Vacuum preloading for a truck parking area in Luhtaanmäki

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ABSTRACT
City of Vantaa considers constructing a truck parking area and a filling station on a soft clay deposit in Luhtaanmäki, Vantaa (in Finland). The aim of this study was to evaluate whether vacuum preloading with vertical drains is a feasible ground improvement method for the site. Besides in situ –tests and classification tests, overall 18 oedometer tests were carried out. Preliminary design of vertical drainage was based on Hansbo's solution, and smear effect was taken into account. Time-settlement -calculations were carried out using tangent modulus method and so called equivalent vertical permeability. In calculations, the following loading history is assumed: First, a surcharge fill of 30 kPa is constructed. After that, vacuum of 70 kPa is applied for 8 months. In order to compensate for the settlement caused by vacuum preloading, embankment load of 10 kPa is later applied. Total settlement is 825 mm, and that considered the final embankment height is 1.2 m. After the treatment, the minimum degree of overconsolidation is 220%. According to the results, vacuum preloading suits well for Luhtaanmäki.

Keywords: ground improvement, vacuum consolidation, prefabricated vertical drains, clay, surcharge fill

1 INTRODUCTION

In this paper, soft soil improvement via vertical drains with vacuum preloading is studied. The soft clay deposit in question is located in Vantaa, Finland. City of Vantaa considers constructing a truck parking area and a filling station on this site. Design of vertical drainage and settlement predictions are carried out in order to evaluate whether vacuum preloading is a feasible soil improvement method for the truck parking area. The goal is to acquire a degree of over-consolidation over 120 % after 7-8 months of vacuum preloading. The desirable final embankment height is 1.2 meters.

Vacuum preloading was first proposed by Kjellman (1948) for cardboard wick drains. In this ground improvement method, the site is covered with an air-tight membrane, and suction is created underneath via prefabricated vertical drains (PVDs) by using a vacuum pump. In general, the benefits of vacuum preloading are: (i) faster consolidation (also compared to conventional PVD improvement), (ii) low shear strength of soil will not prevent heavy preloading, (iii) less fill material is needed, (iv) there is no need for heavy machinery, (v) and as heavy preloading is possible, settlements due to creep after construction can be reduced (Brandt et al., 2015; Chaumeny, 2011; Ganesh Kumar et al., 2015; Indraratna et al., 2010; Saowapakpiboon et al., 2010; Voottipruex et al., 2014).

Despite its benefits, vacuum preloading is relatively unknown method in Finland. It was first used in winter 1974–1975 at a soft clay deposit test site Itäkeskus, Helsinki. However, sufficient suction pressure was not reached, and the soil was later improved using traditional preloading (Puumalainen, 1996). Leminen and Rathmayer (1979) presented that the reason for insufficient suction was water flowing out of silt layers or from coarser soil under the soft soil deposit. In vacuum preloading, pumped water should be pore water from clay layers, not freely seeping water. Similarly, Ganesh Kumar et al. (2015) observed that in a vacuum preloading site in India, the water pumped out was partly from surrounding areas (the sea shore) through thin sand seams. According to Indraratna et al. (2010), cut-off wall is useful at sites that need to be sealed.

In order to avoid these kind of leakage problems, PVDs should not pass sand or silt layers at all. In addition, the distance between PVDs and coarser layers below clay deposit should be at least 1 m. However, the accurate depth and location of coarser soil layers is rarely known. The risk of leakage-related insufficient suction can be decreased by using a membraneless system (Indraratna et al., 2010): The vacuum channel is connected directly to each individual PVD by tubes. As each PVD acts independently, single leak will not affect the whole drainage system. However, according to Seah (2006), the extensive tubing in membraneless system can prolong installation time and add costs.

Despite the fact that the vacuum preloading did not work as expected in Itäkeskus, in another test site in Torpparinmäki, Helsinki it was successful. According to Puumalainen (1996), vacuum preloading is time saving, efficient and economically viable ground improvement method also in Helsinki region.

2 GEOTECHNICAL CONDITIONS OF THE SITE

At the site in Luhtaanmäki, the soft clay deposit has a varying thickness of 5 to 13 meters. In this paper, only one point (point 36) is studied. This point is approximately in the middle of the planned truck parking area. At studied point the clay deposit reaches a depth of 8 meters. Dry crust is 1.5 m thick. Under the clay deposit there is a 4-5 m thick layer of sandy silt, and the deepest soil layer is moraine. The groundwater is pressurized and above the confined aquifer there is perched water table at 1.5 m. Due to pressurized groundwater, conventional PVD improve-ment (without vacuum preloading) would not be efficient.

Both laboratory tests and *in situ* tests were carried out near the studied point. Two kinds of sampling tubes were used: city of Vantaa used tube called ST2 (diameter of 50 mm) and Aalto University used bigger tube TKK-86 (diameter of 86 mm).

Besides several classification tests, overall 18 incremental loading (IL) oedometer tests were carried out. Five of these tests were horizontal IL oedometer tests and one was radial consolidation test. Only samples from TKK-86 tubes were used. As vertical drains reach the clay deposit only, test results below 8 m are not included in the graphs.

Soil layers used in calculations, unit weight γ, water content w and liquid limit w_L are represented in Figure 1. At the studied point, the amount of clay content is mostly 50-80 %. The amount of organic content is less than 2 %.

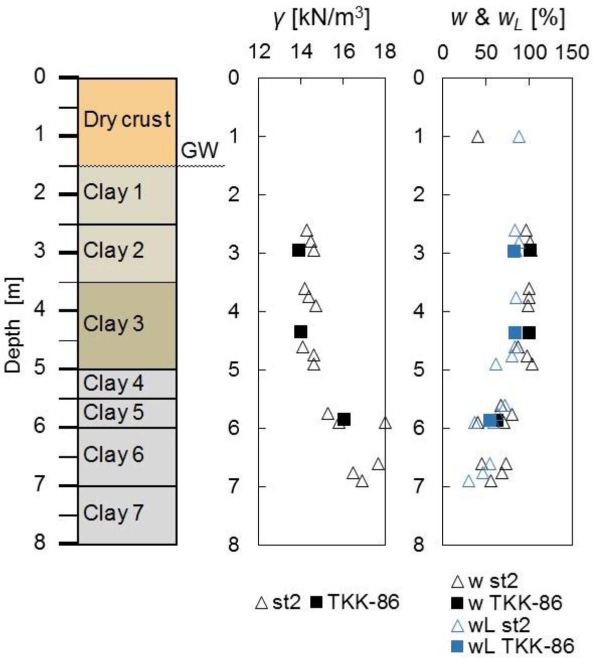

Figure 1 Soil layers, unit weight (γ), water content (w) and liquid limit (wL) from different samplers.

Undrained shear strength c_u was estimated using the results of field vane shear test, CPTU sounding test and fall cone test. Remoulded (=disturbed) undrained shear strength c_{ur} was determined using fall cone test. Soil profile with undrained shear strength determined using different tests can be seen in Figure 2 (right).

Undrained shear strength c_u can be estimated from CPTU results using Equation 1 (Lunne et al., 1997):

$$c_u = \frac{q_c - \sigma_{vo}}{N_k} \quad (1)$$

Where q_c (kPa) is measured cone resistance, σ_{v0} (kPa) is total vertical overburden pressure and N_k is cone factor.

Cone factor N_k is an empirical value, and according to Lunne and Kleven (1981) it usually varies between 10 and 15 for clays when field vane is used as reference test. In present study, fall cone test results were used as reference values, resulting in N_k = 12. This estimated c_u profile is marked as ”CPTU” in Figure 2 (right).

Interestingly, fall cone tests carried out on samples from TKK-86 sampling tubes yielded higher values of c_u compared to the ST2 values (Fig. 2). This variation might be due to difference in either sample quality or in test procedure. The determined values of sensitivity were 10-23 for ST2 and 23-37 for TKK-86. Due to scarce amount of TKK-86 results, reference values for CPTU-based estimation were selected from the highest ST2 results instead.

The preconsolidation pressure σ'_p profile was estimated not only using results of oedometer tests but also c_u profile that was estimated using CPTU results. It has been empirically proven, that there is a relation between σ'_p and c_u (Leroueil et al., 1990). Relation between c_u / σ'_p and plasticity index I_p was first proposed by Bjerrum (1972). The ratio c_u / σ'_p has been found to vary between 0.2 and 0.35 (Leroueil et al., 1990). In present study, value of 0.25 was used. The estimated preconsolidation pressure σ'_p profile (“CPTU”) is represented in Figure 2 (left). In addition, σ'_p values determined from oedometer test results, selected σ'_p and estimated effective overburden pressure σ'_{v0} profile are represented.

Only the clay layers near the surface are highly overconsolidated; Beneath 3.5 meters overconsolidation ratio (OCR) is between 1.2 and 1.8.

The selected values used in calculations for different soil layers (Fig. 1) are listed in Table 1. Preconsolidation pressure σ'_p profile was defined by giving values at top and bottom borders of each layer. The values of c_u were selected based on ST2 results of fall cone test. For dry crust, c_u of the clay layer underneath was selected.

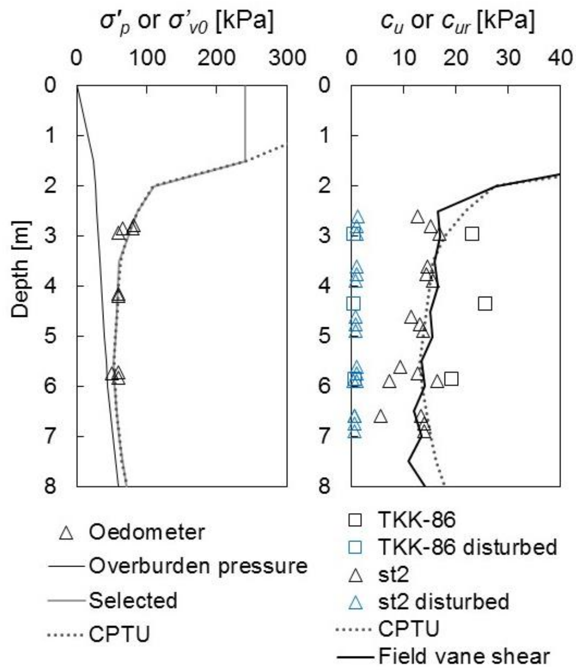

Figure 2 Left: Preconsolidation pressure (σ'_p) and effective overburden pressure (σ'_{v0}) versus depth. Right: Undrained shear strength (c_u) and remoulded undrained shear strength (c_{ur}) versus depth.

Table 1 Values used in calculations.

Layer	γ [kN/m^3]	σp' top [kPa]	σp' bottom [kPa]	cu [kPa]
Dry Crust	16	240	240	17
Clay 1	15	240	87	17
Clay 2	14.3	87	60	17
Clay 3	14.3	60	55	15,5
Clay 4	14.5	55	53	14
Clay 5	15.5	53	54	16
Clay 6	17	54	61	14
Clay 7	17.3	61	72	15

3 DESIGN OF VERTICAL DRAINAGE

3.1 Vertical drain theory

In order to evaluate the drain spacing that is needed in order to acquire a certain degree of consolidation at given time, vertical drain theory presented by Hansbo (1981) is often adopted.

According to Technique Systems/Cofra (1995) and Ye at al. (1991), settlement caused by vacuum preloading can be calculated using a vertical load as a substitute for vacuum pressure. Thus, at least in preliminary design, vacuum preloading can be modelled using methods of conventional PVD improvement.

Hansbo's solution is based on Barron's (1948) "equal strain" solution to the problem of radial (horizontal) consolidation. Hansbo (1981) included smear effect and well resistance into Barron's solution. When vertical drains are installed, the soil near the drain disturbs. This smear effect decreases the horizontal permeability near the drains and thus slows down the consolidation. If PVDs are over 20 meters long, drain resistance must be taken into account (Vepsäläinen and Arkima, 1994). At the studied point the length of PVDs is only 7 m, and thus only smear effect is taken into consideration.

According to Hansbo's solution, the average degree of consolidation for a site with PVDs is (Equations 2–4):

$$U_h = 1 - e^{\frac{-8T_h}{\mu}} \tag{2}$$

$$\mu = \ln\left(\frac{n}{s}\right) + \left(\frac{k_h}{k_s}\right) \cdot \ln(s) - \frac{3}{4} + \pi \frac{2l^2 k_h}{3q_w} \tag{3}$$

$$T_h = \frac{c_h t}{d_e^2} \tag{4}$$

Where U_h is average degree of radial consolidation, T_h is time factor for radial consolidation, c_h is horizontal coefficient of consolidation, t is time, q_w is discharge capacity of PVD, l is drainage length, k_h is average horizontal permeability of the undisturbed zone, $n = d_e/d_w$ where d_e is equivalent diameter of the cylinder of soil around the drain and d_w is equivalent diameter of PVD, $s = d_s/d_w$ where d_s is diameter of the smear zone and k_s is average horizontal permeability in the smear zone.

PVDs are band-shaped, and thus an equivalent circular cross section is needed in order to apply Barron's theory. Originally proposed by Kjellman (1948) and later verified by finite element analysis (Hansbo, 1979), equivalent diameter of PVD can be calculated using Equation 5:

$$d_w = \frac{2(w_{PVD} + t_{PVD})}{\pi} \tag{5}$$

Where w_{PVD} is width of the PVD and t_{PVD} is thickness of the PVD.

Equivalent diameter d_e depends on PVD spacing and pattern. In this paper, triangle pattern is selected, leading to equivalent diameter of $d_e = 1.05a$, where a is drain spacing (Vepsäläinen and Arkima, 1994).

3.2 The determination of horizontal coefficient of consolidation

In Finland, one of the most used method to determine horizontal coefficient of consolidation (and horizontal permeability) is horizontal oedometer test. In this test, the sample is turned 90 degrees, and horizontal parameters are determined as in conventional oedometer test (Puumalainen, 1998; Hassan, 2006). Thus it is assumed, that horizontal water flow is independent of the direction of stress and compression (Leminen and Rathmayer, 1983).

In present study, vertical and horizontal coefficients of consolidation (c_v and c_h) were determined from time–settlement data of oedometer test using both Taylor's and Casagrande's method (Taylor, 1948; Casagrande, 1936). Values determined using Taylor's method at different load increments are represented in Figure 3. In the figure V stands for vertical and H for horizontal.

However, more realistic simulation of drainage and stress conditions of PVD improved subsoil can be obtained using a radial consolidation test. One radial consolidation test was carried out. In the test, a vertical drain made of geotextile with a diameter of 10 mm (=d_w) was installed in the sample. The diameter of the sample was 81 mm (=d_e). Because of the added insulation, only radial drainage occurs during the loading.

The radial coefficient of consolidation c_r is generally determined using methods that are based on Barron's "equal strain" solution. The values of c_r were determined using a method proposed by Sridharan et al. (1996), and these values are represented in Figure 3. For verification, so called inflection point method proposed by Robinson (1997) was adopted as well.

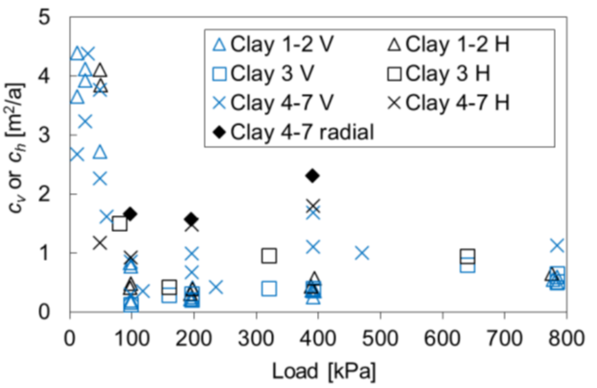

Figure 3 Vertical and horizontal coefficients of consolidation (c_v and c_h) at different load increments and layers.

Radial consolidation test yielded higher values of c_r compared to other values of c_h. In addition, there is no significant difference between c_v and c_h determined from oedometer test results. Puumalainen and Havukainen (1996) observed the same outcome and ended up using estimated value of c_h (= $2c_v$). The assumption made by Leminen and Rathmayer (1983) about insignificance of direction of compression and stress might be incorrect: fine horizontal layers that cause the anisotropic permeability might get distorted due to in-plane compression.

3.3 Smear effect

The intensity of smear effect depends on the diameter of the disturbed zone and the amount of decrease in horizontal permeability in the zone. Due to its impact on the efficiency of the vertical drainage, the smear effect has been studied extensively in both laboratory and on site.

The diameter of the smear zone d_s is often estimated to be $d_s = 2d_m \approx 2d_w$, where d_m is equivalent diameter of the mandrel (Bergado et al., 1991; Hansbo, 1987b; Saowapakpiboon et al., 2010; Voottipruex et al., 2014; Tielaitos, 1994). Indraratna and Redana (1998) investigated the smear diameter in laboratory and observed the ratio d_s/d_w being as high as 4-5. Based on previous studies, Basu et al. (2006) proposed using a ratio of d_s/d_w between 2 and 4.

Similarly, estimates for ratio k_h/k_s vary greatly. Basu et al. (2006) suggest a ratio from 2 to 10. Saowapakpiboon et al. (2010) observed a ratio of 6-7 based on field data back-calculation, and Voottipruex et al. (2014) a ratio from 7 to 10. Based on laboratory tests (large-scale oedometer test), the ratio has been observed to be smaller, only 1-5 (Hansbo, 1987a). According to Bergado et al. (1991), when horizontal coefficient of consolidation is small (less than 2 m^2/a), the ratio stays under 2. Vepsäläinen and Arkima (1994) suggest a ratio of $k_h/k_s = 2$ for preliminary design.

3.4 Design and analysis

The spacing of PVDs was selected based on Hansbo's solution. In design, vacuum of 70 kPa is assumed. Besides vacuum, surcharge fill of 30 kPa is applied. Thus the combined load is 100 kPa, and as such, a representative value of c_h is 1.4 m^2/a. Vacuum is applied for t = 7–8 months. Desirable minimum degree of consolidation at the end of the vacuum consolidation is U_h = 85 %. Needed value of d_e is calculated using Equations 2 and 4.

In design, 100 mm x 6 mm PVD is selected from a type-examined group (Vepsäläinen and Arkima, 1994). Equivalent diameter d_w is calculated using Equation 5, resulting in d_w = 67.48 mm.

In terms of smear effect, three extreme cases were studied. For Case 1, smear effect was considered to be moderate and for Case 2, more realistic values were used (based on laboratory tests and back-calculation conducted by other researches, as discussed in previous section. As for Case 3, worst possible smear conditions were assumed. For Cases 2 and 3, PVD spacing a was adjusted in order to meet desirable values of U and t.

The used parameters in Hansbo's solution (Equations 2–4) and required spacing a in each case are listed in Table 2. For Luhtaanmäki, drain spacing of a = 1 m was chosen.

Table 2 Design of vertical drainage: three cases.

Case	**1**	**2**	**3**
***a* [m]**	1.10	0.75	0.50
d_e (triangle) [m]	1.155	0.788	0.525
n	17.12	11.67	7.780
s	2	3	4
k_h/k_s	2	5	10
μ	2.783	6.101	13.78
T_h	0.660	1.447	3.267
***t* [months]**	7.55	7.69	7.72
***t* [a]**	0.63	0.64	0.64

4 SETTLEMENT PREDICTIONS

4.1 Stability analysis

In the case of natural subsoil, a preliminary short term stability analysis (friction angle φ = 0, undrained strength analysis) was carried out. Used parameters (c_u) for clay layers are listed in Table 1. For surcharge fill, suggested values for gravel are used; φ = 36 ° and γ = 20 kN/m^3 (Tielaitos, 1999).

In the safety analysis, the height of the embankment is 1.5 m. This surcharge fill leads to a load of 30 kPa as discussed in previous section. On the embankment, an equally distributed load of 10 kPa is applied.

Safety factor was obtained using method of slices. Several solution methods were applied in order to find the lowest safety factor. Morgenstern-Price method yielded the lowest safety factor F = 1.91. Thus there is no stability issues when the embankment is constructed before the vacuum.

4.2 Settlement without ground improvement

For comparison, total settlement of natural subsoil was estimated using tangent modulus method. The method is based on concepts represented by Ohde (1939) and Janbu (1963).

In present study, the modulus numbers and stress exponents were determined from vertical oedometer test results by curve fitting. In order to avoid possible errors caused by negative stress exponent, a correction method proposed by Länsivaara (2003) was used: stress exponents and modulus numbers were tied to a given preconsolidation pressure σ'_p. This given σ'_p is a product of curve fitting that was used in determination of other parameters of tangent modulus method. The selected sets of parameters are listed in Table 3. The sets are from a single test, and representative sets are selected for each layer. Again, for dry crust, rough estimates are used.

Table 3 Parameters for tangent modulus method.

Layer	σ_p' [kPa]	m_1	β_1	m_2	β_2
Dry Crust	240	100	1	100	1
Clay 1-2	84,4	3,04	-0,89	53,3	0,41
Clay 3	60,3	4,21	-0,72	63,2	0,40
Clay 4-7	46,4	7,24	-0,27	54,6	0,45

As the truck parking area is large and the clay deposit is relatively thin, the embankment can be modelled as an equally distributed extensive load. Therefore, full additive pressure is applied in the deepest layers as well.

Total settlement caused by 2 m high embankment (40 kPa) is 388 mm. Parameters used in the settlement calculation are listed in Tables 1 and 3. The rate of settlement was estimated using the vertical coefficients of consolidation. As expected, without PVDs the time required to reach U = 85 % was significantly longer compared to PVD improved case: With an embankment height of 2 meters, the time required is around 8 years (if U is calculated using total settlements. Using pore pressures, however, the time required is around 4 years).

4.3 Rate of settlement with vacuum preloading

The effect of PVDs and the vacuum to the rate of settlement is taken into account by using an approximate method proposed by Chai et al. (2001). In this method, PVD-improved soil is modelled by using equivalent vertical permeability, which takes into account both vertical and horizontal consolidation. As such, PVD improvement can be modelled under 1D condition. In addition, the effect of the surcharge fill is taken into account as well.

The equivalent vertical permeability k_{ve} according to Chai et. al (2001) is:

$$k_{ve} = (1 + \frac{2.5l^2}{\mu d_e^2}\frac{k_h}{k_v})k_v \tag{6}$$

Where k_v is vertical permeability, l is drainage length and k_h is horizontal permeability. Other parameters are defined earlier.

The values of vertical and horizontal permeability were determined from oedometer and radial consolidation test results. For all determined coefficients of consolidation, corresponding values of permeability k were calculated using unit weight of water and secant modulus.

Initial permeability k_1 (when vertical strain ε_1 = 0 %) was determined using linear extrapolation in *log* k - ε_1 plot. Values of initial permeability k_1 are represented in Figure 4. In the graph "direct" means that the permeability was determined using falling-head method integrated with IL oedometer test.

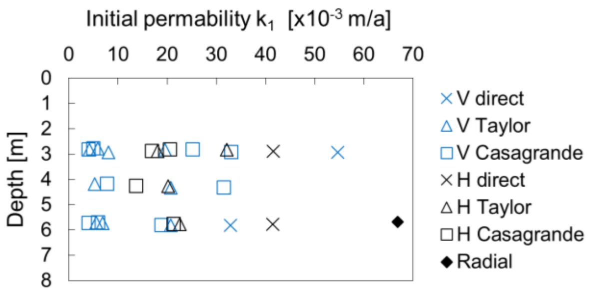

Figure 4 Initial vertical (V), horizontal (H) and radial permeability versus depth.

Direct measurement and radial consolidation test yielded significantly higher values of permeability compared to the ones estimated via coefficient of consolidation. As the amount these values is relatively low, the selected values of permeability are from the group of the highest indirectly determined. Selected values of k_v and k_h are listed in Table 4.

As the PVDs cannot reach the sandy silt layers, drainage length of l = 7.5 m is selected. However, the actual length of PVDs is only 7 m: The suction caused by vacuum affects layers beneath the drains as well. As d_e = 1.155 (Case 1 in Table 2), the affected total depth is approximately 7 m + 0.5$d_e \approx l$ = 7.5 m.

The values of k_{ve} used in calculations are listed in Table 4. Values of k_{ve} are calculated using term μ of Case 1 (Table 2). However, it should be noted that k_{ve} is approximately identical for all three cases if spacing a is adjusted to meet the same requirements for U and t. Decrease in permeability due to compression is not taken into account in the time–settlement analysis.

Table 4 Selected values of permeability.

Layer	k_v [m/a]	k_h [m/a]	k_{ve} [m/a]
Dry Crust	1	1	38.88
Clay 1-2	0.025	0.031	1.199
Clay 3	0.017	0.017	0.661
Clay 4-7	0.021	0.042	1.612

4.4 *Loading history and time-settlement for vacuum preloading*

Settlement caused by surcharge fill and vacuum is calculated using tangent modulus method. Loads are modelled as vertical equally distributed extensive load Thus compression parameters are the same as for natural subsoil (Table 3).

Applied load history and calculated time–settlement graph are represented in Figure 5. First, an embankment is constructed on the PVDs during one month. Height of the surcharge fill is 1.5 m, resulting in load of 30 kPa. Then, vacuum preloading (suction of 70 kPa) is applied for 8 months. The largest settlement is reached at the end of the vacuum, 890 mm. Soon after the removal of the vacuum, settlement decreases to 823 mm due to heave.

In order to compensate for the settlement caused by surcharge fill and vacuum, height of the embankment is increased by 0.5 m (10 kPa) during 6 moths. The increase in the height of the embankment causes a settlement of 2 mm, resulting in the final value of total settlement of 825 mm. Thus, the final height of the embankment with settlement considered is 1.2 m. The load at the end is 40 kPa.

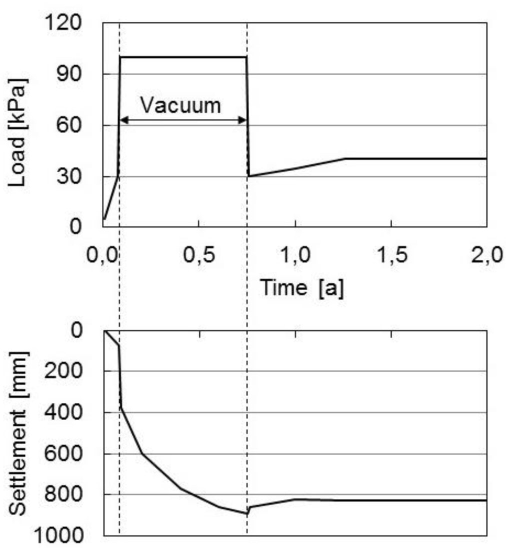

Figure 5 Loading history and time-settlement.

4.5 *Discussion*

In order to estimate the degree of overconsolidation U_{OC} caused by the vacuum, the values of effective vertical pressure before and after the vacuum were compared. Preconsolidation pressure, initial effective overburden pressure and effective vertical pressure at different loading conditions (before the vacuum, t = 0.08a, at the end of the vacuum, t = 0.75a and two years after the vacuum, t = 2a) are represented in Figure 6 (left).

At the end of the vacuum (t = 0.75a) some excessive pore pressure is present in the deepest layers ($u \approx 11$ kPa). The degree of consolidation at this point is U = 90…95 %, which is relatively close to the set goal of U = 85 % (Hansbo's solution). Immediately after the removal of the vacuum pore pressures turn to negative. However, using negative values of pore pressure would overestimate the reached value of U_{OC}. Half a year after the removal of the vacuum (t = 1.26a), excessive pore pressure is zero. Thus, U_{OC} is estimated by dividing the maximum "new" preconsolidation pressure at the end of the vacuum by the effective vertical (overburden) pressure half a year after the removal of the vacuum. The effective overburden pressure before any loading is subtracted from these values. The degree of overconsolidation U_{OC} profile and the targeted value are represented in Figure 6 (right).

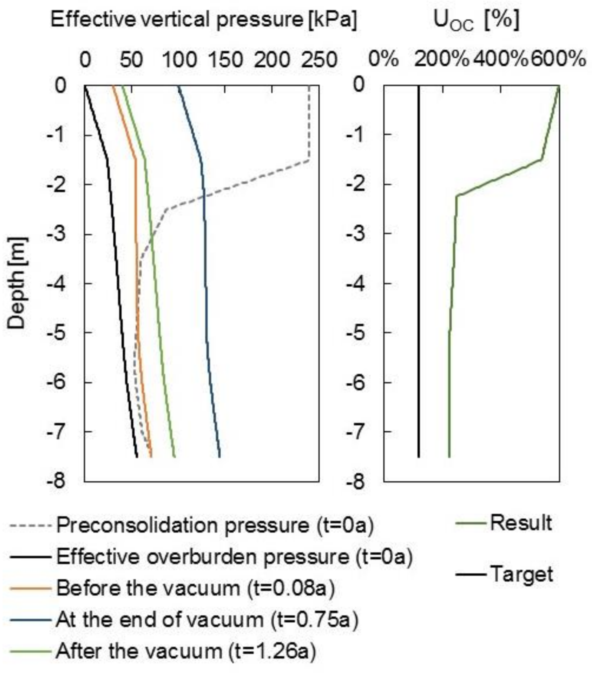

Figure 6 Left: Preconsolidation pressure, effective overburden pressure and; effective vertical pressure before, at the end and after the vacuum. Right: Degree of overconsolidation; target and the result.

In the layers near the surface, the preconsolidation pressure σ'_p does not change as the additive pressure caused by vacuum does not exceed the original values of σ'_p. Thus in these layers, degree of overconsolidation U_{OC} > 250 %. The deepest layers (beneath 2.3 m) are the most critical: The minimum value is only U_{OC} = 222 %. The average value is $U_{OC} \approx 229$ %. Hence, the average difference between maximum preconsolidation pressure and final effective overburden vertical pressure is 51,5 kPa. At minimum, the difference is 48,9 kPa.

The targeted U_{OC} was far exceeded, and at such high values settlement caused by creep is most certainly at its minimum. Moreover, the design load for the truck parking area will not cause any primary consolidation.

Time-settlement -analysis based on equivalent vertical permeability (Chai et. al., 2001) is simple and fast and seems to agree well with Hansbo's method. However, similarity in estimated degrees of consolidation might be a coincidence, because c_h in normally consolidated state used in Hansbo's method cannot yield similar time-settlement behavior as initial permeability. The reason for apparent similarity is probably in drainage boundary conditions: In settlement calculations, the bottom was assumed impermeable. However, drainage length is already taken into account in the definition of k_{ve}. Nevertheless, for preliminary design this approach seems feasible as set goals for U and t are met.

Treating vacuum preloading as an equally distributed vertical load might lead to errors. Indeed, Indraratna et al. (2012) questioned whether k_{ve} –method can be used to simulate the propagation of vacuum.

The rate of consolidation is probably faster than estimated. Thus here required PVD spacing might be sparser in actual field conditions. There are several reasons for possible error. Firstly, values of permeability in field are probably higher than the ones determined in laboratory. Secondly, the horizontal oedometer test might underestimate the values of horizontal coefficient of consolidation and permeability due to parallel loading conditions. As a matter of fact, in several recent studies radial consolidation test is preferred over horizontal oedometer test in the design of PVD improvement (Bergado et al., 1991; Ganesh

Kumar et al., 2015; Indraratna et al., 2010; Saowapakpiboon et al., 2010). Thirdly, all the methods used in this paper are originally designed for PVD improved subsoil without the effect of vacuum. Compared to conventional PVD improvement, vacuum preloading reduces the time required for consolidation by increasing the back-calculated value of c_h and by decreasing the smear effect (Saowapakpiboon et al., 2010; Voottipruex et al., 2014).

5 CONCLUSION

Clearly, vacuum preloading is a feasible soil improvement method for Luhtaanmäki. With vertical drainage, the rate of primary settlement increases significantly compared to natural state soil.

The targeted degree of overconsolidation (U_{OC} = 120 %) after 7-8 months of vacuum consolidation was easily reached. In the most critical layers the average value is $U_{OC} \approx 229$ %. Vertical drain spacing of 1 m was chosen, but in actual field conditions the needed drain spacing might be sparser.

The final height of the embankment is 1.2 m with total settlement of 825 mm taken into account. Thus the desired height was reached.

In order to increase the accuracy of vacuum preloading design, more reliable values of horizontal coefficients of consolidation and permeability are needed. Either more radial consolidation tests or for example CPTU dissipation tests (t > 24 h) should be conducted. Reliability of time-settlement –analysis could be increased by modelling the decrease in permeability due to compression. Furthermore, the vacuum should be modelled more realistically, for example by setting boundary conditions for pore water pressure. Thus FEM analysis using either axisymmetric unit cell or drain elements in plane strain conditions is needed.

6 NOTATION

a	drain spacing
c_h	horizontal coefficient of consolidation
c_r	radial coefficient of consolidation
c_u	undrained shear strength
c_{ur}	remoulded undrained shear strength
c_v	vertical coefficient of consolidation
d_e	equivalent diameter of PVD-affected soil
d_s	diameter of the smear zone
d_w	equivalent diameter of PVD
k	permeability
k_1	initial permeability
k_h	horizontal permeability
k_s	horizontal permeability in smear zone
k_v	vertical permeability
k_{ve}	equivalent vertical permeability
l	drainage length
n	d_e/d_w
N_k	cone factor
OCR	overconsolidation ratio
q_c	measured cone resistance
q_w	discharge capacity of PVD
s	d_s/d_w
t	time
t_{PVD}	thickness of PVD
T_h	time factor for radial consolidation
U	degree of consolidation
u	pore water pressure
Uh	average degree of radial consolidation
U_{OC}	degree of overconsolidation
w	water content
w_L	liquid limit
w_{PVD}	width of PVD
γ	unit weight
σ_{v0}	total overburden pressure
σ'_{v0}	effective overburden pressure
σ'_p	effective preconsolidation pressure
φ	friction angle

7 REFERENCES

Barron, R.A. (1948). Consolidation of fine grained soils by drainwells. Transactions of ASCE 113, 718-724.

Basu, D., Basu, P. & Prezzi, M. (2006). Analytical solutions for consolidation aided by vertical drains. Geomech. Geoeng. Int. J. 1 (1), 63-71.

Bergado, D.T., Asakami, H., Alfaro, M.C. & Balasubramaniam, A.S. (1991). Smear effects of vertical drains on soft Bangkok clay. Journal of Geotechnical Engineering 117 (10), 1509-1530.

Bjerrum, L. (1972). Embankments on soft ground. Proc., ASCE Specialty Conf. Earth and Earth-Supported Structures, Purdue University. Vol. 2, 1-54.

Brandt, V., Kirstein, J.F., Nörenberg, G. & Wittorf, N. (2015). Winner of German road innovation price: "Peaty soils become suitable foundation ground of a federal highway by implementing the vacuum system. Proceedings of the XVI ECSMGE for Infrastructure and Development, 295-300.

Casagrande, A. (1936). The determination of preconsolidation load and its practical significance. Proceedings of the I ICSMFE, 3. Mass. Cambridge

Chai, J.-C., Shen, S.-L., Miura, N. & Bergado, D.T. (2001). Simple method of modelling PVD-improved subsoil. Journal of Geotechnical and Geoenvironmental Engineering 127 (11), 965-972.

Chaumeny, J.-L. & Kirstein, J. & Reiner, J. (2011). Consolidation of extremely soft mud for the A-380 factory in Hamburg – long time experience. Proc. XV ECSMGE, 2011. IOS Press. 971-976.

Ganesh Kumar, S., Sridhar, G., Radhakrishnan, R, Robinson, R.G. & Rajagopal, K. (2015). A Case Study of Vacuum Consolidation of Soft Clay Deposit. Indian Geotechnical Journal 45 (1), 51-61.

Hansbo, S. (1979). Consolidation of clay by band shaped prefabricated drains. Grnd. Eng. 12 (5), 16-25.

Hansbo, S. (1981). Consolidation of fine-grained soils by prefabricated drains. Proceedings of the X ICSMFE, Stockholm, June 1981. A.A. Balkema, Rotterdam, The Netherlands. Vol. 3, 677-682.

Hansbo, S. (1987a). Design aspects of vertical drains and lime column installations. Proceedings of the 9th Southeast Asian Geotechnical Conference, Bangkok, Thailand. Vol. 2, 1-12.

Hansbo, S. (1987b). Fact and fiction in the field of vertical drainage. Proceedings of the International Symposium on Prediction and Performance in Geotechnical Engineering, Calgary 1987. A.A. Balkema, Rotterdam. 61-72.

Hassan, M. (2006). Deformation behavior and permeability of soft Finnish clay. Master's Thesis. Helsinki University of Technology.

Indraratna, B & Redana, I.W. (1998). Laboratory determination of smear zone due to vertical drain installation. Journal of Geotechnical and Geoenvironmental Engineering 124 (2), 180-184.

Indraratna, B., Rujikiatkamjorn, C. & Sathananthan, I. (2005). Radial consolidation of clay using compressibility indices and varying horizontal permeability. Can. Geotech. J. 42, 1330-1341.

Indraratna, B., Geng, X. & Rujikiatkamjorn, C. (2010). Review of methods of analysis for the use of vacuum preloading and vertical drains for soft clay improvement. Geomechanics and Geoengineering: An International Journal Vol. 5 (4), 223-236.

Indraratna, B., Rujikiatkamjorn, C., Balasubramaniam, A.S and Mcintosh, G. (2012). Soft ground imrpvement via vertical drains and vacuum assisted preloading. Geotextiles and Geomembranes 30, 16-23.

Janbu, N. 1963. Soil compressibility as determined by oedometer and triaxial tests. Europäische Baugrundtagung 1963, Wiesbaden.

Kjellman, W. (1948). Accelerating consolidation of fine grain soils by means of vardboard wicks. Proceedings of the 2nd ICSMFE, Vol. 2, 302-305.

Leminen, K. & Rathmayer, H. (1979). Heikosti kantavan maan hienorakeisen maapohjan parantaminen pystyojituksen ja syvästabiloinnin avulla. Kenttäkokeet Helsingin Itäkeskuksessa. Report 39. VTT Technical Research Centre of Finland Ltd, Geotechnical laboratory, Espoo.

Leminen, K. & Rathmayer, H. (1983). Experience on horizontal oedometer tests to determine the parameter c_h. Improvement of Ground. Proceedings of the XIII ECSMFE, Helsinki 23-15 May 1983. A.A. Balkema, Rotterdam. 647-651.

Leroueil, S., Magnan, J.-P. & Tavenas, F. 1990. Embankments on soft clay. (Translator Wood, D.M.) Ellis Horwood, Chichester.

Lunne, T. & Kleven, A. 1981. Role of CPT in North Sea foundation engineering. Session at the ASCE National Convention: Cone Penetration Testing and Materials, St. Louis. ASCE. 76-107.

Lunne, T., Robertson, P.K. & Powell, J.J.M. (1997). Cone Penetration Testing in Geotechnical Practice. Blackie Academic & Professional, London.

Länsivaara, T. (2003). Problems related to settlement calculations on soft Scandinavian clays. Int. Workshop on Geotechnics of Soft Soils-Theory and Practice. Vermeer, Schweiger, Karstunen & Cudny (eds.). VGE.

Müller, R. & Larsson, S. (2013). Aspects on the modelling of smear zones around vertical drains. Proc., XVIII ICSMGE, Paris 2013, 2965-2968.

Ohde, J. (1939). Zur Theorie der Druckverteilung im Baugrund. Bauingenieur 14, 451-458.

Puumalainen, N. & Havukainen, J. (1996). Vakuumikonsolidointikoe Helsingin Torpparinmäessä. Report 68/1996. Geotechnical Division, Real Estate Department, City of Helsinki.

Puumalainen, N. (1998). Mechanical Modelling of vacuum consolidation (in Finnish). Licentiate's Thesis. Helsinki University of Technology.

Robinson, R.G. (1997). Determination of radial consolidation of consolidation by the inflection point method. Geotechnique 47 (5), 1079-1081.

Saowapakpiboon, J., Bergado, D.T., Youwai, S., Chai, J.C., Wanthong, P. & Voottipruex, P. (2010). Measured and predicted performance of prefabricated vertical drains (PVDs) with and without vacuum preloading. Geotextiles and Geomembranes 28, 1-11.

Seah, T.H. (2006). Design and construction of ground improvement works at Suvarnabhumi Airport. Geotechnical Engineering, Journal of the Southeast Asian Geotechnical Society 37: 171-188.

Sridharan, A., Prakash, K. & Asha, S.R. (1996). Consolidation behaviour of clayey soils under radial drainage. Geotech. Test. J. 19 (4), 421-431.

Taylor, D.W. (1948). Fundamentals of Soil Mechanics. John Wiley & Sons Inc., New York.

Technique Geosystems/ Cofra, J.V. (1995). Design and proposal for the execution of: Vacuum consolidation Shah Alam Expressway Package A, Selangor, Malaysia. Amsterdam.

Tielaitos. (1999). Pohjarakennusohjeet sillan-suunnittelussa. Tiehallinto. Edita Oy, Helsinki.

Vepsäläinen, P. & Arkima, O. (1994). Geotekniikan informaatio-julkaisuja. Nauhapysty-ojitus. Tielaitoksen selvityksiä 42/1994. Helsinki.

Voottipruex, P., Bergado, D.T., Lam, L.G. & Hino, T. (2014). Back-analyses of flow parameters of PVD improved soft Bangkok clay with and without vacuum preloading from settlement data and numerical simulations. Geotextiles and Geomembranes 42, 457-467.

Ye, B.R., Shang, S.Z. & Ding, G.Q. (1991). Consolidation of soft soil by means of combined vacuum surcharge preloading. Proc., Int. Conf. Geotech. Eng. for Coastal Development - Theory and Practice on Soft Ground - Geo-Coast '91 Yokohama 1991, Vol. 2, 277-280.