le Kouby, 2008). The columns were built following the current procedure :

- Set up of a ϕ 170 mm steel tube across the railway platform and allowing the opening up of the blades of the drilling rotary head at a depth of embedment of about 1.50-1.75m.
- Drilling of a ϕ 168 mm hole with a tricone down to a depth of 7m,
- Drilling of a 400mm soil-cement column from 1.50-1.75m to 7m,
- Withdrawal of the 170mm steel tube,

- Filling up of the hole with grout from the bottom of the sublayer to the bottom of the ballast and then with ballast.

Figure 64 – Built up of ϕ400mm diameter soil-cement columns on the side track (SOLETANCHE-BACHY)

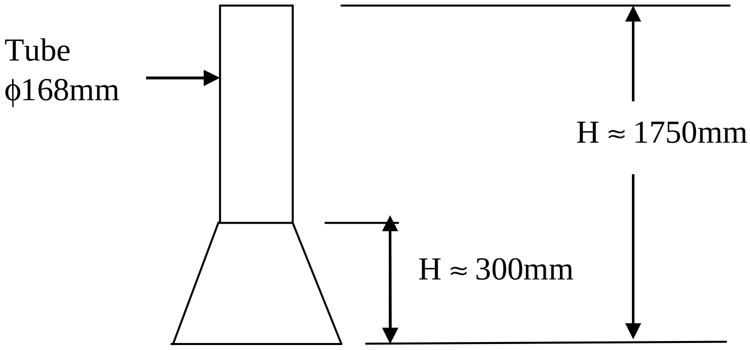
The column built under the track following the procedure described in Figure 64 was extracted for research purpose. An important result was that some grout was stuck to the tube but the grout did not spread within the ballast layer and the underlayers. Figure 65 shows the excavation of a column as well as a schematic transverse view.

From the top to the bottom, we can describe :

- the visible part of the tube (□168mm) used to make the tool go down to the existing soil at a depth of 1.36m. The thin grout layer around the tube shows a viscous grout which did not spread up. This was due to the tool which required a diameter bigger than 168mm to deploy during the opening phase
- -the different layers of the railway platform can be identified; the ballast layer, the treated silt, stone cement layer and brought soil from earthworks,
- the soil-cement column begins a few centimetres below the fill layer. Between 1350mm and 1750mm (end of excavation), the diameter of the column increases (cone shape) as the tool needed a certain length to deploy before building up the 400mm diameter column.

Some usual values are given in Table XXI and Table XXII.

(a)



(b)

Figure 65 – Excavation of columns on side track (a) Excavation of the single column and (b) General description of the excavated column (Rocher-Lacoste and Le Kouby, 2008)

Table XXI – Major capacity and execution of European and Japanese wet mixing techniques (European-Standard, 2005)

Equipment	Details	On land, Europe	On land, Japan	Marine, Japan
Mixing machine	Number of mixing rods	1 to 3	1 to 4	2 to 8
	Diameter of mixing tool	0,4 m to 0,9 m	1,0 m to 1,6 m	1,0 m to 1,6 m
	Maximum depth of treatment	25 m	48 m	70 m below sea level
	Position of binder outlet	Rod	Rod and blade	Rod and blade
	Injection pressure	500 kPa to 1 000 kPa	300 kPa to 600 kPa	300 kPa to 800 kPa
Batching plant	Amount of slurry storage	3 m^3 to 6 m^3	3 m^3	3 m^3 to 20 m^3
	Supplying capacity	0,08 m^3/min to 0,25 m^3/min	0,25 m^3/min to 1 m^3/min	0,5 m^3/min to 2 m^3/min
Binder storage tank	Maximum capacity		30 t	50 t to 1 600 t

Table XXII – Typical execution values of European and Japanese wet mixing techniques (European-Standard, 2005)

Mixing machine	On land, Europe	On land, Japan	Marine, Japan
Penetration speed of mixing shaft	0,5 m/min to 1,5 m/min	1,0 m/min	1,0 m/min
Retrieval speed of mixing shaft	3,0 m/min to 5,0 m/min	0,7 m/min to 1,0 m/min	1,0 m/min
Rotation speed of mixing blades	25 rev/min to 50 rev/min	20 rev/min to 40 rev/min	20 rev/min to 60 rev/min
Blade rotation number	mostly continuous flight auger	350 per meter	350 per meter
Amount of binder injected	80 kg/m^3 to 450 kg/m^3	70 kg/m^3 to 300 kg/m^3	70 kg/m^3 to 300 kg/m^3
Injection phase	Penetration and/or retrieval	Penetration and/or retrieval	Penetration and/or retrieval

The production rates will vary depending upon the diameter of column mixed, the power of the stabilisation machine and the in situ strength of the soil. Table XXIII below gives guide values to assist in estimating production rates.

Table XXIII Guide values of the volume of soils that can be stabilised per hour by the different processes (EUROSOILSTAB, 2000)

Process	Typical depth of treatment (m)	Volume of soil treated/hour (m/hr)
Deep dry mixing in 0.6 m dia columns	20	15 to 20
Deep wet mixing in 0.8 m dia columns	20	12 to 20
Mass stabilisation	6	100

Development and adjustment of monitoring and control system

Many types of existing equipment do not monitor or report production data in a way, which is sufficient to satisfy the expected test specifications.

Monitoring

The following data shall be monitored automatically and continuously during the process of installation:

- amount of binder / flow rate
- lift speed
- depth
- revolution rate
- internal and external pressure
- applied energy (if possible)
- applied power (if possible)
- pushing and lifting force (if possible)

and also if possible temperature.

All data shall be stored on a PC-card or similar. The data shall be presented on a graphical user interface to the operator in order to make it easy to adjust the installation process as necessary. It shall be possible to view all data as function of time and depth. All presented measured values shall be unchanged.

A paper copy shall be possible to print on site. The complete documentation shall be registered on the PC-card. The data on the PC-card shall be printed out on a separate PC. An example of a documentation layout produced from a PC is shown in Figure 66 and Figure 67.

It is obvious that instrumentation for monitoring the stabilization process carefully is very important since the stabilization process itself seldom lends itself to direct inspection. The amount of binder injected in a certain soil volume, as well as the geometry and homogeneity of the stabilized soil volume, whether it is columns or mass-stabilisation, must be evaluated by indirect measurements of binder use, slurry flow, or similar.

The technical problems related to monitoring are more pronounced for the dry than for the wet method. The binder contained in a flow of slurry is easier to measure than binder contained in a compressed air stream. Therefore, the weight loss of the binder storage tank is usually used as a measure of binder used when the dry method is used, whereas direct measurement of flow is more common for the wet method.

For the wet method the quantity of each binder material needed for each batch is weighed as it is added to the measured water volume in the mixer. This process can be made easier with ready batched or pre weighed bagged materials.

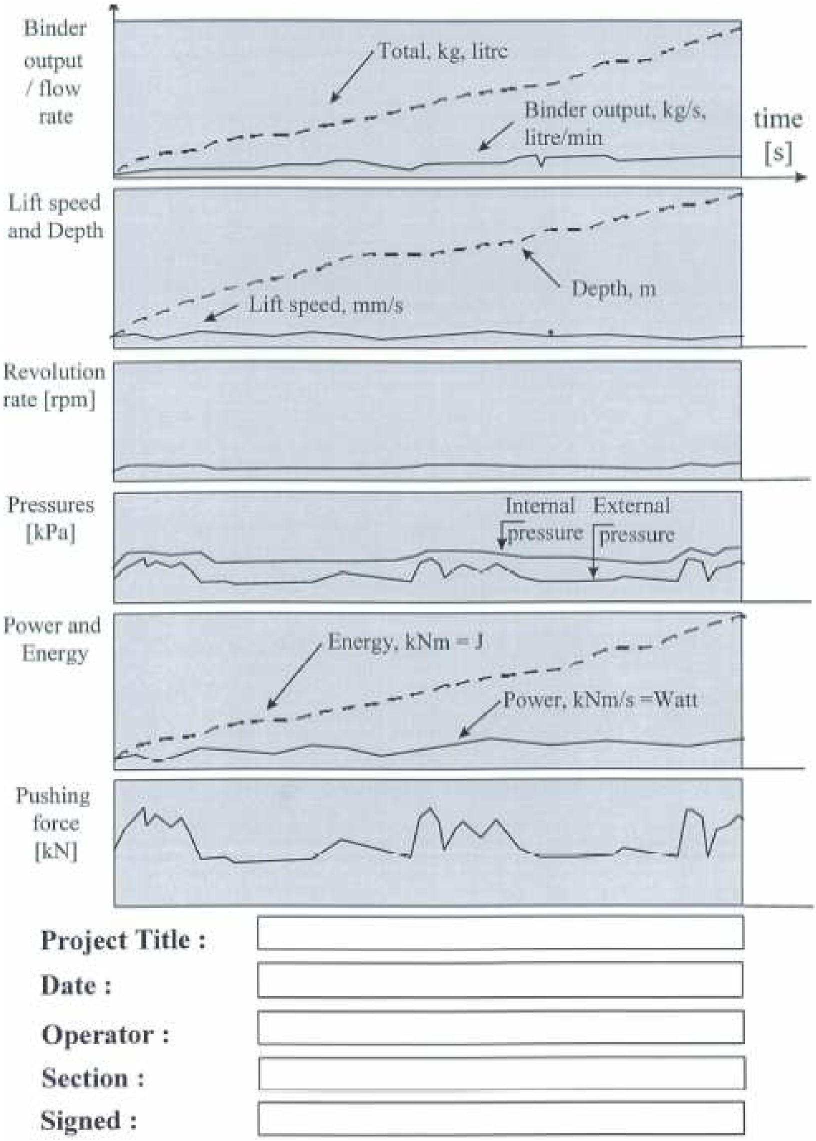


Figure 66 – Example documentation layout

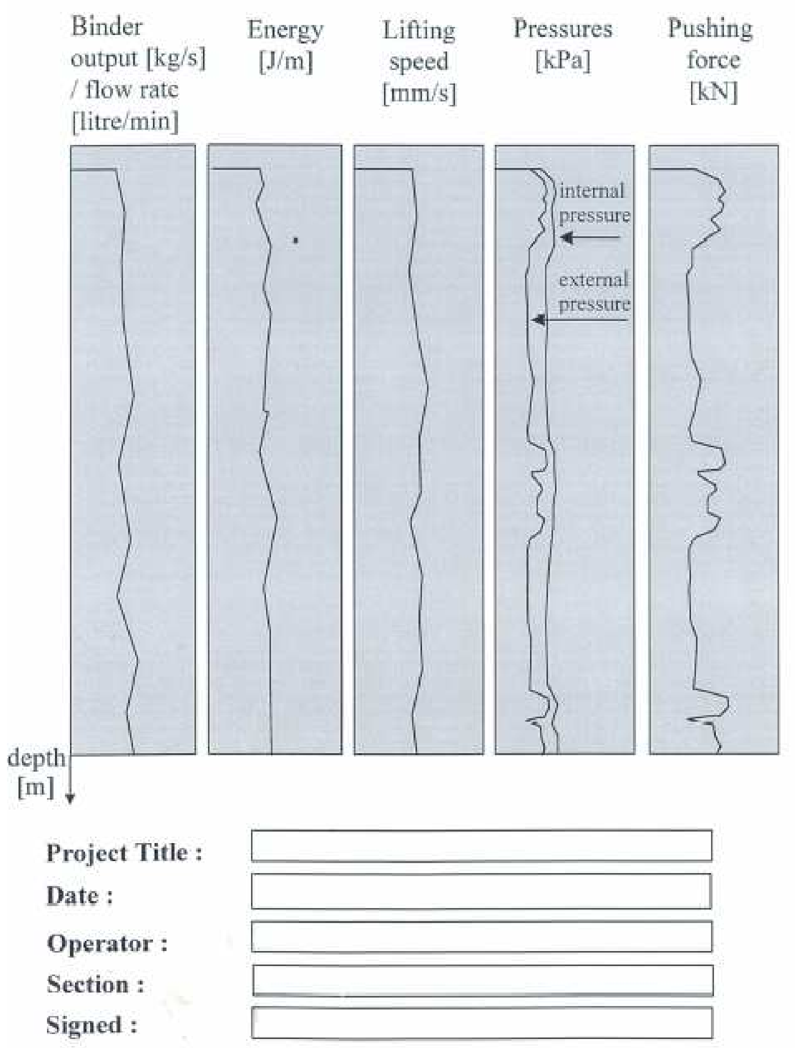


Figure 67 – Example documentation layout

Monitoring after stabilization

For the case of Ledsgard as check up for the design and verification of the stability as well as the serviceability of the embankment, a monitoring program was installed along three selected sections. A number of vertical and horizontal inclinometers as well as geophones had been installed for monitoring purpose. Furthermore, the settlements of the rails had also been measured at the selected sections. The long term monitoring has confirmed the stability and serviceability of the structure.
Instrumentation for monitoring the stabilization process carefully is very important since the stabilization process itself can generally not be submitted to direct inspection. The amount of binder injected in a certain soil volume, as well as the geometry and homogeneity of the stabilized soil volume, whether it is columns or mass-stabilisation, must be evaluated by indirect measurements of binder use, slurry flow, or similar.

Constraints linked to the railway environment

During the construction of columns, one has to consider the effect of the building of columns beneath or near the track. The upward or downward movements of the soils must be prevented.

The movements of tracks, the increase of pore pressure, the movements of surroundings embankments, the settlement should be recorded using the classical geotechnical measurement methods such as inclinometers, pore pressure transducers, force sensors, etc…

Some restrictions should also be given in terms of speed of trains during these special works in terms of speed of trains and of train traffic.

In addition, in situ tests (penetrometer or pressumeter tests) should be carried out; just after the building of columns and before the train traffic starts. They will aim at checking that the resistance of the liquid mixture is at least as strong as the previous subsoil.

Such in situ tests should have already been done on columns to check the feasibility of the proposed soil-mixing method.

5.3.5 Control of column quality

In situ tests

Field tests should be performed on trial columns in order to verify their homogeneity and quality. Typical tests are the Standard Column Penetration Test (SCPT) and Reversed Column Penetration Test (RCPT), see Figure 68 and Figure 69.

In the SCPT, a steel probe fitted with vanes is pressed down into the trial column. The vanes span the main part of the column diameter, thus giving a good estimate of the column strength. A pre-drilled hole is made in the centre of the column before the SCPT. The measured tip force (in kN) is plotted against depth (m). The homogeneity and shear strength of the column are then evaluated.

An alternative when installing long columns is to use the reversed column penetration test (RCPT). A preinstalled steel probe fitted with vanes is pulled through the finished column, from the bottom of the column to the surface. The measured force (in kN) is plotted against depth (m). The homogeneity and shear strength of the column can then be evaluated. Three of the twelve trial columns were submitted to a reversed sounding probe for later pull tests.

Because of the probe geometry (20 x 400 mm2 and pre-boring of a 55 mm diameter hole) the penetration resistance from CPT tests can be converted into an undrained shear strength. The evaluated undrained shear strength from the three SCPT tests and the two RCPT tests is presented in Table XXIV in the case of the Lesgård site (Holm et al., 2002).

As judged from CPT tests (Holm et al., 2002) and Table XXIV, the effect of the improvement is quite good. The undrained shear strength is in the order of 150 - 250 kPa two weeks after installation. (Holm et al., 2002) also show that the effect is as good in the gyttja as in the underlying clay.

Table XXIV – Results of field tests on Dry DMM columns (Holm et al., 2002)

Column No.	Method	Time [days]	Shear strength [kPa]
2	RCPT	5	(110 to 460)
3	SCPT	5	120 to 180
4	RCPT	5	(110 to 460)
5	SCPT	14	180 to 260
11	SCPT	14	150 to 200

Figure 68 – SCPT for quality control of dry Deep Mix Method columns (Holm et al., 2002)

Figure 69 – RCPT for quality control of dry Deep Mix Method columns (Holm et al., 2002)

A comparison of the shear strengths evaluated from the RCPT/SCPT field tests (Table XXIV) with the tests of the laboratory mixed samples is presented in Figure 70. Results show that the laboratory values are somewhat higher. Using "average" values, the field test shear strengths are in the order of two-thirds of the corresponding laboratory tests. In the design of countermeasures, the stiffness of the material is of prime interest and such data cannot be deduced from the field column penetration tests.

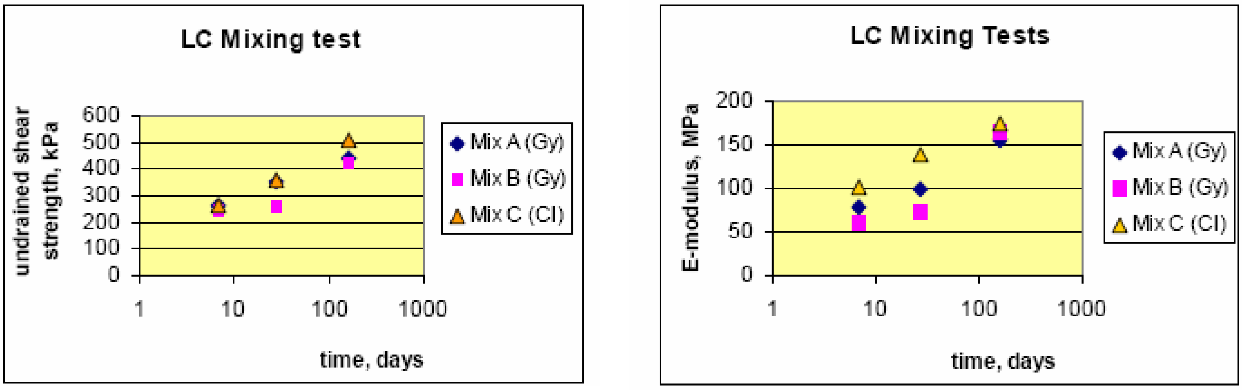


Figure 70 – Results of unconfined compression tests on lime/cement laboratory mixed samples of gyttja (Gy) and clay (Cl) (150 kg/m3 lime/cement in a proportion of 25/75) (Holm et al., 2002).

5.3.6 Time schedule and cost

As an example, we can consider the Ledsgård case (Sweden) (Holm et al., 2002).

The time schedule for the total works was approximately three months starting in May 2000. The deep mixing with lime-cement columns took two weeks (excluding test columns). Traffic was interrupted between July 9 and July 31, 2000 on the one track, where the track was removed and subsoil improvement performed. The total cost was 5.1 million SEK (about 0.56 million €). Figure 71 shows the cost distribution, which indicates that the cost for the dry DMM soil improvement itself (lime/cement columns) is only a minor part of the total cost.

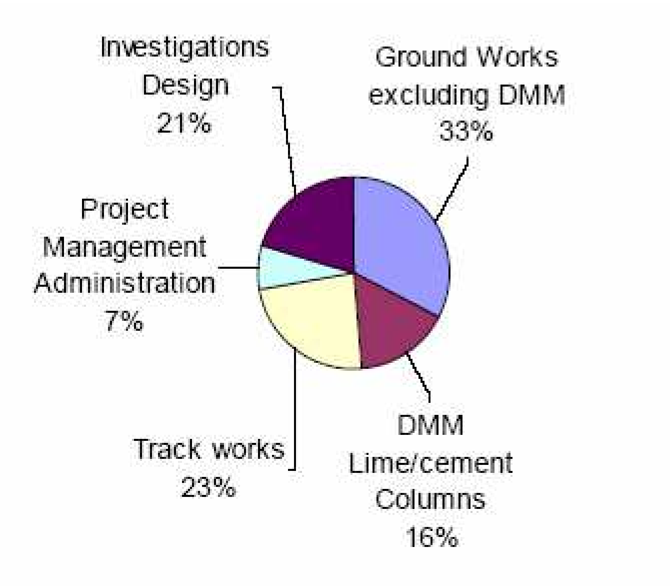


Figure 71 – Distribution of cost of countermeasures (Holm et al., 2002)

5.3.7 Risk assessment

It is essential therefore that the appropriate measures are taken to mitigate the risk to the safety and health of personnel. The risks can be listed and rated in a risk assessment for the site works. An example of a risk assessment is given in Table XXV and while this does not cover all risks it is intended as an illustration of the risk assessment process.

Table XXV – Risk assessment (after (EUROSOILSTAB, 2000))

Operation	Hazard	Who might be harmed?	Risk Factor	Is the risk adequately controlled?	What further action is necessary?
Describe the operation(s) being assessed.	List hazards here	List groups of people at risk from the hazards identified.	Calculate the Risk Factor.	List existing controls, or where the relevant information may be found.	List the risks that are not adequately controlled and the action you will take where it is reasonably practicable to do so. You are entitled to take cost into account, unless the risk is high. (note 5)
Pedestrian access	Debris falling, equipment, oil or other spills, uneven ground, trailing pipes	Users, other staff, contractors	1	Instruction, wear protestive clothing.	Cone off working area to restrict access to users only
Use of vehicles,	Collision with pedestrians	Users, other staff, contractors	1	Warning signs, Instruction	Cone off working area to restrict access to users only
Manual handling	Lifting, lowering, pulling, pushing	Users	1	Instruction, users must attend manual handling course, use mechanical assistance where necessary.	None.
Working with contractors crane	Collision with suspended equipment, falling debris, Worn, faulty or wrong lifting attachments.	Users, other staff, contractors	1	Instruction, users must attend manual handling and slinging course, wear protective clothing.	None
Mixing of binders	Inhalation of dust, lifting bags of materials, lowering bags of materials, opening bags of materials	Users, other staff, contractors	1	Instruction, Manual handling course, wear protective clothing, use of mechanical assistance where possible.	Work in well ventilated areas
Storage and transfer of materials	Inhalation of dust, lifting bags of materials, lowering bags of materials, opening bags of materials, Escaping high pressure gas and gas driven particles	Users, other staff, contractors	1	Instruction, Manual handling course, wear protective clothing, use of mechanical assistance where possible.	Work in well ventilated area, Clean up contingency in place.

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