Installation effects of statically compacted stone columns for a railway embankment on soft soil: results from a large-scale field test in Northern Sweden

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Abstract. Stone columns are a common ground improvement technique for soft soils, and a more sustainable binder free alternative to deep mixing. However, conventional vibro-displacement installation often causes significant lateral soil displacement and property alteration. This has limited the application of stone columns in sensitive soils, e.g. in Scandinavian silts and clays. To reduce soil disturbance, statically compacted stone columns are being explored as an alternative installation approach inspired by the Silent, Advanced Vibration-Erasing Compozer method developed in Japan. A full-scale field test is ongoing in northern Sweden to evaluate installation effects and long-term performance of statically compacted stone columns under a railway embankment on soft sensitive soil. Comprehensive in-situ and laboratory testing was conducted before and after column installation to evaluate changes in soil properties and assess the degree of disturbance caused by the method. The test site, set up in autumn 2024, was extensively instrumented to monitor pore pressures, lateral movements, settlements, and load transfer during installation and subsequent loading. Data collected during the construction and initial loading phases in autumn 2024 and spring 2025 provide valuable insight into soil structure interaction and column performance. This paper presents monitoring results pre, during and post installation of the stone columns, focusing on the immediate effect on the surrounding soft soil. Preliminary results indicate that the statically compacted stone columns caused limited soil disturbance and provide sufficient load transfer, demonstrating their potential as a sustainable and competitive foundation solution for embankments on soft Scandinavian soils.

1 Introduction

Embankment construction on soft soil deposits usually requires ground improvement to control settlements and ensure stability. Among the available ground improvement techniques, stone columns represent a widely used and cost-effective solution that is also a more sustainable alternative to deep mixing due to the absence of binder material.

However, the use of stone columns has been limited in Scandinavia due to the significant lateral soil displacement and property alteration that the conventional vibro-displacement installation causes in the surrounding soil, see e.g. [1]. For sensitive clays, there is risk for a decrease in undrained shear strength due to remoulding during the installation procedure [2]. The wide-spread existence of sensitive soils in Scandinavia and the uncertainties regarding how those soils would be disturbed by the conventional method, has led to the method being ruled out early in most projects.

With the aim to find an alternative installation approach that can be performed in sensitive soils, statically compacted stone columns are being explored within a full-scale field test. The constructed test embankment is equipped with monitoring instruments to enable the study of effects during column installation as well as the long-term performance of statically compacted stone columns under embankment loading in a soft sensitive soil.

2 Statically compacted stone columns

The tested installation method for static compaction of stone columns is inspired by the Silent, Advanced Vibration-Erasing Compozer method developed in Japan [3]. A rotary penetrating system is used to drive the probe into the ground without the use of vibration or ramming. Similarly, the compaction steps are performed 'statically', i.e. without application of dynamic forces like vibrations or ramming.

The column installation includes four steps:

1. The hollow probe is filled with filling aggregate.
2. The probe is driven down to design depth.
3. From bottom up, the filling aggregate is discharged in intervals while the probe is raised.
4. By re-driving the rotating probe, the discharged filling aggregate is statically compacted.

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The discharging and re-driving (step 3 and 4) are repeated while the probe is lifted. Using this wave pattern, the column is gradually constructed from the bottom up. During compaction (step 4), the column diameter is enlarged to the nominal column diameter. The main difference to common vibro-displacement stone columns is that steps 2 and 4 involve no vibration of the probe when driving the probe into the soil profile and compacting the filling aggregate, respectively. Thus, the filling aggregate is expected to have less degree of compaction than that of a vibro-displacement column, but with less disturbance of the adjacent soil. A crucial element of the test site project is to evaluate if sufficient column stiffness can be reached.

3 Site description

The test embankment was constructed in the Degermyran area in the municipality Robertsfors in northern Sweden. The location is within the stretch of the planned Norrbotniabanan, a railway construction project connecting the cities of Umeå and Luleå with a new high-speed coastal railway. The Degermyran area is mainly a peat bog overlaying soft compressible sulphide bearing soils, which are common in the coastal areas around the Gulf of Bothnia. The site was chosen to study the applicability of statically compacted stone columns in soft sensitive soils. If applicable, the method will be used in the full production of the planned Norrbotniabanan railway project.

3.1 Soil profile and material properties

The soil profile based on geotechnical survey is shown in Fig. 1 and consists of five main soil layers. The top layer of 2 m peat is followed by an upper soft layer (Soft Soil I) of approx. 1.3 m thickness that consists of silty gyttja (nekron mud) and clay to silty clay. The upper soft layer is underlain by a thin layer of 0.7 m fine granular soil (silty sand or sandy silt) which in turn is underlain by the lower layer of soft soil (Soft Soil II) which consists of cohesive sulphide bearing soils such as clay and silt. The layer has a thickness of approx. 3.2 m. Below the soft soil, a 9 m thick layer of granular soils and silty moraine with varying contents of sand, silt and gravel rests above the bedrock. The area is wet, with the groundwater table appearing to be at or 0.5 m below the ground surface.

The properties of the soil were investigated in situ by piezocone dissipation tests and field vane tests. Soil sampling was performed at different depths. Disturbed samples were extracted using solid stem flight auger drilling. Undisturbed samples were taken with a Swedish standard piston sampler of type 2, St2. Strength and compressibility characteristics of the soft soil were determined by means of oedometer tests with incremental loading (ISO 17892-5:2017), direct simple shear tests, and fall-cone tests. The obtained values for the undrained shear strength, shown in Table 1, have been corrected for the presence of sulphide soil according to Westerberg [4] and are classified as extremely low to very low according to Eurocode EN ISO 14688-2:2004.

Sensitivity of the soft soil was obtained from field vane tests and fall-cone tests performed on undisturbed and remoulded soil. The present soil in the upper soft soil layer may be classified as low to medium sensitive while the soil in the lower soft soil layer may be classified as medium to highly sensitive according to Eurocode EN ISO 14688-2:2004.

The main soil parameter values obtained by in-situ and laboratory tests are summarized in Table 1.

Table 1. Soil parameter values obtained by in-situ and laboratory tests.

Parameter	Layer 2 Soft Soil I	Layer 3 Fine granular soil	Layer 4 Soft Soil II
ρ [t/m^3]	1.44–1.48	1.95	1.61–1.73
w_N [%]	69–93	10–28	41–64
w_L [%]	80–113	27	37–60
s_u [kPa]	1–21	-	8–17
S_t [-]	5–22	-	5–46
c_v [10^{-8} m^2/s]	2.8–13.0	-	2.7–9.0

Note: ρ, bulk density; w_N, natural water content; w_L, liquid limit; s_u, undrained shear strength; S_t, sensitivity; c_v, consolidation coefficient

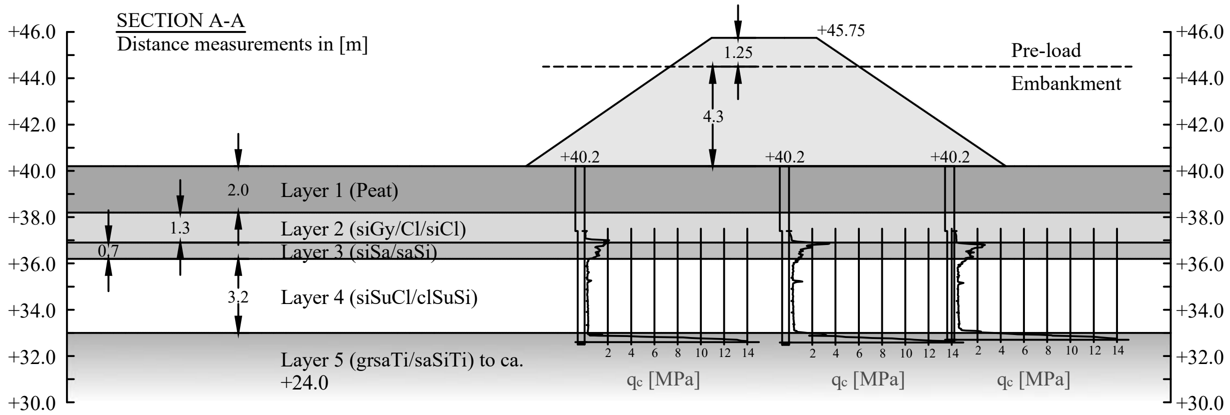

Fig. 1. Soil stratigraphy at Degermyran. The presented section, Section A–A, is located along the centerline of the embankment and oriented northwest–southeast. Measured values of cone tip resistance, q_c, from single cone penetration tests are shown.

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3.2 Ground treatment for embankment

The embankment in the Degermyran railway stretch has a planned final height of up to 4.3 m. Due to the presence of organic soils and deep soft soils, ground treatment was necessary to ensure that the maximum allowable settlements of 200 mm are not exceeded and that the overall stability of the embankment is guaranteed.

The superficial peat layer of approx. 2.0 m was excavated and replaced with granular material which functioned as a working platform for the construction machinery. Well-graded crushed material was used with a fraction size of 0–90 mm and fines content below 5%. A geotextile of class N4 and a polypropilen geogrid with aperture size 20 mm x 20 mm were placed in the middle of the layer. The geosynthetic-reinforced working platform also functions as a load-distribution layer, transferring the vertical load onto the columns.

The remaining soil, i.e. the soil below the excavated peat and above the load-bearing strata, was improved with stone columns.

3.3 Stone column installation

The equipment used to construct the stone columns for the test embankment was a prototype. A Bauer BG 23 H drilling rig was modified to enable the static installation method. The probe had an inner diameter of 0.4 m. The bottom part of the probe was fitted with a hollow stem auger drill bit to drive the casing down into the soil profile under rotation.

To avoid soil material entering the probe while it is being driven down, a back air pressure of 5 bar was applied. The air pressure was kept constant throughout the whole installation process.

The columns had a nominal diameter of 0.8 m and were implemented with a centre-to-centre spacing of 1.8 m in a square pattern, corresponding to a column-to-soil ratio of 0.155. The column material, i.e. filling aggregate, was crushed stone with a fraction size of 16–32 mm. The average column length was 7.65 m, as the columns were installed from the upper part of the lower load-bearing strata (Layer 5) to the top of the working platform.

In total 185 columns were installed, nine of these were constructed outside the embankment to calibrate and verify installation parameters. The installation of the 176 production columns was first performed in the northern half, and then in the southern half, indicated by the dotted line in Fig. 2. Within each half, the installation was performed in two cycles. During the first cycle approx. every second column was installed, and in the second cycle the remaining columns were installed. This installation sequence was adopted to minimise the accumulation of excess pore water pressure in the soft soil layers.

The installation of one production column was on average performed within 26 minutes, of which 3 minutes were drilling down (step 2), and 23 minutes were discharging and redriving (step 3 and 4). Moving the equipment to the subsequent column and preparing for installation took on average 3 minutes.

3.4 Embankment construction

A 0.5 m thick granular layer was placed at the base of the embankment, directly on top of the working platform. This layer consisted of crushed material with particle size range 0–32 mm. The subsequent embankment layers were constructed using crushed rock meeting the same material requirements as the working platform. The unit weight of the fill material was assumed to be 19 kN/m^3.

The embankment construction was performed in several layers up to 4.3 m designed height, considering the consolidation interval between the stages. To reduce settlements of the final structure, a pre-load of an additional 1.25 m thick layer of the crushed material was placed at the top of the embankment. The final height of the embankment, including the pre-load, was 5.55 m, corresponding to a total vertical stress of 105 kPa. However, the present paper focuses solely on the first two loading stages, resulting in an embankment height of 1.9 m and an applied total vertical stress of 36 kPa. This loading level is sufficient to demonstrate that the installed stone columns perform as intended.

3.5 Instrumentation

The embankment and the underlying soil were instrumented to measure short- and long-term effects of the stone column installation on the in-situ soil. Additional instruments were installed to evaluate load distribution between the columns and the surrounding soil, as well as consolidation and settlements under the embankment and surcharge load.

The location of the instruments, relevant for the scope of this paper, is shown in Fig. 2. The instrumentation for measuring the development of pore pressure, settlements and load transfer is mainly concentrated in the centre of the embankment where the maximum vertical stress is applied. Lateral displacement measurements are performed at the sides of the embankment. Thus, both displacement during the installation and consolidation can be measured as well as indications for slope failure. Load

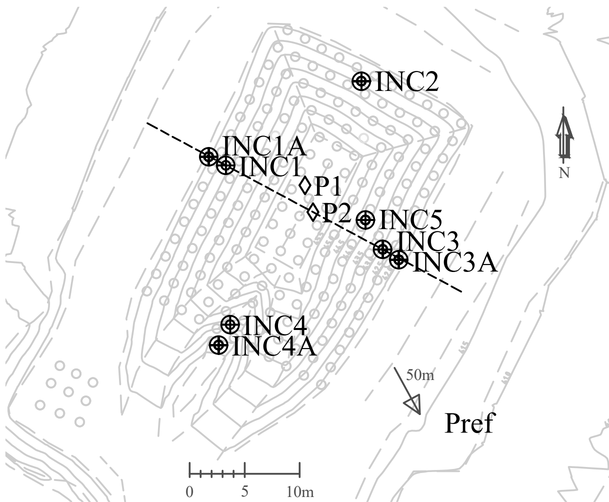

Fig. 2. Plan view of monitoring instrumentation. Instrument locations are indicated in black. Installed calibration and production columns and embankment shown in grey for reference.

cells and instrumentation for measuring vertical displacements have been installed after the stone column construction. Sensors for measuring pore pressure and lateral displacements were partly installed prior to column installation and partly after. However, some of the sensors that were installed prior column construction were severely damaged during installation and had to be replaced. Details of the instrumentation relevant for this study are presented below.

Fully-grouted vibrating wire multi-level piezometers were installed to measure pore pressure in three different locations. Two are located within the embankment, see P1 and P2 in Fig. 2, and one, Pref, is located 50 m southeast of the centre of the embankment and functions as a reference. Sensors were placed at three different levels: +37.0, +35.5, and +34.0 m. The upper one is located in the Soft Soil I layer while the two lower ones are located in the Soft Soil II layer.

Inclinometers were installed in five different locations, see INC1 to INC 5 in Fig. 2. INC1 and INC4 were initially monitored manually, with measurements taken approximately once per day at 0.5 m intervals during column installation. However, both tubes were damaged during installation and replaced with automatic systems post column installation (INC1A and INC4A, respectively). The automatic inclinometers have four to five biaxial sensors, providing displacement measurements at five to six depth levels. Two types of automatic inclinometers were used: INC3 was comprised of probes connected by a flexible steel wire, allowing relative movement between individual probes, whereas the remaining inclinometers employed a rigid point-to-point connection in which adjacent probes were mechanically linked, forming a continuous assembly. INC2 remained intact during the whole column installation process and the following embankment construction. INC3 survived the installation of three of the four closest columns and valuable measurements could be taken. However, it was damaged during the installation of the fourth column in its direct proximity and replaced post column installation (INC3A). INC5 was installed after column installation but prior to the embankment construction.

4 Monitoring results and analysis

The installation of the 176 production columns was performed within 12 days. The start of the column installation is marked as t = 0 d. There was no production on day 3 and 10. Embankment construction commenced five weeks later, i.e. at t = 46 d, when two layers of 0.8 m and 1.1 m, respectively, were placed and packed within a duration of 14 days, resulting in a total embankment height of 1.9 m at t = 60 d. The shown monitoring results also include the subsequent consolidation period of 20 days, i.e. t = 80 d.

4.1 Pore water pressure

Piezometer measurements commenced two days prior the installation of the first columns, i.e. at t = −2 d. Measuring continued throughout column installation and subsequent embankment loading. The measurements were recorded continuously at P1 and Pref for the entire period. However, no measurements were obtained at P2 during column installation; monitoring at this location commenced two days after completion of the column installation, i.e. at t = 14 d. Fig. 3 shows the whole measuring period for all three locations and all levels. Pore pressures measured at relevant milestones are shown in Table 2 for all three locations. The pore water pressures were recorded every 15 minutes. Note, that the pore pressures for P1 at the start of the measuring period are clearly affected by the construction of the working platform which was performed shortly before column installation started. The generated excess pore pressures were not left to dissipate fully before column installation started.

4.1.1 During column installation

The generation of excess pore water pressure in location P1 at three different depths during the installation of the stone columns is shown in greater detail in Fig. 4. The figure also shows the time of installation of the single columns and their respective centre-to-centre distance to the piezometer location P1.

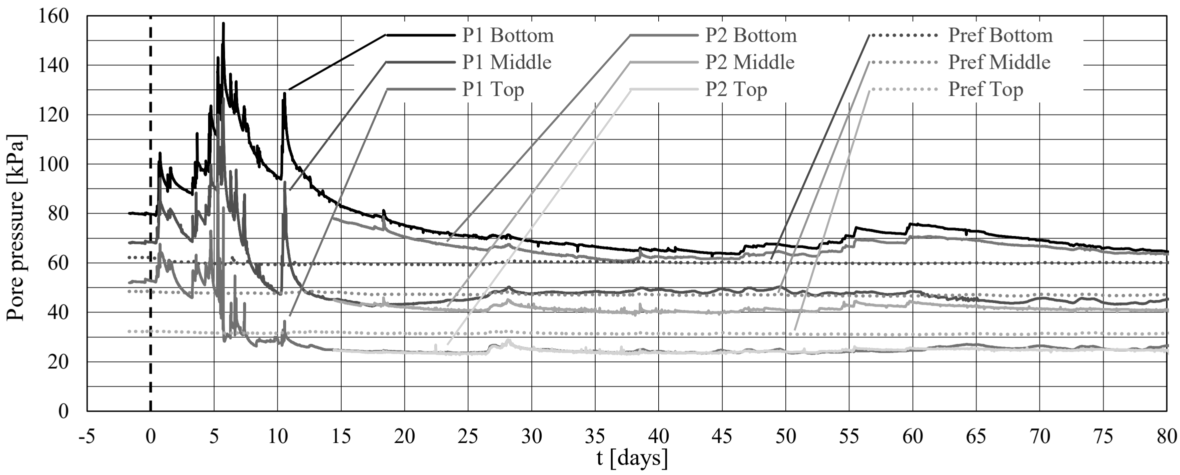

Fig. 3. Pore pressure measurements before and during column installation and subsequent embankment loading.

Table 2. Measured pore pressures [kPa].

t =		0 d	12 d	46 d	60 d	80 d
P1	Top	52.8	26.3	23.7	24.6	26.5
	Middle	68.4	49.3	48.2	48.4	45.2
	Bottom	79.8	96.6	63.7	75.6	64.6
P2	Top	-	-	23.7	25.4	24.8
	Middle	-	-	40.1	44.2	40.9
	Bottom	-	-	61.7	70.5	63.5
Pref	Top	32.5	32.0	31.4	31.1	31.5
	Middle	48.3	48.1	47.0	46.6	47.2
	Bottom	62.1	59.3	60.1	59.7	60.1

A clear relation between the proximity of the installation and an increase in pore pressure can be seen in Fig. 4 for all three sensor levels. The closer to the sensor the column was installed, the greater the increase in measured pore pressure. Further, it was observed that the column installation generally leads to higher pore pressure increase in the top sensor, where build-ups of up to 45 kPa were observed during the installation of a single column. However, dissipation is also significantly faster for the top sensor which can be seen in the decrease of pore pressure after each installation. This can be attributed to the existence of the working platform and the sandy layer that allowed for significant dissipation of the generated excess pore pressure in the Soft Soil I layer, before the next column was installed. Thus, there was less excess pore pressure accumulation in the Soft Soil I layer compared to in the Soft Soil II layer, which has a greater thickness and thus longer drainage paths.

Fig. 5 shows the measured pore pressure during day 5 and 6 in higher data resolution. The corresponding production time for a single column of 3 min for drilling down (step 2) and 23 min for retracting and compaction (steps 3 and 4, respectively) is highlighted for two example columns in grey. The evaluation of pore pressure changes during the installation of a single column indicates that the main generation of pore pressure occurred during step 2 (drilling) rather than step 3 and 4. Furthermore, the measured accumulation of pore pressures lessened with every time that P1 was passed by the construction works. Comparing the direct response of pore pressure due to the installation of the closest columns, it can be observed that the increase in pore pressure is similar for all passings or even slightly higher for the second or third passing. However, the dissipation in between installations is significantly faster for the subsequent passings than for the first, see arrows in Fig. 5. Thus, it is clear that the stone columns were acting as vertical drains, with no evidence of the soft soils clogging the filling aggregate within the timeframe of construction.

With respect to the influence radius of column installation, an apparent boundary is observed at a centre-to-centre distance of approx. 10 m, regardless of the number of passings. During the first passing, despite the absence of previously installed columns that could impede pore pressure generation, only negligible pore pressure changes of max. 4 kPa are observed in response to column installations at centre-to-centre distances of 10 m or greater. During subsequent installations near P1, no measurable pore pressure response was detected at these large distances. Significant pore pressure responses of above 10 kPa were only observed for column installations at centre-to-centre distances of approx. 6.4 m, i.e. 8 times the nominal column diameter, or less.

4.1.2 During embankment construction

Embankment construction commenced at t = 46 d and continued for a duration of 14 days. The measured pore pressure responses at P1 and P2 are shown in Fig. 6, along with the corresponding embankment height for reference.

The pore pressure evolution recorded at the bottom sensors of P1 and P2 are highly comparable. Although the magnitudes differ by approx. 2–5 kPa, their overall trends are very similar. In general, the magnitude of the excess pore water pressure, generated from the placement of each embankment layer, increased with depth, with the largest magnitude observed at the bottom sensor and the smallest at the top sensor. This indicates that the embankment load is transferred to the bottom of the stone column, meaning that the stone columns exhibit load-carrying behaviour.

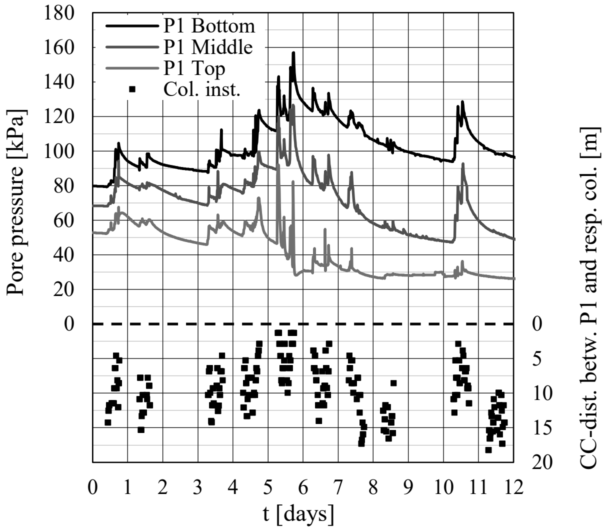

Fig. 4. Pore pressure at P1 during column installation. Installation times and center-to-center distances to the respective columns are shown for reference.

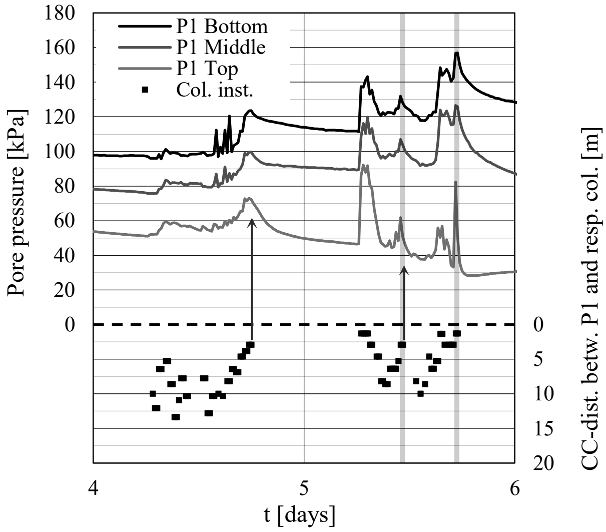

Fig. 5. Detailed view of pore pressure response at P1 during the installation of single columns. The production times of two example columns are indicated in grey.

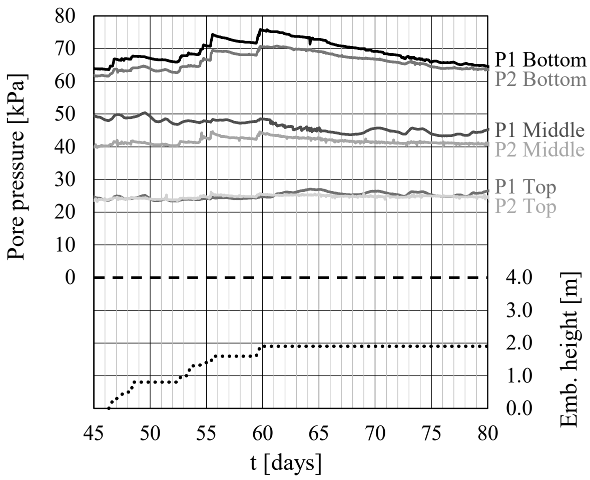

Fig. 6. Pore pressure response at P1 and P2 during early embankment loading. Embankment height shown for reference.

Besides the load-carrying capacity, the columns as well as the working platform have a significant drainage effect on the surrounding soil. The excess pore pressure that is generated by the embankment loading dissipates fast after each loading step in both P1 and P2. That indicates that the stone columns function similarly to vertical drains but with higher drainage capacity. The pore pressure values obtained at t = 80 d in comparison to those obtained at t = 0 d suggest that there has been a significant alteration of the natural ground water table due to the construction works. The pore pressures measured in P1 and P2 at t = 80 d are lower for the top two sensors than the pore pressures measured in Pref at t = 0 and 80 d. Considering the trend between t = 60 and 80 d, the pore pressure in the bottom sensors is projected to be lower than Pref for both P1 and P2 if the soil is left to consolidate further. This indicates that there is a drainage effect from the columns, lowering the natural groundwater level.

However, there are indications for vertical sensor movement. The sensors in P1 may have moved upwards during column installation, while sensors in both P1 and P2 may have moved downwards under embankment loading. As their exact positions were not monitored, only rough estimates of relative movement can be made.

4.2 Lateral displacements

Relevant for the scope of this paper are the lateral displacement measurements of inclinometers INC2 and INC3. The former remained intact throughout the whole column installation period, the latter failed at t = 6 d. Manual measurements from INC1 and INC4 did not capture the effects of individual column installations and are therefore excluded from the detailed results, though included in the overall evaluation.

Automatic measurements in INC2 and INC3 were recorded at 15-minute intervals, corresponding to approx. two to three readings between successive column installations. The sensors provide displacement measurements on five levels, respectively: +32.2, +33.7, +35.7, +37.7, and +39.7 m for INC2 and +32.1, +33.7, +35.7, +37.7, and +39.6 m for INC3. For both inclinometers, the bottom-most measurement level is anchored in the underlying moraine layer, where minimal displacement is expected. Thus, zero displacement at the base was assumed for inclinometer data interpretation. The top-most level is located within the relatively firm working platform. The analysis therefore focuses on the three middle levels. The upper of these is situated in the Soft Soil I layer, while the remaining two are situated in the Soft Soil II layer.

Relative lateral displacements induced by individual column installations are presented with respect to centre-to-centre distance between the column and the respective inclinometer. Due to the differing properties of the two soft soil layers, results are evaluated separately for each layer, see Fig. 7 for Soft Soil I and Fig. 8 for Soft Soil II. For both layers, the magnitude of displacement increased with decreasing distance to the installed column. Relative lateral displacements in the Soft Soil I layer measured in INC3 were consistently larger than those measured in INC2. This is attributed to the previously described difference in inclinometer design of the two. Due to long probes and flexible linkage of the probes of INC3, there is most likely a significant relative displacement between the bottom of the probe in the stiff working platform and the top of the probe below it, causing an inflated evaluated lateral displacement in Soft Soil I in INC3.

Comparing the results in Fig. 7 and Fig. 8, larger relative displacements were observed in the Soft Soil II layer than in the Soft Soil I layer for the automatic

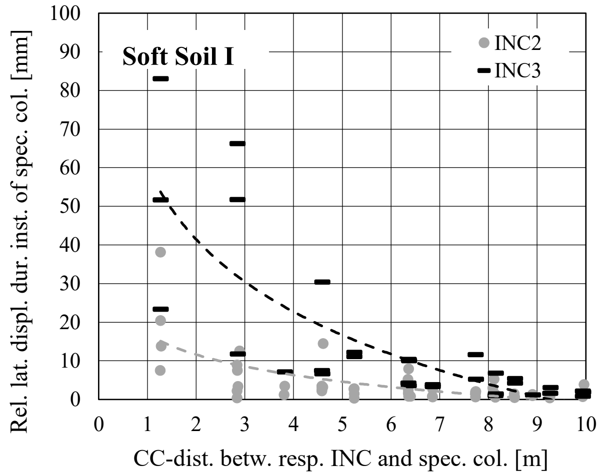

Fig. 7. Relative lateral displacements in Soft Soil I layer induced by individual column installations.

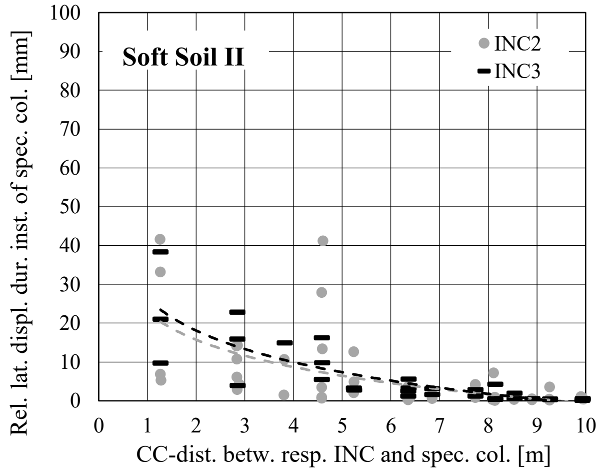

Fig. 8. Relative lateral displacements in Soft Soil II layer induced by individual column installations.

measurements from INC2, a trend that is also confirmed by the manual measurements from INC1 and INC4 (lending further support to the hypothesis that the INC3 results in Soft Soil I are inflated). The difference is attributed to lower peak values of undrained shear strength and higher values of sensitivity in the Soft Soil II layer, as well as an overall greater thickness of the layer. Besides that, additional lateral displacement may occur from the sequential discharging and compaction. As the column is constructed from the bottom up, the lower part of the column undergoes greater cumulative compaction.

In agreement with the pore pressure measurements, the lateral displacement response indicates a limited radius of influence of almost 7 m. Lateral displacements exceeding 10 mm occurred almost exclusively within a centre-to-centre distance of 6 m from the installed column, i.e. 7.5 times the nominal column diameter.

The influence of installation sequence was evaluated by analysing the lateral displacements from the installation of each column within 6 m radius of the inclinometers. Fig. 9a and Fig. 9b show the relative lateral displacements associated with individual column installations for INC2 and INC3, respectively, ordered according to installation sequence. Each column is represented by two values of relative displacement, corresponding to the Soft Soil I layer and the greater of the two measured values in the Soft Soil II layer. Columns were installed within 6 m distance of INC2 at t = 0, 3 and 4 d, and of INC3 at t = 4, 5 and 6 d. Monitoring of INC3 was interrupted following installation of Column SP088 at t = 6 d due to equipment failure. With progressing installation, a clear stiffening effect is observed. That can be seen by comparing the relative displacements caused by the columns installed first, listed high in Fig. 9a and 9b, and those installed later, listed low. This trend indicates a progressive increase in lateral stiffness of the improved subgrade as the network of columns develops.

5 Discussion

To evaluate the installation effects and performance of the tested method of statically compacted stone columns at the Degermyran test site, a comparison should be made against that of similar test sites where the common vibro-displacement stone column method has been applied. The results from the test site reported by Castro and Sagaseta [5] is used herein as the reference site for comparison, as it had similar stone column geometries and soil profile.

In their field test, the stone columns were 9 m deep, with an average diameter of 0.8 m and a spacing of 2.8 m in a triangular pattern, corresponding to a column-to-soil ratio of 0.074, compared to 0.155 at the Degermyran test site. The soil stratigraphy similarly consisted of an upper soft clay layer, underlain by a granular layer, which in turn was underlain by a deeper clay layer. Consolidation coefficients were of the same order of magnitude (10^{-3}–10^{-4} cm²/s), whereas undrained shear strength values were significantly higher at the reference site.

The measured pore pressure response shown herein is comparable to that observed by Castro and Sagaseta [5]. The clear correlation shown at Degermyran between the proximity of column installation and the magnitude of excess pore pressure generation is consistent with their observations.

Pore pressure measurements in the present study were recorded at 15-minute intervals, compared to 10-second intervals used for the reference site, and peak pore pressure values may therefore not have been captured for all column installations at Degermyran. Nevertheless, in the case of column installations well timed with the data recording, the generated excess pore pressures were generally lower than those reported by Castro and Sagaseta [5], despite the lower column-to-soil ratio. Maximum excess pore pressures of up to 45 kPa were observed during installation of a single column, compared with peak values of up to 100 kPa at the reference site. The difference is primarily attributed to the differing installation methods, suggesting that the installation method used in this field test is less violent than the conventional vibro-displacement method. The difference is secondarily attributed to the higher undrained shear strength at the reference site. According to the undrained cylindrical cavity expansion theory, the total stress increase required for cavity expansion—and thus the generated excess pore pressure—increases with undrained shear strength assuming elastic–perfectly plastic clay and the Tresca yield criterion [6].

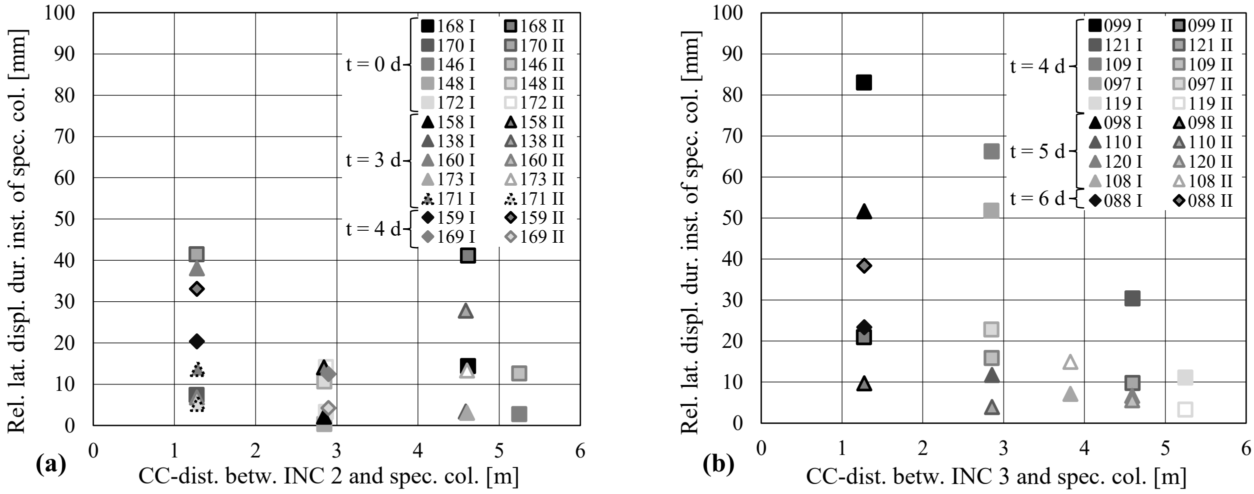

Fig. 9. Relative lateral displacements from individual column installations. (a) INC2; (b) INC3. Legend entries are ordered by installation time (earliest at top). "I" and "II" denote displacement in the Soft Soil I and Soft Soil II layers, respectively.

Furthermore, for both the reference study and the current study, the observation was made that the peak excess pore pressure occurs when the probe passes the level of the sensor during the penetration phase (step 2). The measurements presented by Castro and Sagaseta [5] indicate two periods with significant pore pressure increases during column installation. The first increase is due to mass displacement when the probe is driven down (step 2). In the second period, pore pressure increases due to the radial expansion of the column during compaction on the way up (step 3 and 4). Though the measurement intervals are not as frequent in the present study as it was at the reference site, the pore pressure measurements suggest that significant excess pore pressure was generated during step 2, but was generally not followed by noticeable subsequential spikes during step 3 and 4. This further supports the hypothesis that the current installation method is less violent than the conventional vibro-displacement method as subsequent spikes are almost non-existent. However, this needs to be verified by measuring pore pressure with a higher frequency.

Excess pore pressures dissipated more rapidly in the Soft Soil I layer than the Soft Soil II layer, which is attributed to the presence of the working platform. The coarse-grained material of the platform provides high permeability and acts as an efficient drainage layer. A similar drainage effect is observed in the vicinity of previously installed stone columns. Owing to their significantly higher permeability of the filling aggregate compared with the surrounding soft soil, the columns act as vertical drains. Fast dissipation of excess pore pressures in the vicinity of stone columns has been observed in many field tests, see e.g. [1, 5].

It was further observed, across successive installation passes, that the magnitude of the generated excess pore pressure remained the same or increased slightly, indicating that the undrained shear strength was maintained or modestly enhanced following consolidation after previous installations. This behaviour is consistent with increased horizontal stresses from the mass displacement of the column installation and suggests that, despite the high sensitivity of the soil, no degradation of the undrained shear strength occurred. However, this hypothesis needs to be tested through further analysis of the performed geotechnical investigations.

Inclinometer measurements indicate a progressive stiffening of the surrounding soil as column installation proceeded. This is evidenced by decreasing lateral displacement with each column installation within close proximity, suggesting an increase in the composite stiffness of the improved ground. Notably, although the presence of installed columns significantly accelerates pore pressure dissipation and reduces installation-induced displacements, the peak magnitude of excess pore pressure generated during column installation remains largely unaffected which indicates that the undrained shear strength is not reduced and that liquefaction does not occur even though the soil has a high sensitivity. During subsequent embankment loading, the largest pore pressure increases are observed at large depth, indicating load transfer toward the column toe.

6 Conclusions

This paper presented the behaviour and monitoring results for a test embankment on soft soil improved by statically compacted stone columns. Pore pressure development and lateral displacements were analysed to evaluate the immediate effect of installation on the surrounding soil, as well as the performance of the ground improvement technique during the initial stages of embankment loading. The main conclusions are as follows:

- The results suggest a radius of the zone of influence of 8 times the nominal column diameter, as insignificant increases in pore water pressures and lateral displacements were measured beyond that distance.
- Previously installed neighbouring columns act as vertical drains, accelerating the dissipation of excess pore pressure from additional column installation.
- Previously installed neighbouring columns also significantly reduce the lateral displacement of the adjacent soil during additional column installation.
- There is no indication of a decrease in undrained shear strength through the installation despite the high sensitivity of the soft soil layers.
- Load bearing behaviour of the columns is evident.

Overall, the installation method does not appear to induce excessive soil disturbance. Instead, the statically compacted stone columns effectively enhance ground stiffness and drainage capacity as well as provide load transfer to the lower load bearing strata. Thus, fulfilling their intended function as a ground improvement measure.

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