



Svenska Geotekniska Föreningen
Swedish Geotechnical Society



Rapport 1:2025

Nordic Geotechnical Meeting 2024

Next Generation Meeting

Volume 2



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Nordic Geotechnical Meeting 2024

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FOREWORD

In 2024 it was the Swedish Geotechnical Society's time to organize NGM. Organizing an event gathering geotechnical people from not only all the Nordic countries, but from countries all over the world requires some planning. We decided for the beautiful city of Gothenburg, on the west coast of Sweden, famous for its rainy weather. However, Gothenburg really exceeded all expectations with summer weather in September.

I think NGM 2024 really managed to show what the Nordic Geotechnical Meeting is all about – getting to interact with fellow geotechnical engineers. During the days we got to see great presentations, a wonderful study visit/bus tour in sunny Gothenburg, a dinner with a magic touch and finally we got to meet a whole bunch of new people all sharing the passion for geotechnical engineering.

Without the management of the NGM 2024-general Victoria Svahn and the working group (Andreas Flyckt, Jonas Axelsson, Helen Kennedy and myself) none of this would have been possible – a huge thanks to all of you! I would also like to thank the secretariat Ragna Hellberg and Hillevi Hellberg, without your support the event would probably have been a disaster. In this book you will find the papers from all the great presentations. Enjoy!



Fanny Deckner
Chairman of the Swedish Geotechnical Society



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FOUNDATION ON ORGANIC DIATOMITE

J. D. Andersen, L. Bødker, J. Dolby, A. Damsgaard and N. Okkels¹

KEYWORDS

Diatomite, organic, interglacial, strength, deformation, pre-consolidation, foundation, piling.

ABSTRACT

In glacially formed landscapes, organic soils of interglacial age are sometimes found in geotechnical site investigations for building projects. The thickness of such layers is often minor and the pre-consolidation of the layers from e.g. ice transgressions in glacial periods produces soil parameters that are often acceptable for foundations of structures with delimited loads in soils above the interglacial organic layer.

On a building project in the Danish town Vejle, an unusually thick layer of interglacial organic diatomite was found. Furthermore, the geology was surprisingly complex on the ~40 meter wide building site – with large variations over short distances – included layers of soft postglacial organic soils and dense sands with varying thicknesses.

Diatomite is not commonly found in Denmark and maybe as a consequence not well described in the geotechnical literature. This – besides the local found thickness of the formation and significant classification parameters – called for further investigations of the geotechnical properties, also to comply with the demand of testing in geotechnical category 3 in the European geotechnical standard.

In this article, the geotechnical characteristics and achieved knowledge of the found organic diatomite are described. The investigation includes borings, CPTs, laboratory classification tests, oedometer and triaxial tests and pile driving results with pile driving analyses.

INTRODUCTION

This article is initiated by a project in Vejle in Denmark, where a sewage treatment plant in 2023 was to be enlarged by a new tank and an adjacent pumping station.

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At first, a borehole in the center of the tank area showed several meters of fill and postglacial organic soils, which brought up the necessity of a piling foundation into subsoils of sand. The following extended geotechnical site investigation for the design of the project revealed unexpected differing ground conditions within the relatively narrow building site, including the presence of relatively thick interglacial layers of organic diatomite. Although diatomite is well-known in this part of Denmark, it is not commonly found and handled. The geotechnical properties of diatomite have not previously been investigated deeply and, as a consequence, are not well described in the literature. As the characteristics of the diatomite were essential for the project, an extended investigation was necessary to comply with the demand for testing in geotechnical category 3 in the European geotechnical standard.

The authors aim with this article to present the knowledge gained about diatomite soils in order to increase the general knowledge for future projects involving diatomite soils.

PROJECT AND GEOTECHNICAL INVESTIGATIONS

The project comprised an expansion of an existing sewage treatment plant in the Danish town of Vejle, where a storage tank and an adjacent pumping station were to be built. The tank has a diameter of 34 meters to approx. 3 meters depth at terrain levels +2 to +3 m DVR90 (meter above sea level). The pumping station is 7 m by 8 m and taken to approx. 7 m depth.

Initially, a geotechnical boring (B1) to 20-meter depth had been executed. The boring showed 5 meters of fill and 4 meters of postglacial marine and freshwater deposits of organic soils (including peat and gyttja) over inorganic marine and meltwater dense sands to 16 meters depth. Below this, clay and gyttja of assumed Miocene age were described. Based on these results, a foundation on driven concrete piles was presumed, with pile toes in the sand. The groundwater table was found at a depth of 1.5 meters, which called for groundwater lowering in the construction phase, especially for the pumping station.

Afterward, Geo performed a geotechnical parameter investigation for the project, as shown in Figure 1 (ref. 3). First, three Cone Penetration Tests (CPTu 101 – 103) were performed at the perimeter of the tank area, where a penetration depth of 30 meters was presumed sufficient and abundant. The evaluation of the CPTu's showed some severe variations in the layering and soil firmness compared to the initial boring. It was found necessary to supplement these with two borings (104 and 105), originally solely planned for pumping tests, but deepened to 25 and 40 meters depth and expanded to include fast field vane tests and extraction of soil samples for advanced laboratory testing.

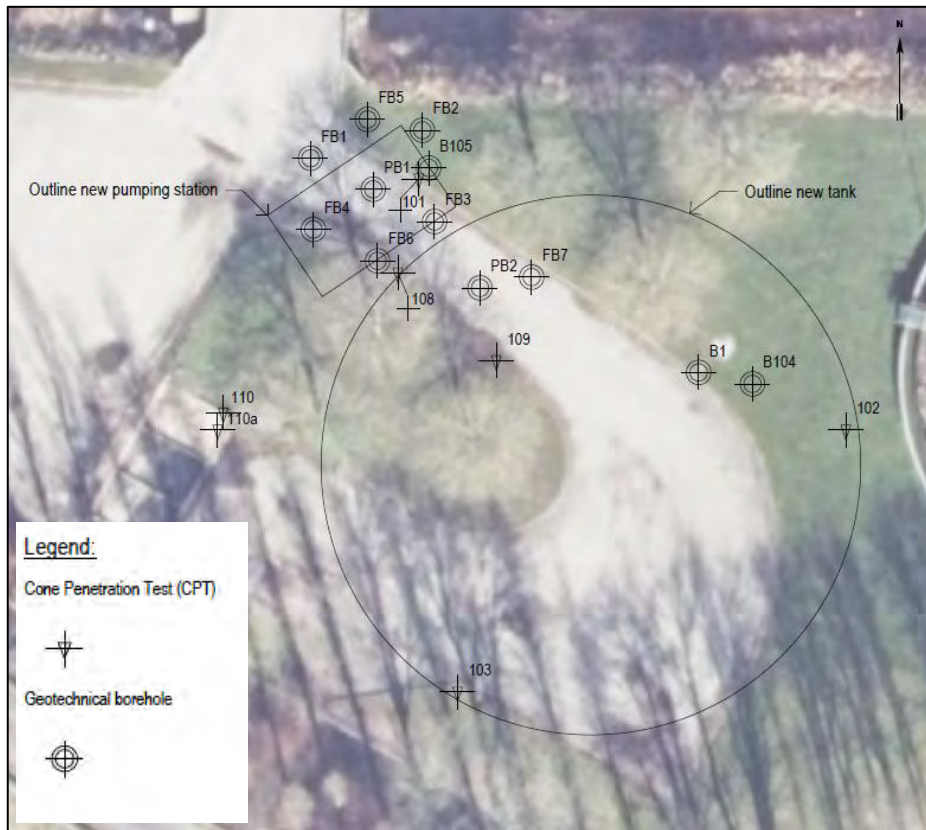


Figure 1: Site plan with borings and CPTu's.

The geological description of the extracted samples showed dominating layers of organic diatomite of interglacial age. Furthermore, the thickness of the sand layer – in which the piling was planned – seemed to thin-out towards one side. After driving of 24 relatively short test piles equally spread in the tank area, three supplementary CPTu's (108 – 110) were performed in a part of the tank area, where the sand layer was very thin or even absent, followed by driving of five extra 30 meter test piles. The long piles were testing with pile driving analyses (PDA) six days after installation with CASE and CAPWAP analyses. For groundwater lowering, additionally nine borings to 13 – 23 meters depth, especially in the area of and around the pumping station, have been executed. Besides a geological description of all extracted samples to establish a geological model for the site, special focus has been on evaluating the geotechnical parameters of the found organic diatomite. Besides a number of classification tests (water content, density, plasticity index and loss of ignition), oedometer tests and later triaxial tests have been made on the extracted soil samples of diatomite. The results of the investigations is further described in the following, focusing on the organic diatomite.

DIATOMITE IN GEOLOGIC CONTEXT

In Denmark, freshwater diatomite is found in south-western, see Figure 2, deposited as non-calcareous sediments from differing interglacial or interstadial periods and as marine deposits from the geological Paleogene period in the north-western part. Diatomite is a fine-graded, biogenic sediment that predominantly consists of shells from microscopic algae, also called diatoms. Diatom shells are a so-called opal-A, a silicate with very high water content in the chemical form $\text{SiO}_2 - n\text{H}_2\text{O}$, an inorganic biogenic material. Pure dry diatomite is able to float on water due to its high porosity. Natural diatomite deposits contain variable levels of finely divided organic material and minerals of silt and clay, typically deposited during winter periods between summer algae blooms. If the geotechnical characteristics of the sediment are dominated by high organic content, it is tradition in Denmark to use the term diatomite gyttja.



Figure 2: Location of boreholes with interglacial and interstadial freshwater diatomite in Denmark according to the GeoAtlas Live database (green dots), combined with locations of known diatomite pits (open rings). The site location (Figures 1 and 3) is marked with blue text.

Diatomite absorbs water very easily. In saturated state, the water content is very high and the unit weight accordingly low. For the same reason, diatomite is often confused with the more commonly found organic sediments such as

gyttja. Like organic material, diatomite holds water in a way that does not contribute to the plasticity. The diatomite shells are hollow, and the encapsulated water increases the plasticity and liquid limits, but pure diatomite is non-plastic.

The site of interest is located on the northern edge of a known diatomite area in the so-called Mølholm Valley, which runs into the broader Vejle Fjord Valley, as shown in Figure 3. In previous boreholes in the Mølholm Valley, the diatomite is typically covered by postglacial organic deposits and occasionally thin layers of tills. Presumably, the diatomite extends at similar depths beyond the sides of the valley, and the postglacial formation of the Mølholm Valley eroded down to the diatomite surface.

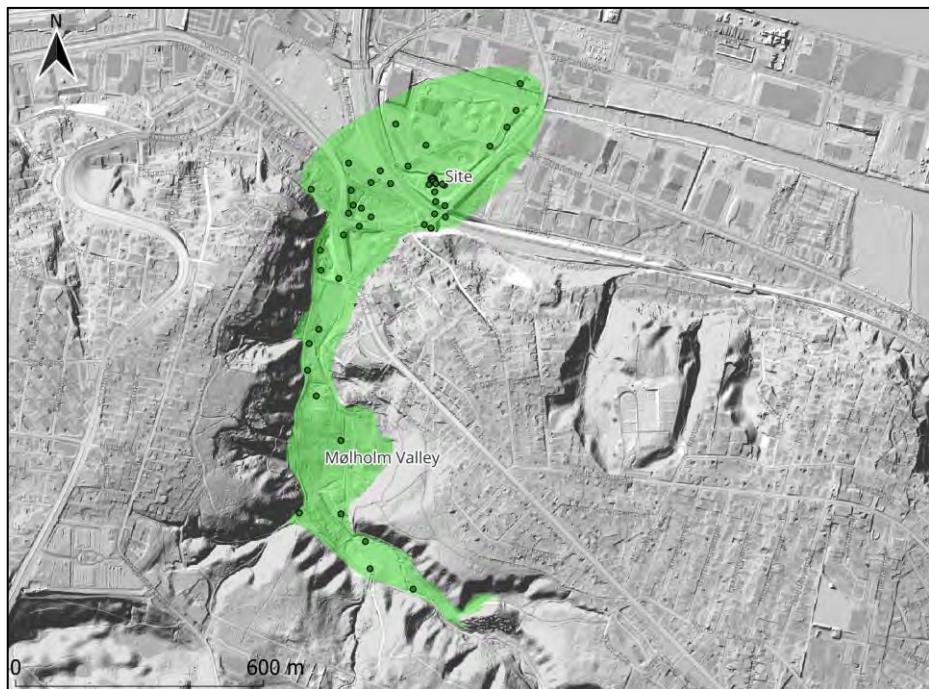


Figure 3: The extent of diatomite in the Mølholm Valley in Vejle. Green dots mark boreholes with diatomite found in GeoAtlas Live, and the green shading marks the presumed extent of shallow diatomite occurrences, covered by thin younger deposits.

The larger extent agrees with interglacial diatomite findings from similar sites in the Vejle Fjord area (ref. 1 and 2), where it is dated through pollen species assemblages to Holstein interglacial (424,000 – 374,000 yr BP) or from an intermittent warmer period within the Saale glacial complex (373,000 – 130,000 yr BP, ref. 1 and 2). The most recent glacial period, Weichsel (115,000 – 11,700 yr BP) resulted in three glacial advances over the area, forming the main Vejle Valley through subglacial meltwater erosion, making the diatomite deposits susceptible to glacial deformation and tectonism.

CLASSIFICATION TESTS

The geotechnical classification parameters of the found diatomite are listed in Table 1. Density is measured on intact tube samples at advanced laboratory tests, water contents are measured by dry-out in an oven at 105 °C, plasticity tests are performed by the fall cone method, and loss of ignition measured as weight loss after four hours at 550 °C (on oven-dried material).

Table 1: Classification parameters of diatomite.

Parameter	No. of measurements	Measured values
Density, ρ (g/cm ³)	12	1.18 – 1.26
Water content, w (%)	107	24 – 248
Plasticity limit, w_P (%)	7*	41 – 231
Liquid limit, w_L (%)	7*	22 – 212
Plasticity index, I_P (%)	7*	16 – 29
Loss of ignition, LOI (%)	13	7.4 – 17.5

Note *: Seven of 13 tested samples were found plastic (six were non-plastic).

Figure 4 shows a Casagrande plasticity chart, and all tests are positioned below the so-called A-line. Nearly all shown points represents samples from borehole 105, while nearly all tested samples from borehole 104 were non-plastic. This demonstrates – besides various CPT cone resistances – a variability of the diatomite across the site. The plasticity may be affected by a content – or absence – of e.g. clay and organic material. The loss of ignition shall be seen as the sum of chemically bounded water and organic material.

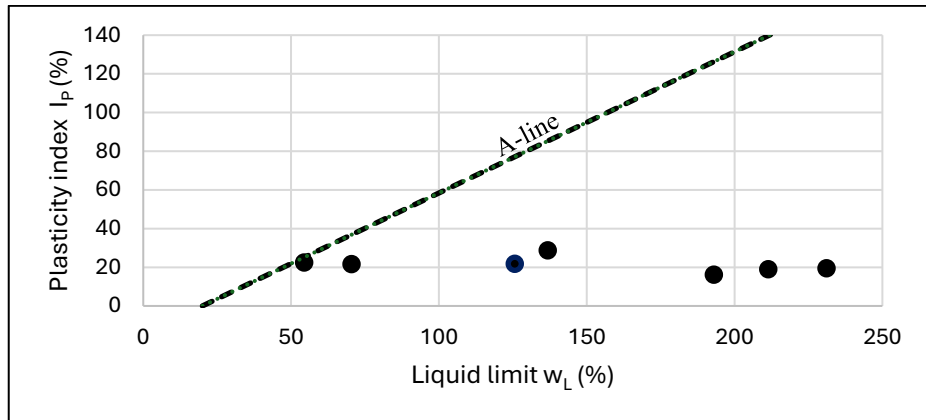


Figure 4: Casagrande plasticity chart for organic diatomite (with A-line).

OEDOMETER TESTS

To evaluate the deformation parameters in the diatomite, three oedometer tests are performed on extracted soil samples in Shelby tubes. Initially, tests have been made at incremental loading up to 3200 kPa, primarily for estimation of the pre-consolidation stress, according to methods by Casagrande, Akai and Becker/Janbu. Afterwards, new partial samples from the same three Shelby tubes are tested to evaluate the oedometer modulus at the actual vertical stress level by incremental loading up to near/below the pre-consolidation stress, stepwise unloading to a stress level (150 kPa) slightly below the evaluated vertical in situ stress and stepwise re-loading to the pre-consolidation stress. The main results are listed in table 2. The tests are performed at room temperature 22 °C.

Borehole no.	104	105	105
Level (m DVR90)	-17.4	-23.0	-25.3
Water content, prior to testing, w (%)	152/159	171/164	178/180
Loss of ignition, LOI (%)	11.3	13.8	16.2
Plasticity index, I_p (%)	NP	16.1	19.4
Measured fast field vane strength, c_{fv} (kPa)	(400?)	160	170

Estimated effective in situ stress, σ'_0 (kPa)	200	220	225
Evaluated pre-consolidation stress, σ'_{pc} (kPa)	1100	900	675
Over-consolidation ratio, OCR (-)	5.5	4.1	3.0
Compression ratio, C_{CR} (% per load decade), virgin line, max.	44.2	31.5	32.5
Coefficient of secondary compression, C_α (% per time decade), virgin line, max.	1.97	1.49	1.56
Oedometer modulus, re-loading, E_{oed} (kPa)	47100	40100	35700

Table 2: Results of oedometer tests.

The oedometer tests show that the diatomite are over-consolidated by a ratio $OCR = 3.0 - 5.5$. The oedometer modulus (for re-loading) $E_{oed} = 35 - 47$ MPa indicates a relatively high stiffness, despite the high water content ($w = 152 - 180$ %) and the high loss of ignition ($LOI = 11 - 16$ %). The drainage of the diatomite during primary consolidation progressed relatively fast, and the coefficient of secondary compression during re-loading was relatively minor.

These findings correspond well with the presumption that the diatomite has a skeleton of silicon shells, where pore water and organic material mainly are located in cavities inside the shells.

TRIAXIAL TESTS

Six triaxial tests have been carried out on two intact samples from borehole 104 at levels -21.5 and -23.5 m DVR90. Hereof, three tests are performed drained (CID_c) and three undrained (CIU_c) after isotropic pre-consolidation.

The tests are carried out by principles given in DS/EN ISO 17892-9 and Danish tradition. According to Danish tradition the test conditions are height $H =$ diameter $D = 70$ mm, with smooth end platens and drainage both from the ends and radially. The specimens are installed in the triaxial apparatus and saturated with backpressure (500 kPa) at the isotropic stress state $\sigma'_1 = \sigma'_3 = 20$ kPa. Afterwards the specimens are consolidated at a cautious mean effective pre-consolidation stress determined as $\sigma'_{m,pc} = 1/3 (\sigma'_{pc} + 2K_0\sigma'_{pc})$ in order to approximately re-established the effect of the ice-load etc. and minimize sample disturbance, where σ'_{pc} is estimated from the oedometer tests and K_0 is estimated to 0.4. At this stress level, time curve is recorded and

shear rate to be used in the shear phase is derived using the principles in DS/EN ISO 17892-9 for each test. The shear rates used are 1%/hour in all tests. Before shearing, the specimens are unloaded isotropically to selected stress levels in the range of the in situ stress using $K_{0,pc} = 1.0$. The shearing is stopped at approximately 20% additional vertical strain. The effective stress paths during shearing for all the tests are plotted in figure 5.

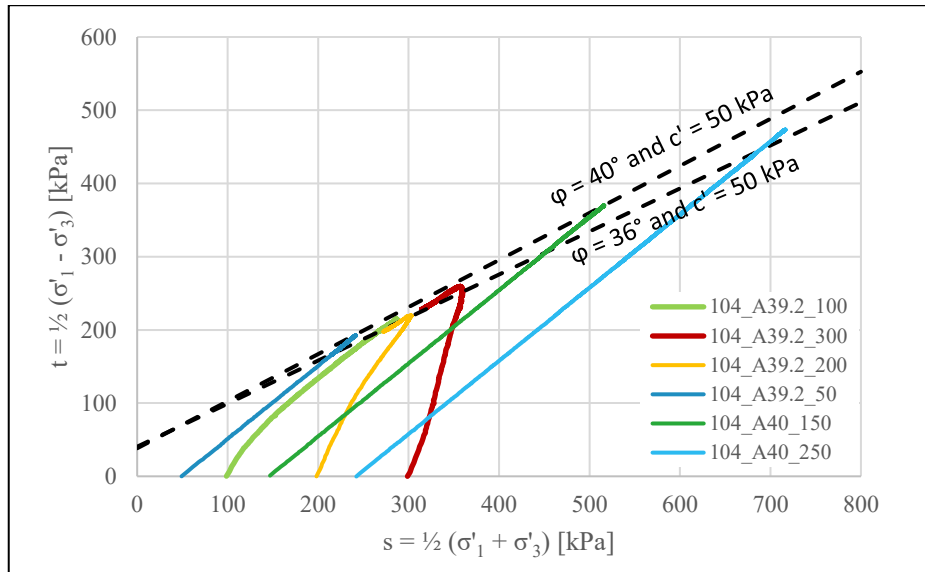


Figure 5: Effective stress paths in shearing for drained and undrained tests.

As seen in figure 5 a common failure criterion for both drained and undrained tests (Mohr-Coulomb: $\tau = c' + \sigma' \cdot \tan \phi$) can be described using a friction angle $\phi' = 36^\circ$ to 40° and an effective cohesion $c' = 50$ kPa. It is noticeable that the drained test with $\sigma'_3 = 50$ kPa and the undrained test with $\sigma'_3 = 100$ kPa show dilative behavior (volumetric expansion) during shearing, while the other tests only show tendency to volumetric contraction during shearing, although all the tests are performed on pre-consolidated specimens and therefore expected to show dilative behavior during shearing.

The drained tests were made on two samples from level -21.5 and -23.5 m DVR90. After isotropical pre-consolidation and unloading to axial stress approx. 50, 150 and 250 kPa, drained compression tests were taken to failure measuring the axial stress σ_A , radial stress σ_R and pore pressure.

The undrained tests were made on one sample from level -21.5 m DVR90. After isotropical pre-consolidation and unloading to axial stress approx. 100, 200 and 300 kPa, undrained compression tests were taken to failure. The undrained shear strength is calculated traditionally at 10% additional axial strain as $c_u = \frac{1}{2}(\sigma_A - \sigma_R) = 213 - 259$ kPa.

It is notable, that the plasticity index was aimed measured on the tested sample, but that the diatomite in these samples was found to be non-plastic. A friction angle $\varphi' = 36 - 40^\circ$ seems reasonable for non-plastic soils, the found cohesion $c' = 50$ kPa (maybe due to a curved failure line at small stresses) and the measured undrained shear strength collides by principle with this.

CPT

Six CPTu's have been performed at the site, all partly or fully penetrating the organic diatomite. In the diatomite, all CPTu's show friction ratios $R_f = f_s/q_c$ in the range of typically 2 – 5 % and evaluated soil behavior index ($I_{SBT} = 2.6 - 3.2$), which – according to normal evaluation procedures – indicates that the diatomite is to be seen as a cohesive soil. This contradicts partly with the evaluation of non-plasticity of some samples, where less friction ratios and frictional behaviour would be expected.

Figure 6 shows the cone resistance q_c in layers, which are interpreted as diatomite. A remarkable variation is seen across the 40-meter wide site with the lowest resistances towards north-west.

It is likely that various geological processes at the particular site have had a major impact on the development of a complex geological soil scene, which is difficult to reveal in detail. The site is located at the edge of a known glacial tunnel valley, where erosion in a bed of previously deposited interglacial/interstadial layers at the edge of the tunnel is followed by depositing of sands under differing sub-glacial drainage patterns. Furthermore, the area may have been feed with eroded material from surrounding areas, and wash-downs and solifluction of soils may have taken place.

Another obvious reason for the variation may be differences in plasticity and differing pre-consolidation stresses, where large q_c -values are typically found in the area with overlying (heavy) sand and non-plastic diatomite, and minor values in area with absence of overlying sand.

The geological complexity is also seeing in the over-lying postglacial deposits, which is interpreted to consist of a variability of marine and freshwater deposits (in this inner part of Vejle Fjord), ranging from in-organic sands to organic deposits (peat, gyttja etc.) to various depths.

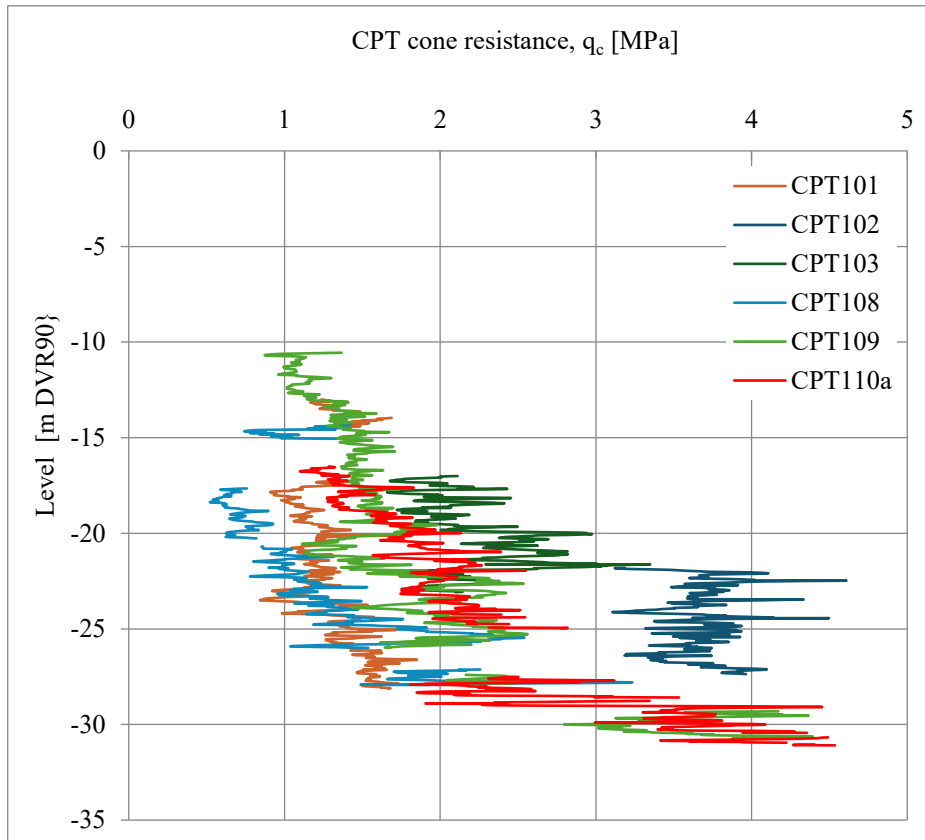


Figure 6: Cone resistance q_c in soil layers interpreted as organic diatomite.

It is notable that major variations in soil profiles are found even over short distances, which in the case of CPT 101 and borehole 104 was less than 2 meter horizontally. The relatively higher cone resistances in CPT 102 may be affected by variations in the plasticity (or even non-plasticity).

FAST FIELD VANE TESTS

Fast field vane tests have been performed in the diatomite in two geotechnical boreholes (104 and 105). The tests are performed according to the procedure in the reference sheet from the Danish Geotechnical Society with measurements of undisturbed (c_{fv}) and remoulded (c_{rv}) resistances. The calculated sensitivity is $S = c_{rv}/c_{fv} \approx 0.5$, which normally indicates a mainly cohesive soil.

Comparison of field vane tests and cone resistances is made by the formula $c_{fv} = (q_c - \sigma_{v0})/N_k$. Borehole 105 is located close to CPT 101 (at approx. 1.5 meters inter-distance), where data is available from level -21 to -28 m

DVR90. Here, $c_{fv} \approx 150$ kPa and $q_c \approx 1.2$ to 1.5 MPa produces $N_{kt} \approx 7 - 10$, which is in an expected range compared to normal cohesive soils.

Furthermore, field vane results in borehole 104 may be compared to CPT 102 (at 6 meter inter-distance). Here, a direct comparison is weakened due to interlayered sand, though. The field vane tests show predominantly $c_{fv} \approx 310 - 410$ kPa (from -16 to -23 m DVR90), and the CPT shows $q_c = 3.1 - 3.8$ MPa at the same levels, but with a layer of sand between level -18 and -22 m DVR90. It seems reasonable to adopt the deviator $N_k \approx 7 - 10$ from above.

A comparison of field vane tests with the undrained triaxial tests from borehole 104 shows $c_{fv} \approx 350 - 407$ kPa at levels around the performed undrained triaxial tests with $c_{u,tr} \approx 230$ kPa (average value). Using the formula $c_{u,tr} = \mu \cdot c_{fv}$, this produces a factor $\mu \approx 230 / (350 - 407) \approx 0.57 - 0.66$. This seems to meet a usual understanding of $\mu \leq 1.0$, but the data is evaluated dubious for further elaboration, e.g. regarding the found non-plasticity of the tested sample.

PILE BEARING CAPACITIES

The pile foundation of the actual tank was planned on relatively short pile (mostly 11 – 13 meter), primarily toe bearing in the sand (above the diatomite). According to normal practice, the axial bearing capacity was evaluated on the basis of the dynamic pile driving resistance and the Danish pile driving formula. Settlements of the piles – including contributions from the diatomite below – was evaluated negligible for the actual loads, according to the performed oedometer tests.

Contrarily to this, the geotechnical investigations had revealed that the layer of sand was practically absent in approx. one fourth of the tank area towards north-west. Here, the postglacial deposits were interpreted directly underlain by organic diatomite. Furthermore, the CPTu's had shown that several meters of the diatomite were relatively soft, but seemed to be firmer at depth, where the diatomite were underlain by sands and presumably Miocene mica clay. Due to a risk of differential settlements for the tank as a whole, it was recommended not to have pile bases in the organic diatomite, and it was decided – for one fourth of the tank area – to use longer piles with base in level -27 m DVR90 in order to penetrate the diatomite and reach harder soils. Although the diatomite was relatively soft, cohesion contributions to bearing capacity from the penetrated diatomite was included. The pile bearing resistance was estimated by geostatic calculation, using the usual formula for shaft resistance $R_s = m \cdot r \cdot c_u \cdot A_s$. Here, $m \cdot r = 0.4$ and c_u based on CPT and a cautious evaluation of N_k was chosen in the diatomite. Furthermore, the piles were cautiously presumed to have pile base in cohesion soil.

The driving resistance of the longer piles were relatively low, and to document the expected development by time in cohesion bearing capacity in

the evaluated cohesionless diatomite, pile driving analyses with CASE and CAPWAP analyses were performed on the five test pile six days after installation. These tests showed shaft resistances of 1 – 2 times the conservatively calculated resistance in the diatomite. Furthermore, the base resistance was 2 – 3 times the calculated resistance. Consequently, the pile design was proven acceptable.

CONCLUSIONS

This article aims to present the knowledge gained and geotechnical parameters for the found organic diatomite for future projects involving diatomite. The findings are intended to contribute to understanding the geotechnical behavior of diatomite, and the work has revealed issues that should be investigated further. For example, the diatomite seems to be a predominantly cohesive soil. Nevertheless, some parameters contradict traditional thinking, e.g. by varying plasticity and general behavior in the laboratory. Also, a traditional approach of evaluating water content, loss of ignition and maybe CPTu's may be challenged and subject for additional investigation. Furthermore, the complex local geology at the project site was a particular issue that required certain robustness in the foundation design.

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GENERALISATION OF EFFECTIVE HEAVE PRESSURE CONSIDERING THE EFFECT OF SMALL-STRAIN STIFFNESS

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KEYWORDS

Excavation, clay, earth pressure, heave, numerical modelling

ABSTRACT

Excavation in low-permeable soils is typically followed by delayed heave during consolidation, which is often restrained by a structural element, affecting the earth pressure. In this paper, system insights on the mechanisms that control heave pressure are complemented by investigating the effect of small-strain stiffness. This is enabled by an extension of the Creep-SClay1S model that considers small-strain stiffness. It is shown that the effect of small-strain stiffness and its degradation with shear strains can be normalized by considering a two-dimensional time factor, a geometrical influence factor and the initial stiffness in the mid-point of the clay layer beneath the excavation. The results provide guidance on how consideration of small-strain stiffness affect effective heave pressure, in addition to factors such as excavation geometry, thickness of clay layer and normalised construction time. The results can be used in preliminary design and to complement project-specific analyses. Overall, the research contributes to a deeper understanding of excavation induced earth pressures and, thus, reduces the uncertainty and enables to optimize the volumes of construction materials.

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INTRODUCTION

Excavation invokes excess pore pressure changes which in low-permeable soils manifest as delayed heave during consolidation. If such a heave process is restrained by a structural element, an earth pressure will form. This pressure was referred to as the effective heave pressure, EHP, by [1]. During the design of Göta Tunnel in Gothenburg in 2001, earth pressure due to restrained heave against the bottom slab was considered and project-specific analyses were carried out by an expert investigation [2]. Such project-specific analyses are typically limited to major infrastructure projects as they are time-consuming.

Recently, an objective of a research project at Chalmers University of Technology [3] was to generalise the formation of EHP on underground structures in deep excavations in soft clay. The methodology utilised dimensional analysis. A parametric study, conducted with the Finite Element Method using the Creep-SClay1S model [4,5] was synthesised and presented in charts using appropriate non-dimensional parameter groups. The approach enables scaling of the results, making them applicable for a wide range of settings and projects.

In this paper, the system insights on the mechanisms that control effective heave pressure are complemented by investigating the effect of small-strain stiffness. This is enabled by a recent extension [6,7] of the Creep-SClay1S model. The results provide guidance on how consideration of small-strain stiffness affects EHP in addition to factors such as excavation geometry, thickness of clay layer, normalized construction time (time, coefficient of consolidation, drainage length).

CONSIDERED SOIL-STRUCTURE SYSTEM

Idealized geometry

The effect of considering small-strain stiffness in the assessment of effective heave pressure is studied for an idealized *Tunnel* geometry, see Figure 1. The slab and the retaining wall are modelled as rigid and the ground water table is located at the excavation base. This simplified scenario was complemented in [3] with a scenario involving the ground water table located 1m below ground surface level. The results were similar; hence the simplified scenario is re-analysed here to study the effect of considering small-strain stiffness. Similar to [3] the lower boundary and slab-clay interface are considered permeable, whereas the horizontal boundaries are impermeable.

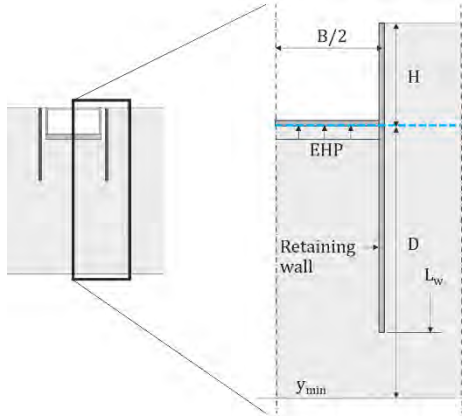


Figure 1 Idealized plane strain Tunnel geometry studied in this paper.

In [3] dimensional analysis was used to form dimensionless groups of the variables considered in the studied system. Furthermore, a two-dimensional time factor T_{2D} was introduced:

$$T_{2D} = [(t_{\text{exv.-slab}} \cdot c_v) / ((y_{\min} - H)/2)^2 + (2B/y_{\min})(t_{\text{exv.-slab}} \cdot c_h) / (B/2)^2] / 2 \quad \text{Eq. 1}$$

T_{2D} is also used in this paper to compile the results of the finite element simulations in which the following variables are varied: the time between the excavation and casting the slab $t_{\text{exv.-slab}}$ (5, 30, 90, 365 days), depth of excavation H (5 and 10 m, with length of the wall $L_w=15$ m and 25 m, respectively), width of the excavation B (10, 25, 50 m) and the distance to the bearing stratum $y_{\min}$ (25, 50, 75, 100 m).

Constitutive model

In this paper a formulation of the Creep-SClay1S model that accounts for non-linear degradation of stiffness with shear strain [7] is used, here referred to as Creep-SClay1S-s. [7] also accounted for strain accumulation under cyclic loading which is not included in the model formulation used in this paper. In Creep-SClay1S-s the secant shear modulus G decays from a reference small-strain shear modulus G_0 with shear strain ε_q as suggested by [6] according to:

$$G = G_0 \left[1 - \frac{\langle \varepsilon_q - \varepsilon_s \rangle}{A + B \langle \varepsilon_q - \varepsilon_s \rangle} \right] \quad \text{Eq. 2}$$

where ε_s is a threshold value set to 10^{-5} , A is a model parameter controlling the degradation and B defined according to:

$$B = \frac{1}{1 - \frac{G_{ur}}{G_0}} \quad \text{Eq. 3}$$

where G_{ur} is the stiffness determined by e.g. unloading-reloading in incremental loading oedometer tests. The small-strain feature of the model thus requires input of two additional parameters, A and $ssMP$ with $ssMP = \kappa^* / \kappa_0^*$ (i.e. a small-strain multiplier and the inverse of G_{ur}/G_0).

Parameter A is estimated to 0.001 based on data found in the literature, see Figure 2, using a plasticity index (PI) typical for Gothenburg clay i.e. in the range of 30-40% (see e.g. [8]) and with $ssMP=5$. Eq. 2 with $A=0.001$ and $ssMP$ in the range 5-10 also agree well with field and laboratory data presented by [11] on Bäckebol clay (just north of Gothenburg) where the PI was reported to be in the range 30-40%.

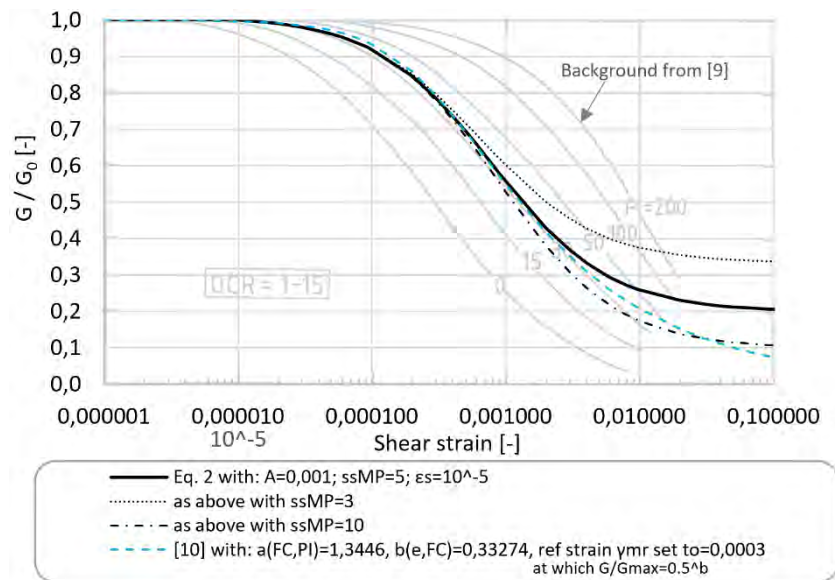


Figure 2. Illustration and estimation of parameters A and $ssMP$ based on plasticity index.

Taking $\kappa^*=0.015$ and $ssMP=5$ result in an attainable range of stiffness as illustrated in Figure 3 for the initial mean effective stress profile considered in this example and a Poisson's ratio of 0.20.

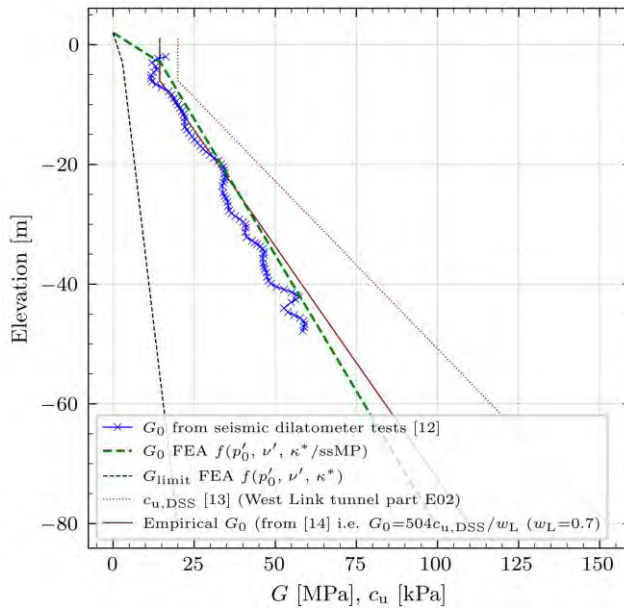


Figure 3. Comparison of measured and empirical G_0 in Gothenburg to initial and limiting stiffness (at large shear strains, from Eq. 2) in the finite element simulations.

The parameters of the Creep-SCLay1S-s model are summarised in Table 1 with additional input parameters vertical overconsolidation ratio $OCR=1.3$, earth pressure coefficient at-rest $K_0=0.6$ and permeability (hydraulic conductivity) $k_v=k_h=10^{-9}$ m/s.

Table 1. Creep-SCLay1S-s model parameter values.

Parameter	Description	Value
γ/γ' [kN/m ³]	Bulk/submerged unit weight	16/6
λ_i [-]	Modified intrinsic compression index	0.07
κ^* [-]	Modified swelling index	0.015
ν' [-]	Poisson's ratio	0.20
M_c [-]	Slope of CSL in triaxial compression	1.45
M_e [-]	Slope of CSL in triaxial extension	1.10
ω [-]	Rate of rotational hardening	200
ω_d [-]	Relative rate of rotational hardening due to dev. strain	1
a [-]	Rate of destructuration	8
b [-]	Relative rate of destructuration due to dev. strain	0.5
α_0 [-]	Initial anisotropy	0.57
χ_0 [-]	Initial amount of bonding	15
μ_i^* [-]	Modified intrinsic creep index	1/550
τ [days]	Reference time	1
K_0^{nc}	Earth pressure coefficient at primary loading	0.42
$ssMP$ [-]	Small-strain stiffness multiplier	5
A [-]	Shape factor for stiffness degradation	0.001

RESULTS

An example illustrating the degradation of stiffness in the model domain is presented in Figure 4 for two of the finite element simulations with $B=50$ m, $y_{\min}=75$ m. In Figure 4a $t_{\text{exv-slab}}=5$ days i.e. the excavation remained open for 5 days before the restraining slab was cast and in Figure 4b this time was 365 days. Hence, the degradation of stiffness has more time to develop (due to increasing shear strains). A detail in Figure 4 is the somewhat greater reduction in stiffness behind the retaining wall in Figure 4a compared to Figure 4b. This is explained by considering that in Eq. 2, the degradation of stiffness is solely dependent on the total deviatoric strain. By checking the value of ε_q in the simulations shown in Figures 4a and 4b it is confirmed that ε_q behind the wall was in fact reducing somewhat with time (from 5 to 365 days).

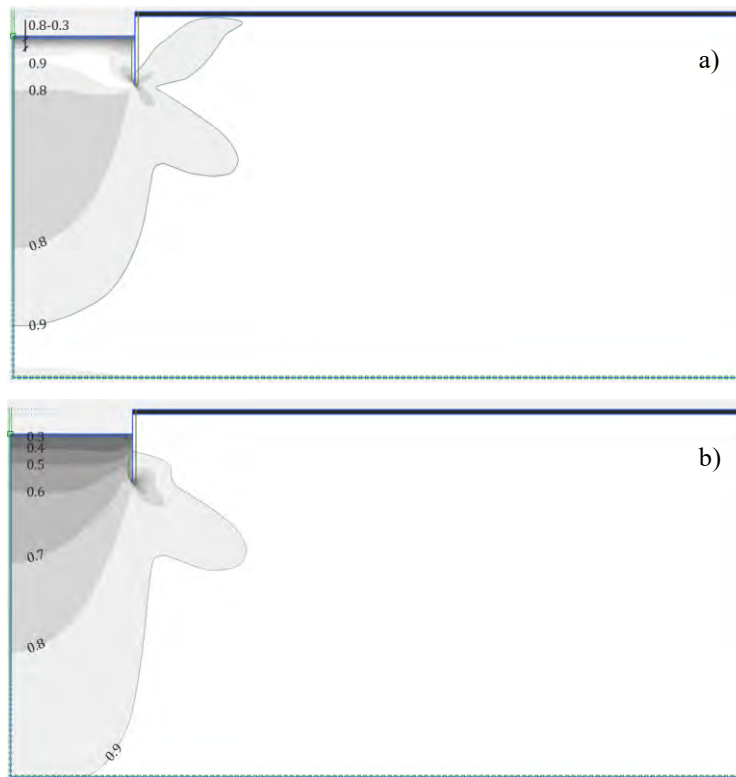


Figure 4. Example of stiffness degradation illustrated by G/G_0 at the time before application of slab for two of the simulations included in Figure 5: a) $t_{\text{exv-slab}}=5$ days, and b) $t_{\text{exv-slab}}=365$ days.

The results of the finite element simulations are compiled in Figure 5 presenting the effective heave pressure at the central location of the slabs, $\sigma'_{n,\text{centr}}$. A key to reading the colours and markers of Figure 5 is provided in

Table 2. The intention is not, however, to be able to separate the result of single simulations, rather it is to study the range of the normalised results.

Simulations with the small-strain Creep-SClay1S-s model are overlaid (red symbols) on the results from [3] (using the Creep-SClay1S and SClay1S models). The plotted results from the simulations with the creep models are maximum values occurring typically within 10 years after the activation of the slab in these simulations. In the case of narrow excavations, however, these maximum values reduce in the long-term due to background creep settlements causing “down drag”. For further details of the effect of background settlements see [3]).

The results are normalized with the *initial* stiffness in the middle of the clay layer (between the excavation base and the lower model boundary). For the Creep-SClay1S-s model κ_0^* is used to calculate the initial stiffness. For simulations with equal settings except for the small-strain stiffness formulation, the normalisation shifts the Creep-SClay1S-s results to the right due to the higher initial stiffness.

In Figure 5b, the simulations with $T_{2D} < 0.02$ are replotted to derive an influence factor μ_i (similar to estimation of initial settlement “influence factors” [15]) which describes that one part of the heave will be instant, and another part delayed, depending on the extent of the excavation in relation to the depth of the clay layer. (see further details in [3]).

In Figure 5c the results are normalized by μ_i . Figure 5c indicates that the results of Creep-SClay1S and Creep-SClay1S-s are comparable, however, when $T_{2D} > 0.1$ the Creep-SClay1S-s results are somewhat higher. This is explained by the fact that when the normalized time increases, e.g. if the excavation remains open for a longer time before the slab is activated, the stiffness is given more time to degrade. Hence normalisation using the initial small-strain stiffness shifts the results somewhat excessively to the right, most notably when $T_{2D} > 0.1$. The agreement of the results normalized by μ_i and T_{2D} is still considered satisfactory, which provides further credibility to the presented charts as well as partial validation of the recent implementation of non-linear stiffness in the Creep-SClay1S model.

Table 2. Key to colours and markers for in Figure 5.

Parameter	Variation	Unit
$y_{\min}$	25, 50, 75, 100	m
B	10($\square$), 25($\triangleleft$), 50($\diamond$)	m
H	5, 10 ^{a)}	m
$t_{\text{exc.-slab}}$	5, 30, 90, 365	days
Red symbols = Creep-SClay1S-s, other Creep-SClay1S and SClay1S.		
^{a)} half-filled symbols		

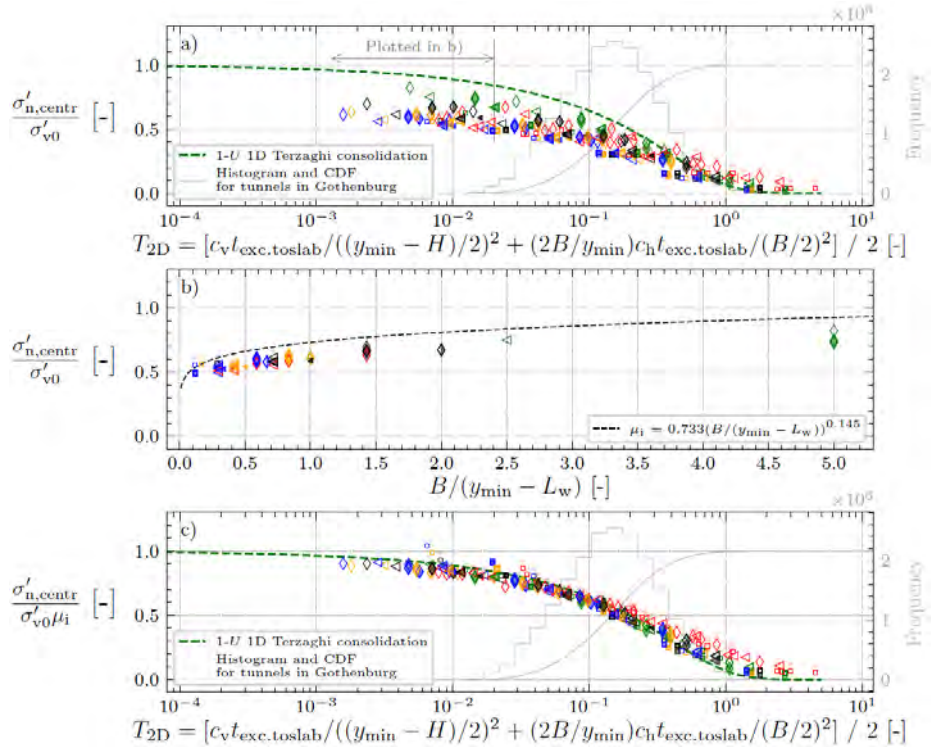


Figure 5. Data from [3] complemented with results of small strain stiffness and histogram and cumulative distribution function (CDF) for fictive tunnels in Gothenburg: a) EHP-ratio at central part of slab plotted against T_{2D} , b) geometrical influence factor μ_i , and c) EHP-ratio normalised by μ_i and plotted against T_{2D} .

A histogram and cumulative distribution function (CDF) illustrating typical values T_{2D} for a tunnel construction in Gothenburg is included in Figure 5. The histogram was constructed drawing 10 random values from each of the ranges presented in Table 2 (except for 20 values for k). The CDF tells us that the EHP-ratio is in the range 0.3--0.5 for the median T_{2D} value. The recommendation is, however, that the engineers use charts such as Figure 5 to estimate the magnitude of the EHP for the specific project at hand.

Table 3. Ranges used to construct the histogram and cumulative distribution function for typical tunnels in Gothenburg.

Variable	Range	Unit
H	5--15	m
B	20--40	m
y_{min}	30--100	m
E at top of clay layer	10--20	MPa
E_{inc}	0.5--2.0	MPa/m
$k_v = k_h$	10^{-10} -- 10^{-9}	m/s
$t_{exc.-slab}$	90--365	days

CONCLUSIONS

When excavating in low-permeable soils, a delayed heave process is triggered that, if restrained, will introduce an earth pressure on the restraining structural element. This earth pressure is referred to as the effective heave pressure, EHP, and has previously been studied in detailed project specific analyses for Göta Tunnel in Gothenburg. In this paper, the effect of considering small-strain stiffness on EHP in deep excavations in soft clay was examined to complement recently published results from a finite element parametric study synthesized into non-dimensional design charts [3]. The effect of small-strain stiffness was studied by the recently developed Creep-SClay1S-s model.

The results show that by considering a two-dimensional time factor T_{2D} and a geometrical influence factor, the results of the parametric study can be normalized by the initial soil stiffness in the clay layer even when considering small-strain stiffness and its degradation due to accumulated shear strains.

The results provide guidance on how consideration of small-strain stiffness affect effective heave pressure in addition to factors such as excavation geometry, thickness of clay layer, normalized construction time (time, coefficient of consolidation, drainage length). The results contribute to a deeper understanding of excavation induced earth pressures and, thus, reduce uncertainty and enable to optimise the volumes of construction materials.

The model parameters describing the small-strain stiffness and its degradation were estimated based on empirical data found in the literature. Hence, it is recommended to study the degradation of small-strain stiffness for soft Swedish clays further, to attain representative values for the small-strain stiffness parameters.

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GENERATION OF EXCESS PORE PRESSURES FROM DTH DRILLED PILES

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KEYWORDS

Piles, Slope stability, DTH drilling, Pore pressures, Clays

ABSTRACT

Piles are frequently installed near slopes in urban environments where the factor of safety for slope stability collapse is low. The installation process can result in excess pore pressure generation and soil displacement which can reduce the stability of the slope and potentially trigger a slope collapse, or horizontal displacements in the soil mass, but there are large differences between the pore pressure generation from various piling methods. Drilled steel piles with down-the-hole (DTH) hammers are an efficient method to penetrate the soil strata and drill the piles into the bedrock. This installation method is considered much safer compared to displacement piles because of the less pronounced soil displacement. Several hammer technologies are available: most are powered by air or by water, in which the hammering and rotating motion is transferred to the drill bit. In the current study, a field test of air- and watered DTH hammers is elaborated, and the differences in the soil response measured and compared previous experience.

INTRODUCTION

Piles are frequently the preferred method for foundations in soft soils and transferring the structural loads from the ground surface to more compact soil layer. In construction practice, piling is frequently executed close to existing slopes or nearby natural slopes. The soil mass displacement and generation of excess pore pressures in the ground can thereby increase risk of slope collapse in nearby slopes, resulting in a significant hazard during construction.

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Several studies of excess pore pressures and soil displacement resulting from pile driving have been presented. The increase in the effective stress state around a bridge abutment was elaborated in Bjerrum and Johannessen (1961). Several methods to reduce the risk of excess pore pressures have been developed, among them prefabricated drains mounted on the piles, (Holtz and Boman, 1974). The risk of landslides resulting from local disturbance in the soil was initially developed by Bernander (2011), discussing several large landslides where piling was assumed to be the triggering agent for the slope collapse.

Alternative methods such as drilled piles have been developed to reduce the risk of excessive ground displacement and excess pre pressures. Drilled steel piles with down-the-hole (DTH) hammers are an efficient method to penetrate the soil strata and drill the piles into the bedrock, which are suitable for soil conditions in which the bedrock can provide bearing capacity of the pile base, (Larisch, 2012). Considering the risk of excess pore pressure, drilled piles have been proven to be a much safer piling method compared to displacement piles because of the less pronounced soil displacement. Several hammer technologies are available: most are driven by air or by water, in which the hammering and rotating motion is transferred to the drill bit. The air-driven technology requires less equipment but can transfer a significant air pressure into the soil during drilling, which can increase the excess pore pressures in the soil and also result in soil displacement. Water-driven hammers cause less pressure generation in the soil due to much lower compressibility of the water, which results in a much lower excess pressure released into the soil during the hammer cycle in the DTH hammer.

A comparison between the DTH hammer methods was demonstrated by Degago and Thakur (2018), when an air-driven hammer was changed to a water driven hammer after the clay fractured during drilling. A study of the mechanism during drilling in sand and a study of different methods was developed by Lande et al. (2021, 2024), in which it was demonstrated that the water-driven method in most cases resulted in less soil disturbance and settlement. However, this study focused mostly on drilled anchors, which are different from piles because of the larger inclination from the vertical and the smaller diameter of the anchor drill bit. In the current study, and series of field tests on drilled steel piles with air driven and water driven DTH hammers in clay are discussed (Ahlund and Ögren, 2016), and the development of the accumulated pore pressure development elaborated.

METHODOLOGY

A series of field tests of drilled piles were carried out during a large infrastructure development project south of Stockholm in the municipality of Haninge. The soil profile consisted of 8-9 m of soft marine clay located on top of 2-4 m moraine till on top of Precambrian bedrock. In the pile design, the

piles were drilled into the bedrock and the pile bearing capacity was verified by dynamic pile load tests.

A comparative study between the different methods was executed with the field test setup cluster shown in Figure 1. Two symmetrical test setup clusters were installed with 17 m distance between the pore pressures gauges, with sensor 1 located in by the air-driven (AD) cluster and sensor 2 in the water driven (WD) cluster. For each drilling method, two 140 mm diameter steel piles were drilled with 2 m c/c distance, according to Figure 1. Figure 2 shows the drilling of a steel pile with an air driven DTH hammer. Measurements were carried out with a pore-pressure gauge located at 5 m depth, as well as with settlement gauges and settlement screw. Only the pore pressure measurements are discussed in the current study.

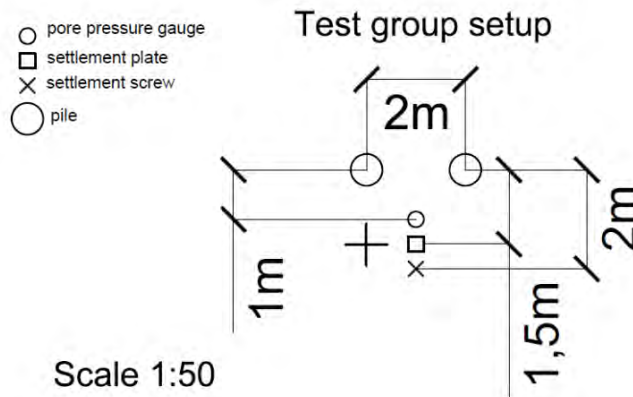


Figure 1. The test set-up for each piling method, consisting of the piles and the measurements devices.



Figure 2. Drilling of a steel pile with an air driven DTH hammer.

RESULTS

The piles drilled with the air driven DTH hammer in the AD-cluster was tested first, followed by the piles drilled with the water driven DTH hammer in the WD-cluster. Figures 3 and 4 show the measurements of the total pore pressure and the time when the piles were drilled. Piles 1 and 2 are the drilled by the air-driven DTH method and Piles 3 and 4 by the water-driven method.

Figures 3 and 4 show the recorded total pore pressure and the time gap when the piles were drilled in the AD- and WD-cluster respectively. The excess pore pressures recorded in the AD and the WD-cluster by the air-driven DTH hammer and the water driven DTH hammer are approximately 14 kPa and 3 kPa. Hence the excess pore pressure level from the air-driven DTH hammer around 3-5 times from the water-driven DTH hammer.

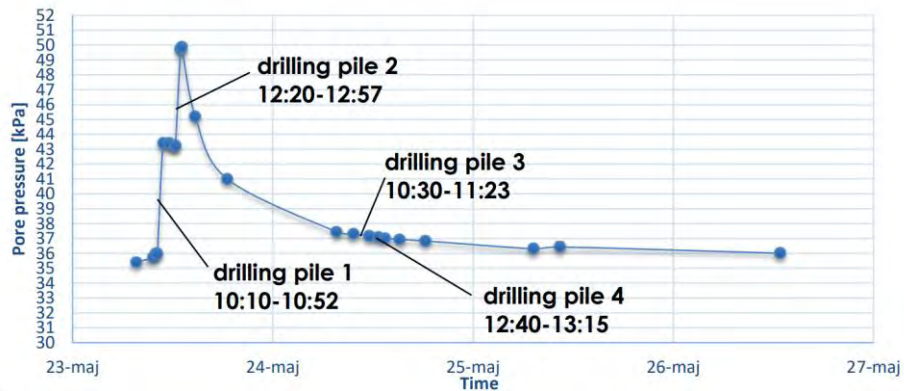


Figure 3. The total pore pressures recorded at the pore pressure gauge 1 located in the AD-cluster.

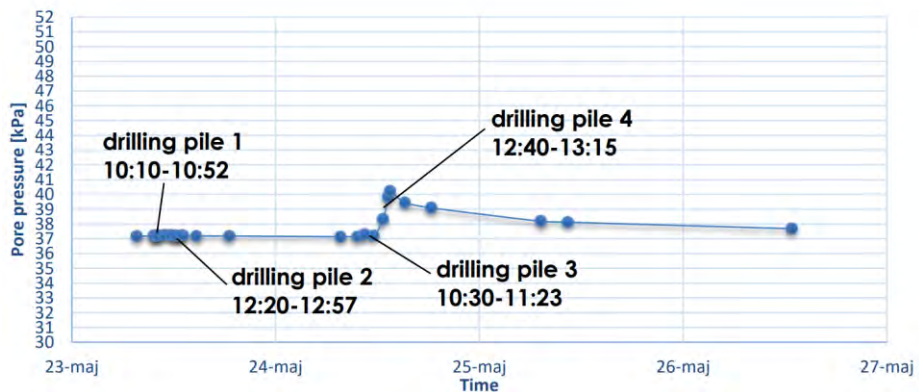


Figure 4. The total pore pressures recorded at the pore pressure gauge 2 located in the WD-cluster.

CONCLUSIONS

A field test has been carried out in typical soil conditions in Eastern Sweden to compare the generation of excess pre pressures from piles drilled with air and water driven DTH hammers. The results show that the difference in excess pore pressures between the air- and water driven DTH method are at a ratio of around 4-5, which corresponds to the ratios demonstrated by Lande et al. Hence, the water driven DTH methods should be preferred when there is a risk of landslides close to the area of piling.

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IMPLEMENTATION OF A NEW THERMO-VISCOPLASTIC SOIL MODEL USING FENICS PLATFORM

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KEYWORDS

FEM, THM coupling, Thermal viscoplasticity, FEniCS.

ABSTRACT

The heating and cooling processes (e.g., imposed by energy geostructures) may cause undesired displacement of soil, which in turn may impact the long-term stability of the geothermal structures. This is true especially those installed in sensitive clays due to the high dependence of their creep behaviour on the temperature. This paper presents a numerical implementation algorithm for an enhancement of an existing hydro-mechanical model into a Finite Element code based on FEniCS platform. It also includes a validation example against experimental data of thermal oedometer tests. The quantitative comparison to experimental results supported the suitability of the framework.

EXAMPLE INTRODUCTION

The rapid expansion of the energy geotechnics sector has driven the wide recognition of the influence of temperature on the behaviour of geomaterials. Thus, it is generally acknowledged that the volumetric behaviour, creep rate, shear strength and pore water pressure of clay are temperature dependent. Given the complexity of the thermal behaviour, it is necessary to enhance the understanding of the fundamental mechanisms of the coupled Thermo-Hydro-Mechanical (THM) behaviour to satisfactorily predict the response of the geostructures concerned.

The response of geostructures, has been recently successfully simulated [1], [2], [3], thanks to the development of THM framework [4], [5], [6], [7]. Most of these numerical codes are not easy to access and vary in complexity, which poses challenges to their applications to specific problems based on the needs of the user.

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In addition, modelling such intricate physical processes requires robust constitutive relationships. A variety of constitutive models focusing on the temperature dependent yield surface (e.g. [8], [9], [10]) have been developed. However, to date most constitutive models were developed in the context of nuclear waste barriers involving significant temperature variation, whereas the creep mechanism of sensitive clay under modest temperature changes, commonly the case in Canada and Scandinavia, is not yet well understood. In this respect, informed by a systematic series of thermal oedometer tests, Li [11] proposed a thermo-viscoplastic soil model, in which thermo-mechanical behaviour is incorporated using a temperature and creep rate dependent yield surface.

Following the work of [12] [13] , this paper aims to provide a reliable computational framework combining a THM coupling and aforementioned thermal constitutive model [11] using the FEniCS platform [14] .

GOVERNING BALANCE EQUATIONS

This section briefly presents the governing balance equations regarding mass, energy and mechanics with assumptions: (1) the porous geomaterial consists of three phases - liquid (l), gas (g) and solid (s); (2) solid phase does not dissolve or sublimate while water may transform between vapour and liquid; (3) dry air is not considered.

a. Water mass balance equation

The mass balance equation for water component is presented by:

$$\begin{aligned}
 & [n(\rho_w^l - \rho_w^g) \frac{\delta S^l}{\delta T} - (1 - n)(S^l \rho_w^l + S^g \rho_w^g) \beta_{sT} - n S^l \beta_{wT} \rho_w^l \\
 & \quad + n S^g \frac{\rho_w^g}{T^2} (4974 + \frac{M_w S}{R \rho_w^l})] \frac{\partial T}{\partial t} \\
 & \quad + [n S^l \beta_{wp} \rho_w^l + n S^g \frac{\rho_w^g M_w}{\rho_w^l R T} - n(\rho_w^l - \rho_w^g) \frac{\partial S^l}{\partial s}] \frac{\partial u_w}{\partial t} \\
 & \quad + (S^l \rho_w^l + S^g \rho_w^g) \frac{\partial \varepsilon_v}{\partial t} + \nabla \cdot [\frac{\rho_w^l K_w}{\rho_w^l g} (\nabla u_w + \rho_w^l g)] = 0
 \end{aligned} \tag{1}$$

where n is the soil porosity, ρ_w^l is the water density in a liquid phase dependent on the pore pore-water pressure u_w and the temperature T , ρ_w^g is the vapour density in a gas phase. S^l and S^g are the degree of saturation for liquid and gas, respectively, where S^l is assumed to be a function of suction (s) for unsaturated soil [15]. β_{sT} and β_{wT} are the coefficients of volumetric thermal expansion in solids and water respectively. M_w is the molar mass of vapour and R is the universal gas constant. β_{wp} is the coefficient of water

compressibility, ε_v is the volumetric strain. K_w denotes hydraulic conductivity of liquid water. g is gravity acceleration.

b. Energy balance equation

The energy balance equation considering the latent heat of vaporization and heat due to conduction and convection can be expressed as:

$$\underbrace{\frac{\partial \Phi_h}{\partial t}}_{\text{heat storage}} + \underbrace{L Q_w^g}_{\text{latent heat of vaporization}} + \underbrace{\nabla \cdot \mathbf{q}_h}_{\text{heat due to conduction and convection}} = 0 \quad (2)$$

with

$$\Phi_h = [(1 - n)\rho^s c_s + nS^l \rho_w^l c_w^l + nS^g \rho_w^g c_w^g](T - T_0) \quad (3)$$

$$Q_w^g = -(1 - n)S^g \rho_w^g \beta_{sT} \frac{\partial T}{\partial t} + S^g \rho_w^g \frac{\partial \varepsilon_v}{\partial t} - n \rho_w^g \frac{\partial S^l}{\partial t} + nS^g \frac{\partial \rho_w^g}{\partial t} \quad (4)$$

$$\mathbf{q}_h = -\lambda_T \nabla T + \frac{\rho_w^l c_w^l K_w}{\rho_w^l g} (\nabla u_w + \rho_w^l g)(T - T_0) \quad (5)$$

where λ_T is the thermal conductivity. T_0 is the reference temperature with a value of 293.16 K for this research. L is the latent heat of water vaporization.

c. Mechanical balance equation

The mechanical balance equation is given by:

$$\nabla \cdot \boldsymbol{\sigma}^{\text{tot}} + \rho^b \mathbf{g} = 0 \quad (6)$$

where $\boldsymbol{\sigma}^{\text{tot}}$ is the total stress tensor, $\rho^b \mathbf{g}$ is body force that typically denotes self-weight of material related to its bulk density.

The corresponding weak formulation of the balance equations (Eqs. (1), (2) and (6)) can be derived by multiplying the corresponding independent test functions $\mathbf{X} = [\delta u, \delta u_w, \delta T]$ and integrating over the spatial domain Ω , where the symbols δu , δu_w and δT are the variations in nodal displacements, water pressure and temperature, respectively. The full details are described in [12].

CREEP-SCLAY1ST

Stress integration relies largely on the constitutive model that describes the stress strain response of material. This section summarizes key aspects of an enhanced version of Creep-SCLAY1S model [16], namely Creep-SCLAY1ST [11], which is capable of capturing the bonding-dependent

thermal softening and hardening of the natural sensitive clay. In what follows, for the sake of simplicity, the model will be discussed assuming triaxial stress state, however, the actual implementation is done with the generalized version.

Following the elasto-plasticity theory, the total strain rate $\dot{\epsilon}$ in Creep-SCLAY1ST that accounts for thermal effects is expressed as follows:

$$\dot{\epsilon} = \dot{\epsilon}_\sigma^e + \alpha_T \dot{T} \delta_{ij} + \dot{\epsilon}^{vp} \quad (7)$$

where $\dot{\epsilon}_\sigma^e$, $\alpha_T \dot{T} \delta_{ij}$ and $\dot{\epsilon}^{vp}$ are vectors of mechanical elastic strains, thermal expansion and viscoplastic strain, respectively. α_T is the thermal expansion coefficient. δ_{ij} is the vector $\{1, 1, 1, 0, 0, 0\}$ indicating that the model assumes that temperature contributes to volumetric strains only.

Similarly to Creep-SCLAY1S model, three surfaces with similar sheared ellipse in triaxial stress plane, namely the Normal Compression Surface (NCS), the Current Stress Surface (CSS) and the Imaginary Intrinsic Compression Surface (ICS) [17][18], are used to represent the stress state and bonding amount of the model (Figure 1). Their sizes are defined by the intersection of the vertical tangent to the ellipse with the mean effective stress axis p'_m , p'_{eq} and p'_{mi} , respectively - e.g. Eq (8):

$$p'_m = p' + \frac{(q - \alpha p')^2}{(M^2(\theta_\alpha) - \alpha^2)p'} \quad (8)$$

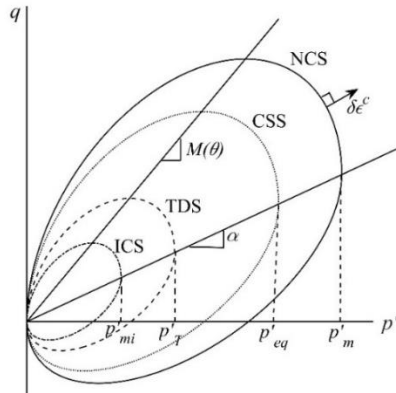


Figure 1 Description of the main characteristics of Creep-SCLAY1ST in the p' - q plane.

where p' is the mean effective stress, q is the deviatoric stress, α is a state variable related to the inclination of the yield surface, $M(\theta_\alpha)$ is Lode angle θ_α dependent stress ratio at critical state (details see [19]). The NCS and ICS can be linked by the amount of natural bonding (χ):

$$p'_m = p'_{mi}(1 + \chi) \quad (9)$$

A novel development of Creep-SCLAY1ST relates to the introduction of an additional temperature-dependent surface – TDS, by incorporating the effect of the temperature (T) and the initial amount of natural bonding (χ_0):

$$\frac{p'_T}{p'_{mi}} = \left(\frac{T}{T_{ref}}\right)^{-(a_T\chi_0 + b_T)} \quad (10)$$

where a_T and b_T are material specific thermal factors. Eq (9) is therefore updated with p'_T to obtain a temperature dependent preconsolidation pressure:

$$p'_m = p'_T(1 + \chi) \quad (11)$$

Such an update induces the shrinking of the NCS during heating (i.e., thermal softening) and its expansion during cooling. The Creep-SCLAY1ST model comprises three hardening laws, in which isotropic hardening (Eq. (12)) and destructuration (Eq. (14)) will affect the size of TDS and NCS, and rotational hardening (Eq. (13)) will affect the orientation of TDS, CSS and NCS. It is worth noting that isotropic hardening evolves as a function of both volumetric creep strains ($\dot{\epsilon}_v^c$) and temperature ($\dot{T}$) rates, while the rest of the hardening laws remain the same as that for Creep-SCLAY1S model:

$$\dot{p}'_T = \frac{p'_T}{\lambda^* - \kappa^*} \dot{\epsilon}_v^c - (a_T\chi_0 + b_T)p'_T \left(\frac{\dot{T}}{T}\right) \quad (12)$$

$$\delta\alpha = \omega \left[\frac{3\eta}{4} - \alpha \right] \langle \delta\epsilon_v^c \rangle + \omega_d \left[\frac{\eta}{3} - \alpha \right] |\delta\epsilon_q^c| \quad (13)$$

$$\dot{\chi} = -\xi\chi(|\dot{\epsilon}_v^c| + \xi_d|\dot{\epsilon}_q^c|) \quad (14)$$

where λ^* is the modified intrinsic compression index, and κ^* is the modified swelling index. η is the stress ratio and ω and ω_d are model constants associated with the evolution of anisotropy. ξ and ξ_d are model parameters related to the bond degradation. The volumetric ($\dot{\epsilon}_v^c$) and deviatoric ($\dot{\epsilon}_q^c$) creep strain rates in Eqs. (15) and (16) are calculated using the concept of a viscoplastic multiplier $\dot{\Lambda}$ expressed by Eq. (17) [19]:

$$\dot{\epsilon}_v^c = \dot{\Lambda} \frac{\partial p'_{eq}}{\partial p'} \quad (15)$$

$$\dot{\epsilon}_q^c = \dot{\Lambda} \frac{\partial p'_{eq}}{\partial q} \quad (16)$$

with

$$\dot{\Lambda} = \frac{\mu_i^*}{\tau} \left(\frac{p'_{eq}}{p'_m} \right)^\beta \left(\frac{M_c^2 - \alpha_{K_0^{nc}}^2}{M_c^2 - \eta_{K_0^{nc}}^2} \right) \quad (17)$$

and

$$\beta = \frac{\lambda^* - \kappa^*}{\mu_i^*} \quad (18)$$

where μ_i^* is the intrinsic creep index, and τ is the reference time.

NUMERICAL IMPLEMENTATION AND VALIDATION

The coupled THM balance equations combined with the Creep-SCLAY1ST model were scripted using Python and ran using the open source FEniCS platform. The following section presents the validation of the model results against experimental observations from thermally controlled oedometer tests.

d. Numerical model

The proposed framework was verified against the experimental test results on Utby clay reported in [11]. A three-dimensional finite element model was established to simulate the oedometer tests, as illustrated in Figure 2. The geometry of the FE model with a height of 18 mm and diameter of 50 mm, is discretized with 496 tetrahedra elements with 4 Gauss points associated with each element.

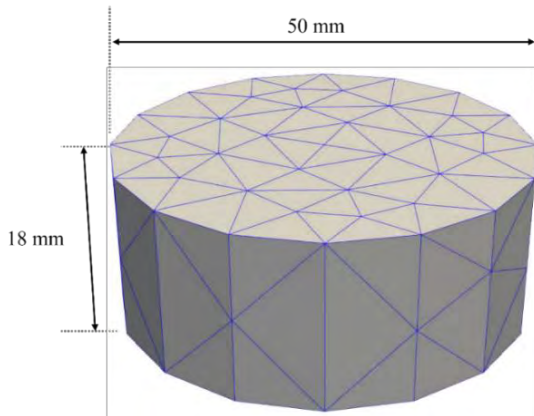


Figure 2 Representative mesh used in the simulations.

The imposed boundary conditions in the model are as follows:

- The displacement boundary conditions include the restriction of movements in all directions at the bottom, while the displacements perpendicular to the cylindrical boundary are also set to zero. The top surface is set to move freely.
- The hydraulic boundary conditions are prescribed as impermeable at the top and the cylindrical boundaries of the mesh, whereas no change in the pore water pressure from the initial condition is set to the bottom surface for a drained condition.
- A closed form of temperature boundary condition is deployed on bottom and cylindrical boundaries of the mesh, while a time-dependent temperature is prescribed to the top boundary to simulate the heating or cooling of the sample. Note that the temperature imposed in the top boundary can spread to the whole sample within a short time owing to the small size of the sample.

The mechanical parameters of Creep-SCLAY1ST model in Table 1 were adopted for the numerical simulation. A total of 18 model parameters are involved. 14 of them, which were previously introduced in the Creep-SCLAY1S, have been kept identical to the ones calibrated and reported following [16]. The 4 thermal parameters are then derived according to the experimental fit.

Table 1. Parameters for Utby clay.

e_0	v'	OCR	λ^*	κ^*	M_c	M_e
1.97	0.2	1.2	0.0862	0.0083	1.45	1.1
μ^*	T (day)	α_0	ω	ω_d	χ_0	ξ_d
0.004	1	0.5	40	0.9	6	0.3
ξ	a_T (1/K)	T_{ref} (°C)	a_T	b_T		
9	3.2e-5	7	8.5e-3	5.1e-3		

e. Loading scenarios

Herein, 4 tests from Li's experiments [11], including two loading paths O_1 and O_2 under either a cooling or heating process, on intact samples with an initial void ratio equal to 1.97 from 6 m depth were selected to check the capabilities of the proposed model, as shown in Figure 2. All samples were initially loaded to the estimated in-situ vertical effective stress level of 45 kPa under temperature T_1 , and then the loading procedure shown in Figure 2

proceeded by bringing the samples to 6 different stress levels σ'_v . The temperature of the sample also varied at each stress level. Details of the selected tests are given in Table 2, where the test step, duration of each step, vertical stress level and temperature are provided.

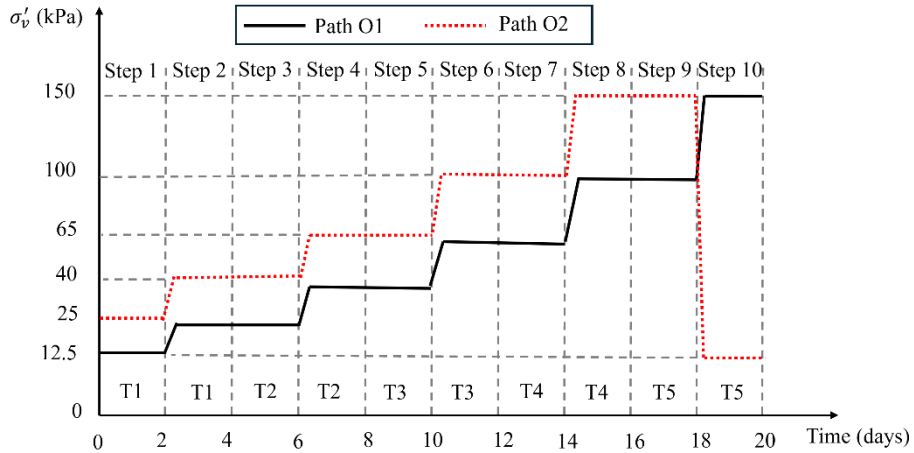


Figure 2 Schematic of the test procedure.

f. Results

The curves regarding void ratio (e) against vertical effective stress (σ'_v) in logarithm obtained from numerical analysis with Creep-SCLAY1ST and Creep-SCLAY1S models and experimental observation are plotted in Figure 3a and c. It is apparent that the overall mechanical behaviour is reasonably well captured by Creep-SCLAY1ST and Creep-SCLAY1S models for the two loading paths under heating and cooling processes. However, the Creep-SCLAY1S model seems unable to reproduce the variation of preconsolidation pressure under thermal loading. For Creep-SCLAY1ST model, as expected, an increase trend of the preconsolidation pressure was observed in the cooling process, while the heating process causes the decrease in preconsolidation pressure.

Table 2. Selected tests for numerical simulations.

Step	Time (days)	Temperature T (°C)			Stress σ'_v (kPa)	
		Cooling	T	Heating	O ₁	O ₂
1	2	5	T1	25	12.5	25
2	2	5	T1	25	25	40
3	2	10	T2	20	25	40

4	2	10	T2	20	40	65
5	2	15	T3	15	40	65
6	2	15	T3	15	65	100
7	2	20	T4	10	65	100
8	2	20	T4	10	100	150
9	2	25	T5	5	100	150
10	2	25	T5	5	150	12.5

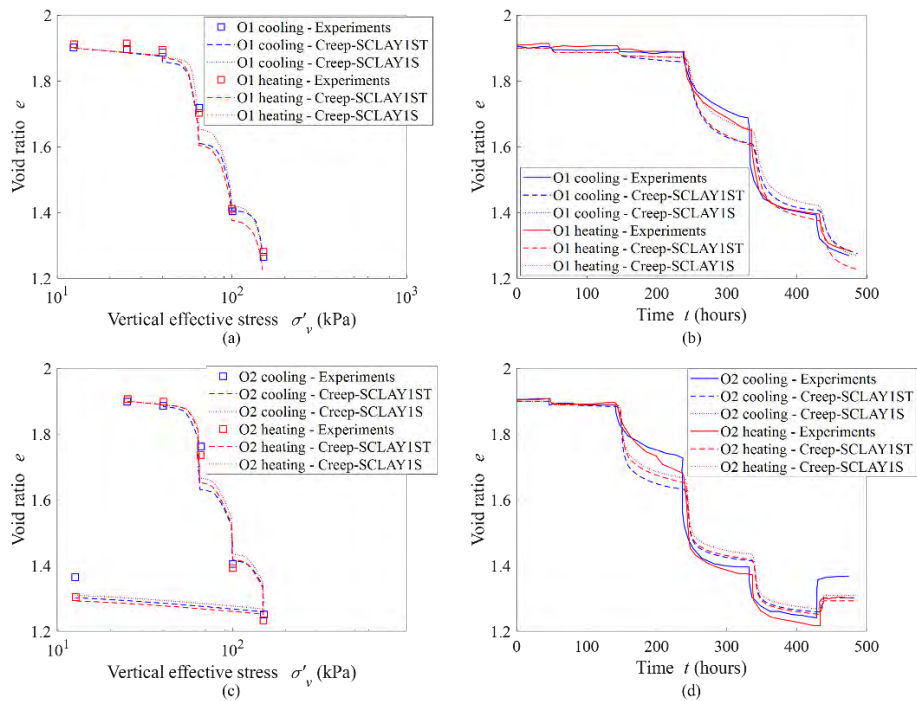


Figure 3 Compression curves (a) for loading path O1 and (c) for loading path O2 and the time-void ratio curves (b) for loading path O1 and (d) for loading path O2. note O1 and O2 cooling/heating Creep-SCLAY1S curves overlapped.

The variations of void ratio in Figure 3b and d again show overall encouraging results regarding the performance of Creep-SCLAY1ST and Creep-SCLAY1S models when compared with experimental results, although a slight overestimation of the decrease in void ratio when samples are loaded beyond their respective preconsolidation pressures (Step 4).

It is worthwhile to investigate the influence of temperature on deformation of the clay under constant stress level. Figure 4 shows the variation of void ratio under vertical stress of 65 kPa at step 7 in loading path O₁, where the intact clay samples with identical stress histories are thermally loaded from an equal temperature for heating (15 °C to 20 °C) and cooling (15 °C to 10 °C). The experimental results indicate that larger compression is induced by heating than by cooling due to the heating softening. Such phenomenon is better reproduced by Creep-SCLAY1ST, while no difference was observed between heating and cooling in the results by Creep-SCLAY1S.

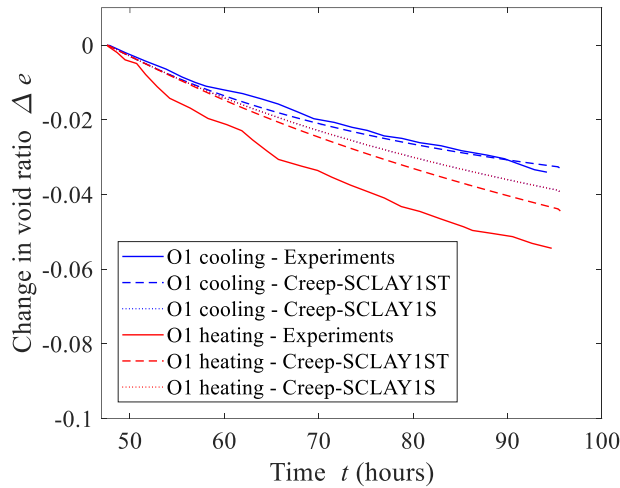


Figure 4 Change in void ratio under temperature change with 5°C under stress level of 65 kPa for O1 stress paths. note O1 cooling/heating Creep-SCLAY1S curves are overlapped.

CONCLUSIONS

This paper presented an implementation of a new thermo-viscoplastic constitutive model into a coupled THM framework based on FEniCS platform. The proposed constitutive model improves the thermal performance of Creep-SCLAY1S by introducing an additional temperature and creep rate dependent surface to keep track of the variation of preconsolidation pressure induced by the thermal loading. The model results were validated by simulating results of oedometer tests on soft soil under combined mechanical and thermal loading conditions. The results highlighted the potential of the model and the computational framework to accurately predict the thermal response of intact sensitive clay, as well as to be employed in the simulation of thermal related geotechnical problems.

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IMPLICATION OF 3D SLOPE STABILITY ANALYSES FOR SENSITIVE CLAY SLOPES

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KEYWORDS

Slope stability, sensitive clays, 3D-analysis, safety factors

ABSTRACT

Stability analyses in sensitive clays is commonly carried out assuming a 2D plane strain condition and limit equilibrium (LE) approach using e.g. the method of slices or the finite element method (FEM). However, there may be instances where these key assumptions may not be representative. In such cases, it is important to quantify the 3D effect on slope stability and to make considerations for the post peak reduction in undrained shear strength of highly sensitive clays. The current Norwegian practice uses different approaches to address these two aspects. This work investigates current approaches adopted to address 3D effects in the current Norwegian practice. Implication of these is illustrated using 2D and 3D numerical analyses. An emphasis is made on highlighting aspects that play key roles in analyses results. Typical variations of key parameters are used to make basis for sensitivity analyses. Different slope widths were analysed, and results demonstrated that the width of the 3D model controls the width of the failure surface and thus also the factor of safety. An attempt is also made to back calculate a failure slope using a strain softening material model in 3D FEM. Sensitivity study showed that the spatial variability of the soil strength has significant effect on the calculated safety factor. The work discusses current challenges and limitations that could be encountered in analysing natural slope with current approach for 3D. Implications of these results considering current design code requirements is discussed.

INTRODUCTION

Slope stability is a commonly encountered problems in geotechnical engineering practice. Stability analyses is commonly carried out based on two-dimensional (2D) limit equilibrium methods (LEM) with the method of slices. A key assumption in these approaches is that the stability analyses is solved assuming a plane strain condition (2D) and an equal level of strength

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mobilization along a pre-selected slip surface using the method of slices. Such approach is commonly adopted in practice to perform slope stability analysis. Still, there are several instances encountered in projects where 2D assumptions may not be representative for various reasons such as the geometry of the slope, soil layering, loading conditions as well as extent of planned construction activity. In such cases, it would be appropriate to do a three-dimensional (3D) modelling of the problem. It is particularly important to take into account 3D effects in back analysis of slopes to avoid overestimation of shear strength (e.g. Stark and Eid 1998; Akthar 2011; Jostad et al. 2021). There has been extensive research going on developing ways to perform 3D slope stability analysis based on three broad categories of LEM, limit analyses methods and stress-strain methods (see summary in Akthar 2011; Zhang 2013, Kumar et al. 2023). In Norway and Sweden, there has been recent works looking at the effect of 3D slope stability problems (e.g. Jostad and Lacasse, 2015; Solli 2020; Edstam 2021; Jostad et al. 2021).

There is huge interest towards incorporating 3D effects into designs. This work investigates current approaches adopted in the Norwegian practice. Implication of these is illustrated using numerical analysis involving 2D LEM and FEM analyses as well as 3D FEM analysis. Further emphasis is made on aspects that play key roles in the analyses results. The work aims to investigate the current state of the practice and point key and relevant aspects related to use of 3D effects on sensitive clay slopes.

MOTIVATION AND PROBLEM FORMULATION

As one of the main regulatory authority bodies in Norway, the Norwegian Public Roads Administration (NPRA) is, among others, responsible for issuing acceptable partial factors to be used in design of slopes associated road infrastructures. The current safety requirements are developed and calibrated using 2D approach with a view to be used in 2D analysis. However, with emergence of powerful computers and technological advancements, there is increasing demand and development towards use of three-dimensional tools in Geotechnical engineering (e.g. BIM-models and Digital Twins). As a result, there is also interest on development and use of calculation tools for 3D slope stability analysis which affects the resulting safety factors. Therefore, it is important to investigate implication of this with a view to complying to current level of safety requirements when performing 3D analysis.

Independent of 2D or 3D modelling, an important feature of Norwegian sensitive clays is their strong strain-softening behaviour, i.e. post-peak shear strength reduction with deformation. Currently, this is addressed by requiring higher safety factors, for strain-softening materials, along with 2D LEM stability analysis tools (Fornes and Jostad 2017, NVE 2020). However, this

effect is not thoroughly investigated and established for a 3D case. Therefore, it is important to address it with respect to 3D modelling.

In the current practice, there is growing tendency to benefit from higher calculated 3D safety factors while adhering to the safety requirements calibrated for use with 2D analyses (Degago, 2019). When this is used for sensitive clays, another challenge is that the effect of strain-softening is not quantified for a 3D case. These effect needs thorough assessment and to address this, a collaborative research project was initiated by NPRA and NGI since 2020 (NGI, 2022).

CURRENT INDUSTRY PRACTICE TO ACCOUND 3D-EFFECT

In the current Norwegian practice, 3D-effect ($F_{3D} = F_{3D}/F_{2D}$) is mainly calculated using two methods. The first one is use of a “3D Side Friction” function in GeoSuite Stability and the second one is using the finite element package Plaxis 3D (PLAXIS V21). In this section, the two approaches are briefly investigated numerically from application point of view into real projects.

GeoSuite 3D Side Friction

The approach adopted in the GeoSuite Stability is based on recommendation by the Swedish commission on slope stability (Skredkommissionen, 1995) which in turn bases itself on the work by Gens et al (1988). This method is developed assuming a simplified slope with homogeneous, isotropic, and purely cohesive soils. It further assumes rotational slides with a cylindrical central slip surface enclosed with plane sides; and this analytical expression is extended to consider curved ends. Another key assumption is that the end surfaces are strong enough to provide support for the end failure surfaces. The simplified theoretical development by Gens et al. (1988) provided the basis for adaptation by Skredkommission (1995) as given in Eqn. [1] with a later suggestion for minor correction (Edstam, 2021) as shown in Eqn. [2]. Both are given for the sake of completeness; otherwise, Dehlbom et al. (2021) showed that the difference between results of the two equations is practically marginal. In this approach, F_{3D} is calculated based on the critical F_{2D} and a safety factor for cylindrical failure volume, F_P , with slide length L and plane ends on the side.

$$F_{3D} = F_{2D} + 0.75 (F_P/F_{2D} - 1) \quad [1]$$

$$F_{3D} = F_{2D} + 0.75 (F_P - F_{2D}) \quad [2]$$

As shown in Eq. [1] and [2], the input needed to calculate 3D safety factor, over a 2D slope stability analysis, is the length of the cylindrical failure

surface (L). This is the input in GeoSuite stability to incorporate the so called “3D Side Friction”.

To get the feeling of side friction implemented in GeoSuite stability, a comparison is made with 3D FEM analyses by Jostad and Lacasse (2015). Jostad and Lacasse (2015) used the FEM package Bifurc and studied 3D effects for a slope with constant undrained shear strength and provided a chart for various slope geometries. For simplicity, an isotropic constant undrained shear strength of $c_u = 40$ kPa and unit weight $\gamma = 20$ kN/m³ were used. The slope width (W) was varied in relation to the slope height (H). In addition, b = slope inclination from toe and $d = D/H$ where D is the depth from the toe level to the bottom fixed boundary. Equivalent analysis is done with GeoSuite and presented along with 3D FEM results as calculated by Jostad and Lacasse (2015), Figure 5. This example is a simplified case involving a single isotropic soil with a constant strength and should be suited for comparison with GeoSuite. However, the results indicate that, the Side Friction formulation as adopted in GeoSuite stability gives higher 3D effect.

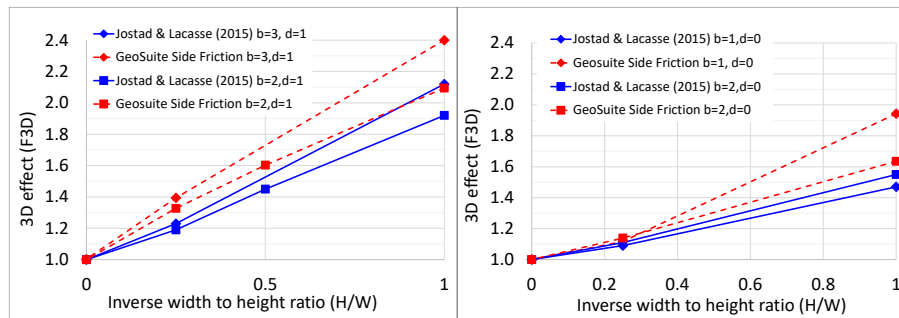


Figure 5. Implied F_{3D} according to 3D FEM Bifurc and GeoSuite side friction (W = slope width, H = Slope height, b = slope inclination, d = (depth to strong layer)/ H)

The practical implication of the Side Friction approach is further elaborated using real example, see Figure 6. Unlike back-calculation of failed slopes, design requires pre-evaluation of parameters. Therefore, in practice, defining the length or extent for the 3D failure surface could be challenging. This is illustrated using analysis on a sensitive clay slope from a road project in Norway with average slope height of 15 m. The two-dimensional analyses gave $F_{2D} = 1.29$ and assuming different length for side shear (Side Friction) gave varying increase in safety factor, see Table 3. Assuming various length for 3D side friction has significant implication on the calculated factor of safety (FOS). It is worthwhile to note that GeoSuite stability does not change the location of the critical slip failure in the 2D profile. In reality, it is expected that the depth of the 3D failure surface differs from the corresponding 2D failure line.

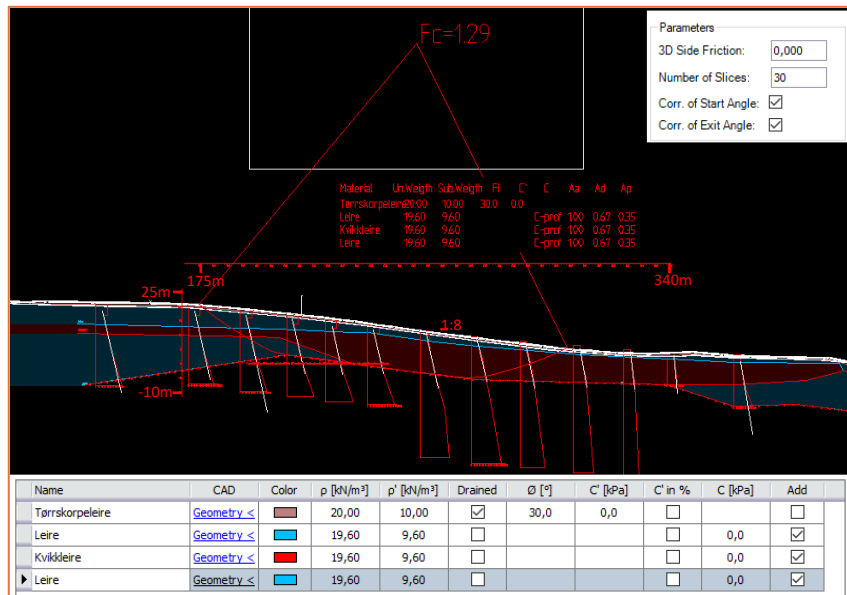


Figure 6 Geosuite stability analyses: Application example from a real project.

In the chosen example, the required FOS for the slope was 1.4 and this is not satisfied in 2D analysis as it gave $F_{2D} = 1.29$. However, one can easily show higher calculated FOS by simply choosing side friction length up to 200 m (see Table 3). The key question is if it is correct to still aim for the same required FOS of 1.4 despite benefiting from side friction.

Table 3. Implication of 3D Side Friction length in safety factor (SF) in GeoSuite

Length for 3D effect, L (m)	50	100	200	400	∞
Calculated SF with Side Friction (-)	1.73	1.51	1.40	1.34	1.29
3D effect ($F_{3D} = F_{3D}/F_{2D}$)	1.34	1.17	1.09	1.04	1.00

Slope stability using FEM Plaxis 3D

FEM is a powerful numerical tool that is based on simulation of stresses and strains in a boundary value problem. Despite being computationally demanding, it has an advantage of simultaneously providing safety factor and identifying critical failure surface when applied to stability problems. For FEM applications, Plaxis 3D V24 is a commercially available tool commonly used in the industry; hence, selected in this work for numerical illustration.

The implications of 3D effect are investigated by using the geometries adopted in Figure 7. Two sets of shear strength profiles are selected for illustration of 3D effects. The first one is based on an idealized constant undrained shear strength of $c_u = 40$ kPa and the second one is a more realistic scenario where a shear strength profile increase with depth is depicted with

$c_u=20 \text{ kPa} + 3 \text{ kPa/m}$. The geometry selected for analysis is a slope with a height of $H=10 \text{ m}$; inclination $1: b=1:3$; $d=D/H$ where D is depth to a strong soil layer or bedrock; and width $W=4H$. For computational efficiency, symmetric modelling is performed. The sloping surface is modelled with ten soil columns each with a width of 3 m . This is a pragmatic way to enable varying reference level for undrained shear strength increase with depth in the sloping part of the slope terrain, see Figure 7. The 3D stability analysis is supplemented by 2D FEM analyses to provide a reference and a 3D plane strain analyses to quantify F_{3D} and effect of element discretization. The resulting factor of safeties as well as incremental displacement at failure for the 2D, 3D plane strain and 3D models are given in Figure 8. A relative comparison of analyses results shows that the 3D effect is higher for a constant c_u as compared to the most realistic scenario of increasing c_u with depth. This is logical as the volume of failure mass subjected to the side shear of the sliding surface becomes significantly higher for the case of constant c_u and the “side shear” has different average shear strength compared to the plane strain case. Further analyses of the results are presented along with element type and mesh discretization aspects in the next section.

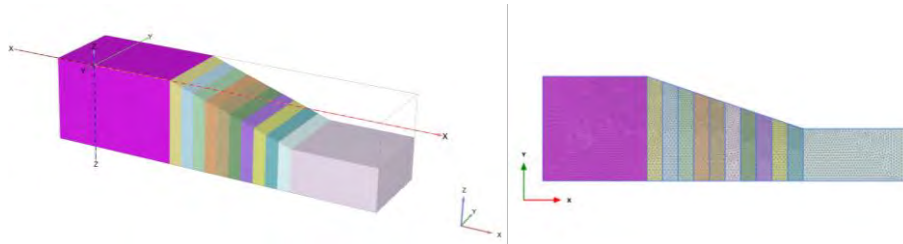
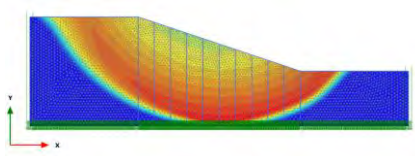
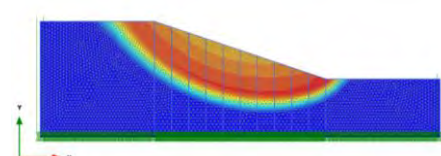


Figure 7 Geometry and soil layering adopted for 3D (left) and 2D (right) analyses.

Constant, $c_u = 40 \text{ kPa}$	Increasing, $c_u = 20 \text{ kPa} + 3 \text{ kPa/m}$
 $F_{2D} = 1.25$ (12944, 15-noded triangular element, average size 0.58m)	 $F_{2D} = 1.44$ (12944, 15-noded triangular element, average size 0.58m)

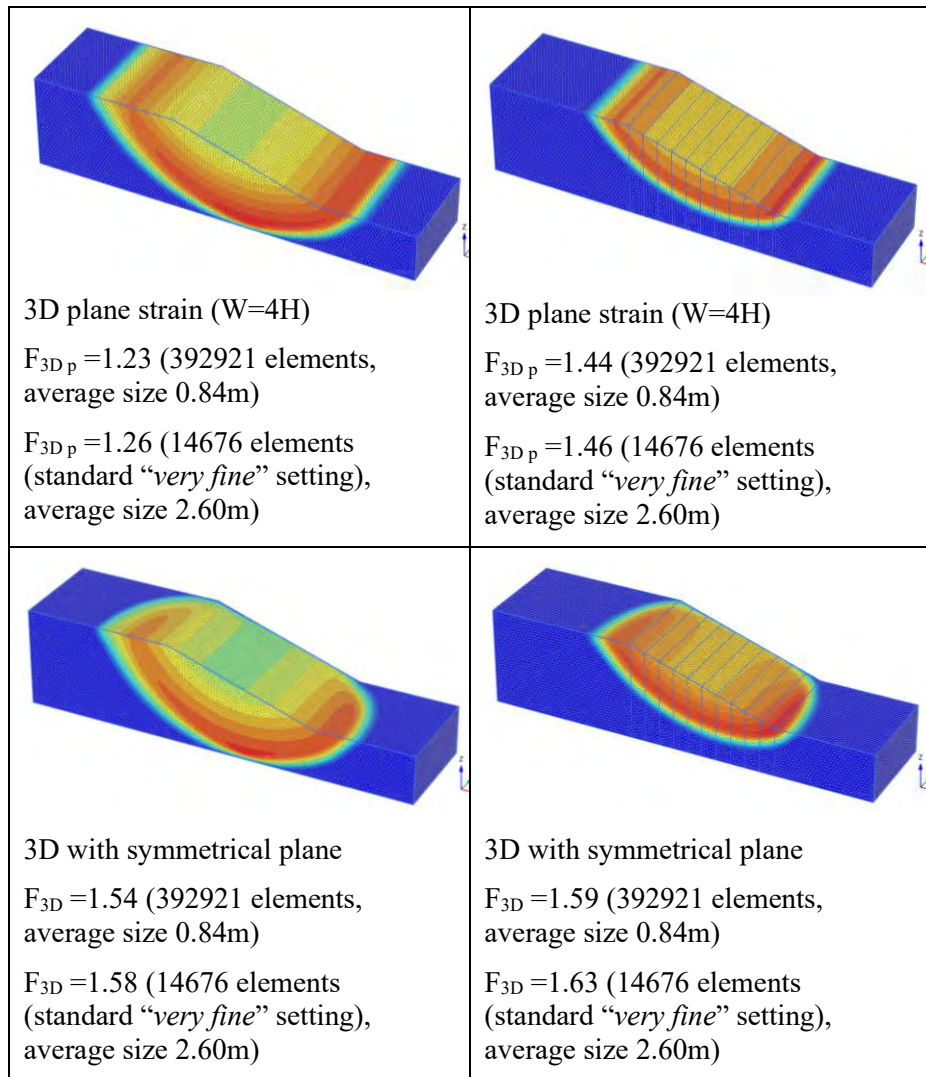


Figure 8 Incremental displacement at failure for 2D, 3D plane strain and 3D models.

Effect of mesh discretization and element type in Plaxis 3D

Mesh discretization and element types are important for analyses in FEM and particularly for 3D modelling. Plaxis 3D uses 10-noded tetrahedral elements with 4 Gaussian points and with second-order interpolation for displacements. Zhang et al. (2013) showed that hexahedral elements are more accurate and much more efficient than tetrahedral elements for stability/capacity analysis. Tetrahedral elements require huge number of elements and take much longer time to provide a factor of safety equivalent to those obtained by hexahedral

element. Otherwise, for a given discretization, tetrahedral element tends to overestimate the calculated safety factor. This is also observed in the current analyses. Jostad and Lacasse (2015) analysis of the case with $b=3$, $d= D/H =1$, $w=W/H=4$ using a mesh containing 1,392 20-noded brick elements with (2x2x2) reduced Gaussian gave $F_{3D} = 1.55$. Equivalent geometry modelling in Plaxis using the standard “very fine” mesh setting gave 14,676 10-noded tetrahedral elements (with average size of 2.60 m). For this case, safety analyses showed a convergence issue giving safety factor of 1.58 after 100 calculation steps. Analysis with 392, 921 elements (average size 0.84m) gave $F_{3D} = 1.54$. The results are dependent on mesh discretization requiring huge number of elements to improve accuracy. This demonstrates the role of element type used in Plaxis for 3D stability analysis.

Effect of varying slope width on 3D factor

A series of highly refined additional numerical analyses were performed to study effect of varying slope width on the 3D stability. This is done for a soil material with constant and increasing c_u with depth. For a slope with height $H = 50$ m, the considered widths were H , $2H$, $3H$ and $6H$. Other fixed features of the geometry of the slope are $b=2.5$ and $d=1.0$. For all the analyses, an attempt was made to maintain an element size of 30 cm. To make the comparison of 3D effects, the corresponding 2D model should have same discretisation error as in 3D model. Therefore, all 2D cases were also simulated in Plaxis 3D ensuring plane strain boundary conditions. Two strength profiles are selected for comparison, i.e. constant $c_u = 35$ kPa and increasing $c_u = 20$ kPa + 3 kPa/m. Further details of the model can be referred to NGI (2022). Summary of the analyses results are presented in Figure 9. The results show that 3D effect is higher for constant c_u and narrower slopes giving 80% increase for $H/W=1$. This reduces dramatically to 20% when increasing c_u with depth is considered. For this example, for $w^{-1} < 0.5$, the 3D effects become less than 10%. This trend implies that a realistic scenario of width of commonly encountered natural slopes and strength profile could rather have a limited 3D effect as compared to constant shear strength. The observed trend is also valid for drained slope stability analysis.

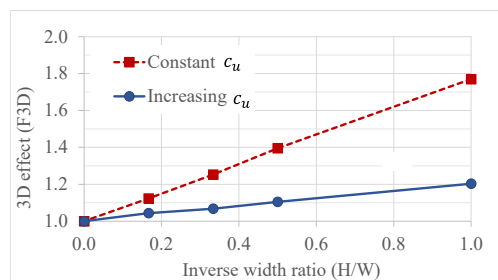


Figure 9 Variation of 3D effect factor with width of slope.

ON EFFECT OF STRAIN SOFTENING

A key aspect of highly sensitive clays is their post-peak reduction in undrained shear strength or strain softening behaviour. For 2D analyses, this aspect is indirectly accounted for by requiring higher safety factor (Fornes and Jostad 2017). An important feature of slope failures in highly sensitive clays is that a small initial slide can trigger and develop into fast and extensive landslide. The idealisation adopted in earlier section may not be suited to study strain-softening behaviour as local triggers initiating failure may not correspond with the rest of the slope geometry. Therefore, this effect is better studied with back-calculation of a failed slope consisting of highly sensitive clays.

Jostad et al. (2021) studied the Skjeggstad landslide, Figure 10a, with respect to 2D and 3D stability. The work demonstrated that one needs to consider 3D effect to realistically explain the observed failure. At the same time, the strain-softening effects in 3D were not accounted for in the study by Jostad et al. (2021). The strain-softening aspect was studied in NGI (2022) with Plaxis 3D using a constitutive soil model referred to as NGI-ADP Soft. The NGI-ADP Soft is extension of the standard NGI-ADP soil model (Grimstad et al., 2012; PLAXIS V21). The NGI-ADP Soft can simulate strain-softening behaviour observed in highly sensitive clays.

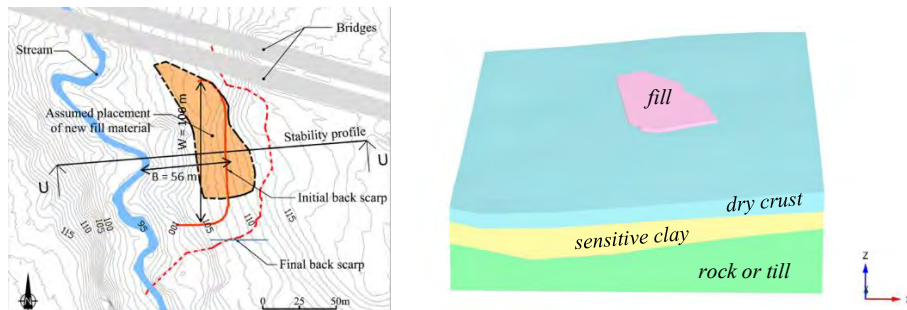


Figure 10 The Skjeggstad landslide a) slide extents (Jostad et al. 2021) b) Plaxis 3D model (NGI 2022)

A representative 3D model of the soil layering where the Skjeggstad landslide occurred was created in PLAXIS 3D based on kriging data, Figure 10b. A three dimensionally varying undrained shear strength profile for the sensitive clay layer was generated by interpolating between existing borehole data from the Skjeggstad landslide area. The 3D effect was analysed by comparing results of 2D and 3D calculations. In addition, the work looked at sensitivity of factor of safety to variation of undrained shear strength profile of the sensitive clay layer, strength of dry crust, rate of softening of the sensitive clay and volume of the fill placed on the slope, that is defined as the trigger for the Skjeggstad slide.

The 3D analysis with strain softening showed 12 % 3D effects for Skjeggstad slope as compared to the critical 2D profile. However, the stability of the slope was highly dependent on the spatial variation of undrained shear strength in the clay. Calculations with various strength profiles for the crust layer, demonstrated the need for extensive field and laboratory tests to accurately estimate the characteristics of the layer as it was found to significantly affect the safety of the slope.

DESIGN CODE IMPLICATIONS

In the current Norwegian practice, sensitive clay slopes are analysed assuming 2D condition. In addition, strain-softening behaviour is not directly assessed. Instead, the analyses are idealised using perfectly plastic behaviour with increased requirements to the calculated FOS. The required increase has been calibrated with extensive 2D numerical analyses comparing behaviour at calculated failure with softening and compared with calculated FOS without softening (NIFS 2015, Fornes and Jostad 2017). It is typically required to satisfy 15% higher safety factor to make provision for softening-behaviour (NVE 2020; NPRA 2022).

As previously mentioned, there is increasing interest for modelling slope stability with 3D as many believe that 3D analysis always gives higher safety for natural slopes (without construction). It is possible to numerically show higher safety factor with 3D effect. However, there are two challenges to this aspect. The first one is that the strain-softening effect in 3D is not thoroughly studied as in the 2D case. The second one is that adding a third dimension into the analyses introduces additional random variable that needs to be accounted for its uncertainty.

Stark and Ruffing (2017) argue that FOS calculated using 3D methods cannot be directly compared to 2D regulatory requirements. This is due to various contributing factors to uncertainty in slope stability analyses including inherent variability of input parameters, difficulty or quality in measuring input parameters, improperly selected critical failure surface, and limitations of stability analyses to represent complex subsurface conditions. As a result, Stark and Ruffing (2017) suggested a higher required safety factor for 3D analyses as a function of level of uncertainty in the soil strength and consequences of failure.

There are key research works attempting to look into effect of spatial variability in connection to 3D modelling (e.g. Hicks et al. 2014; Hu et al. 2022, Varkey et al. 2023, Wu et al. 2024). Wu et al. (2024) concludes that even though 2D generally seems to be more conservative, for a geometrically uniform case, there are cases where due to spatial variability (in 3D) a 3D slide can happen that is not identified in 2D analyses. This implies that a 3D analysis might increase uncertainty (standard deviation). For an example,

case, Varkey et al. (2023) demonstrated that a specific correlation length (in the horizontal plane) lowers the mean FOS with approx. 4% and increases the standard deviation of FOS with 0.06. While the case on only spatial variation in the 2D is considered (correlation length into the plane zero or infinite) the mean value of FOS is not significantly affected. However, standard deviation is highest for the highest correlation lengths. Considering such reduction and increased standard deviation as a possible scenario, a deterministic FOS from a 3D analysis where uncertainties are not considered would require a higher calculated FOS to ensure same probability of failure as the 2D case. Until this is further clarified, only including the positive aspects of 3D should be done with the utmost care.

Current safety factors are calibrated with 2D approach and are meant to be used with 2D analyses. It is not quantified how much of strain-softening and 3D are included in the current design requirements. As a result, this might require a new design philosophy along with a thorough and systematic calibrations.

FINAL REMARKS

The industry is recognizing the need to consider 3D effects in design when 2D idealization is not sufficient. As a result, increasing use of 3D is already being observed in the current practice. Primary focus of this work is stability of natural slopes consisting of highly sensitive clays. For 3D stability analyses of such slopes, it is important to quantify the contribution of side shear while also making considerations for the post peak reduction in undrained shear strength. This dual effect needs thorough study.

Numerical tools commonly used in the Norwegian practice are numerically investigated. Hence,

- The Side Friction approach adopted in GeoSuite overestimates the safety factor even for idealised and simplified case it is meant to be used. Its underlying assumptions could be too simple for a natural slope that exhibits strength increase with depth, anisotropy and involve frictional soil layers. Like all other numerical idealization, it is crucial to see if the underlying assumptions of the approach are applicable for the problem to be used. With calibration to a rigorous approach, one can use the approach for preliminary assessment of 3D effects for local construction activities, e.g. stability of section-wise excavations or fillings and stability of soil mass within lime cement ribs.
- Plaxis 3D gives better alternative to model 3D stability. It requires highly refined mesh to make a realistic prediction and avoid

overshoot of the calculated safety factor, i.e., numerical convergence issue giving higher safety factor. It is important to systematically study discretization error on the obtained safety factor and the corresponding mesh settings.

- For slopes in highly sensitive clays local triggers have huge significance in starting initial slide that can quickly spread to a much larger slide. Local trigger has little or no relation to bigger slope geometry and may not mobilise additional strength from 3D effects. Within the current state of knowledge, it is not recommended to rely on 3D effect for design in natural slopes of highly sensitive clays.

In general, one cannot only aim for increasing calculated safety factor with 3D without making provision for sources of uncertainty. In such cases, the uncertainty in spatial variability of the slope in the third dimension needs to be considered. Current safety factors are developed and calibrated with 2D analyses. Extensive and systematic work needs to be done to make corresponding calibration with consideration for 3D analyses.

ACKNOWLEDGEMENT

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INVESTIGATING SWELLING ACROSS SCALES

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KEYWORDS

Unloading, Swelling, Clay, Pore-water chemistry

ABSTRACT

Volumetric expansion of ground formations containing clay minerals occurs at different scales and depends significantly on the pore water composition. This is both a challenge and an opportunity as prediction of deformations becomes complicated but possibilities to influence them arise. The research presented contribute to answer the questions of whether it is feasible to (1) actively control settlements under linear infrastructure in the far future through changes in pore water and (2) use clay rich formations to seal the annular interface that tends to form between them and the casing of wells so that abandonment oil and gas wells may be repurposed for to store carbon and hydrogen. To investigate these possibilities, both the element and micro scale behavior of a Danish Paleogene Clay is studied. A series of one-dimensional unloading oedometer tests represents the element scale while the microscale is studied through Scanning Electron Microscopy to identify the particle arrangement. At the element scale, three conceptually different swell modes were observed which can be described as increased inhibition of swelling. From variation in pore water composition it was shown how KCl inhibits swelling relative to NaCl, which agrees with previous similar observations from loading. Based on the microscopy it was identified that this is based on interaction between particle and their contacts due to the unchanged fabric of the material during unloading.

INTRODUCTION

The prediction of the behaviour of soil formations known as the Danish Paleogene Clays is a challenge for both new and existing infrastructure in Denmark (the old and new Little Belt Bridges, Århus harbor, Fehmarn Fixed Link, the Kattegat connection, deep geological storage of CO₂, H₂ or heat). These clays can expand and contract significantly during mechanical loading cycles which makes them very reactive to changes in external boundary conditions. The compression behavior of these and similar formations is well studied (e.g. [A], [B], [C]) however a deeper understanding of their swelling behavior is necessary. In addition, the behavior of these formations also

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depends on their pore water composition as seen from samples of similar smectite dominated mineralogy [D]. To enable accurate prediction of deformations in such formations a deeper understanding of their swelling behavior is necessary that combines information from both the micro and element scales.

MATERIALS AND METHODS

The analyzed data represents the Danish Paleogene Clay formation called: Røsnæs Clay. Samples were collected from cores retrieved as part of the preinvestigations for the Fehmarn Fixed Link between Denmark and Germany [E]. The mineralogy of the material was quantified from XRD measurements finding four main components: quartz (17-34%), smectite (8-33%), illite (21-39%) and kaolinite + chlorite (9-25%) as well as trace amounts of other minerals; calcite, dolomite, feldspar and siderite (together: 1-11%). The dominance of clay in the mineralogy and in particular smectite results in the material classifying as high plasticity due to its high liquid limit (w_L), which also agrees neatly with the large specific surface area (SSA) measured. From Table 1 it may also be seen that the material has significant variation in initial state since the initial void ratio (e_i) range from 0.82 to 2.13.

ρ_s	e_i	w_L	w_P	SSA
[g/cm ³]	[-]	[%]	[%]	[m ² /g]
2.68 - 2.87	0.82 – 2.13	84 – 198	26 – 39	55 – 68

Table 1. Range of classification parameters for Røsnæs Clay [E].

All element scale testing was conducted as either Constant Rate of Strain (CRS) or Incremental Loading (IL) oedometer tests following the respective standards ([F], [G]). However, the state of the samples varied from resedimented samples (see [H]), remoulded samples (prepared according to [I]) and natural samples (taken directly from the cores), where these tests are part of those originally published in [J]. The resedimented samples were created based on 0.55M brines either dominated by NaCl or KCl while the remaining tests were carried out with 0.28M (NaCl dominated) brine which represents the in-situ pore water.

The microstructural analysis contains interpretation of SEM images to quantify the orientation of particle edges their amount within a given volume [K]. These SEM images are taken at various stages of loading and unloading.

RESULTS AND DISCUSSION

At the element scale the several unloading branches from the CRS and IL tests are compared in Figure 1a by plotting the change in void ratio during unloading (de) against the Unloading Index (UI), defined as the ratio between the effective vertical stress unloaded from (σ'_{v-max}) and the effective vertical stress at a given point. Figure 1b illustrates conceptually how the unloading branches displays three different swelling modes as defined in [J]; swell mode one (SM1) has continuously increasing swelling index (C_s , defined as the tangential slope of the unloading branch in the compression plane), swell mode two (SM2) finds a plateau C_s (C_{sp}) after an initial increase, and swell mode three (SM3) displays a subsequent decrease of C_s after C_{sp} . From Figure 1a it may be seen that within each swell mode there is a significant variation in C_{sp} as well as the acceleration of C_s . This was identified by [J] to depend on σ'_{v-max} , as illustrated in Figure 2. With increasing σ'_{v-max} , C_{sp} increases, however differently for each swell mode. Note that swell mode 1 is not represented in Figure 3 since C_{sp} is not defined for this mode. The evolution of C_{sp} for SM2 and SM3 is compared to the trends from both remoulded samples (intrinsic trend) [M], and a deeper equivalent formation of Ypresian Clay [C]. This comparison shows that all groups of tests display similar trends but at offset C_{sp} values. Taking the remoulded samples as reference the offset from the intrinsic trend may be interpreted as the extent to which swell is inhibited. This fits neatly with the relative positioning of SM2 and SM3 as well as suggesting that the Ypresian Clay contains more swell inhibiting structure.

In order to elucidate the swelling mechanisms, it is worth looking at soil at the microscopic scale through SEM micrographs for samples at two different depths.

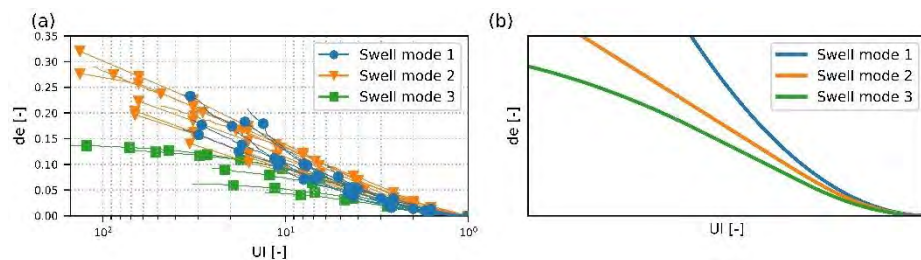


Figure 1: Normalised (change of void ratio since unloading (de) versus Unloading Index (UI)) swelling response under oedometric conditions illustrating the three swelling modes identified by Kinslev et al. 2021 based on (a) the results from oedometer tests and (b) a conceptual more illustrative visualization of the behaviour.

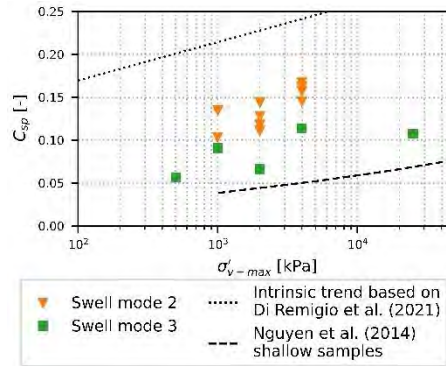


Figure 2: Evolution of the swelling index plateau (C_{sp}) with stress before unloading (σ'_{v-max}) for swell mode 2 and 3 compared to remoulded as well as comparable reference trends.

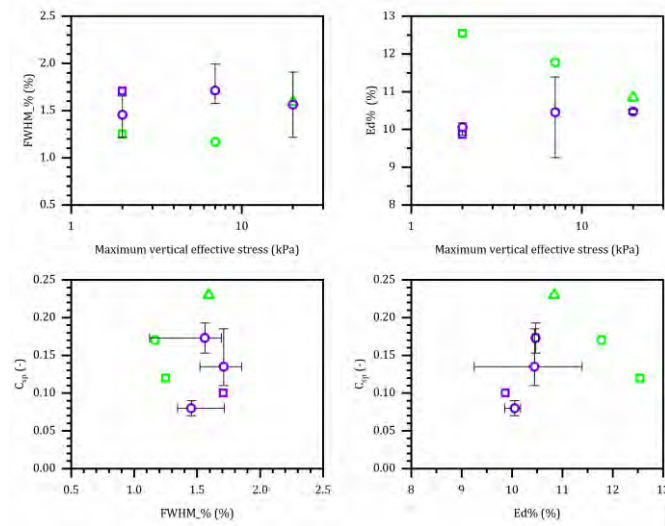


Figure 3: Change in orientation (top left) and amount (top right) of particles within a SEM micrograph as a function of the maximum vertical effective stress from which the specimens were unloaded from. Change in swelling index with orientation (bottom left) and amount (bottom right) of particles for the same tests.

Figure 3 shows the interpretation of such micrographs, where the top row shows the relative scatter in particle orientation (FWHM) since all images show a unimodal distribution in particle orientation, and the amount of edges (Ed%), both as a function of the maximum stress from which the specimens were unloaded. Note that a higher FWHM indicates less scatter. While this parameter is quite erratic for both samples regardless of the stress applied and there is no clear link to c_{sp} , Ed% is more clearly correlated. However for one sample Ed% increases when unloading starts from lower stresses, which corresponds to a lower c_{sp} value, while it is slightly increasing for the other

sample and in this case there is no clear link to c_{sp} . Despite the difficulty in the interpretation, it can be stated that the soil fabric is largely unchanged at the microscopic scale since all particles are aligned to the direction orthogonal to loading. This indicates that for Røsnæs Clay the difference in swelling modes observed at the element scale is predominantly dictated by interparticle contacts and spacing.

The reactivity of the interparticle contacts is further backed up by the significant influence of pore water composition on the deformation behavior of Røsnæs Clay. Figure 4 shows the difference in normalized unloading behaviour of resedimented samples in either NaCl or KCl based brines. This data includes both IL and CRS tests, which for each of the two brines compare very well.

From Figure 4 it may be seen that the swelling response of KCl based samples display a larger degree of swell inhibiting structure compared to NaCl samples, which agrees with observations from loading [N]. The variation in the unloading response between NaCl and KCl based samples stems from changes in the electrical double layer on the surfaces of the individual clay particles, which then changes their interaction and assembly. However, these tests are carried out on samples resedimented in KCl and NaCl based brines. It can be expected that when soils are sedimented in the same brine and the pore water is changed after consolidation, chemical diffusion should be stimulated to achieve similar swelling. However, in low stress scenarios (such as for closing of annulus gaps and close to the surface, hence directly below several types of linear infrastructure) the applicability of pore water change might still be viable.

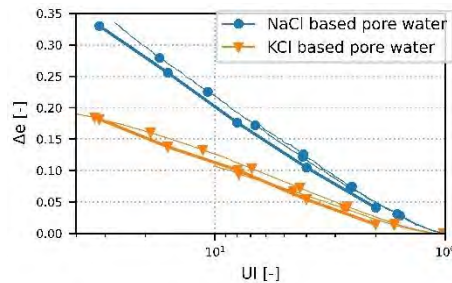


Figure 4: Normalised (change of void ratio since unloading (Δe) versus Unloading Index (UI)) swelling response for samples resedimented in either NaCl or KCl brines.

CONCLUSIONS

The combined interpretation at the element and microscopic scale presented based on oedometric investigation of Røsnæs Clay shows that despite the behavior at the element scale shows three different swelling modes with

increasing inhibition of swelling, there is no fundamental difference in particle orientation. Swelling appears therefore to be dominated by face to face electro-chemical interaction between particles. The strong influence of electro-chemistry is confirmed by the significant difference in swelling observed when unloading occurs in specimens having a NaCl or a KCl based pore water composition. While these results represent an ideal scenario and it can be expected that soils sedimented in the same brine, where the pore water is changed after consolidation, will not show such difference in swelling, in low stress scenarios (such as for closing of annulus gaps and close to the surface, hence directly below several types of linear infrastructure) the applicability of pore water change might still be viable if stimulated for example by electro-kinetics.

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LARGE-SCALE SOIL BIO-CEMENTATION: INSIGHTS INTO HOMOGENEITY, QUALITY CONTROL, AND WASTE HANDLING

R. Harran¹, D.Terzis¹, M. Büyüklü¹ and L. Laloui¹

KEYWORDS

MICP; Upscaling; Quality control; Dynamic penetrometer; Field testing

ABSTRACT

This paper deals with challenges in large-scale soil bio-cementation. Introducing a novel treatment strategy with ex-situ hydrolysis in a 1000 L bioreactor, the study employs a state-of-the-art installation to inject hydrolyzed solutions into a 40 m² area of 0–4 mm sand. A comprehensive multilevel quality control system monitors chemical and hydraulic processes across cycles, resulting in reaction efficiencies exceeding 80%. The paper discusses the gradual increase in pressure at the injection inlet, correlating with higher calcification levels. Findings indicate improved resistance in zones with higher injection pressure, demonstrating the efficacy of ex-situ bio-cementation. The study also explores the mechanical improvements, suggesting comparable results to traditional in-situ methods. Additionally, the paper addresses the monitoring of microbially induced calcium precipitation (MICP) at the geotechnical scale, highlighting the issue of residual ammonium reaching absorbed quantities of 4 mol/L.

INTRODUCTION

Nearly two decades since its inception, Microbially Induced Calcium Carbonate Precipitation (MICP) has gained attention as a sustainable alternative for ground improvement. Utilizing urease-bearing microorganisms, urea hydrolysis induces carbonate ion creation, leading to solid calcium carbonate precipitation. This biochemical process enhances soil properties, impacting permeability, stiffness, compressibility, and shear strength. The technique has been explored in controlled setups and natural environments, addressing diverse geotechnical challenges (Terzis & Laloui, 2019).

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MATERIALS & METHODS

To validate the ex-situ treatment strategy, a bench-scale campaign was conducted. Sand columns, representing the tank's porosity, underwent treatment with hydrolyzed and calcium-rich solutions. Analysis of liquid samples assessed precipitation efficiency through calculations involving influent and effluent reactant concentrations. The laboratory-scale investigation provided insights into the chemical efficiency of the treatment strategy, microstructural comparisons, and its potential for improving soil mechanics.

A reinforced concrete container, $8.0 \text{ m} \times 10.0 \text{ m} \times 2.5 \text{ m}$, was constructed with two compartments for repeated experiments (Harran et al., 2022).



Figure 1 Testing set-up

Unlike previous impermeable setups, this design incorporated draining tubes in the tank's bottom, preventing reactant accumulation. This setup allowed control over water table levels, simulating real environmental conditions. Eight injection wells (Figure 1) per compartment facilitated the bio-cementation protocol's implementation. The design prioritized preventing ammonium leaching and maximizing liquid circulation.

Both compartments were filled with 0–4 mm sand from the Catellani quarry (Switzerland), exhibiting a diverse grain size distribution and composed of 70% quartz and 30% natural carbonate minerals. The sand is labelled SP according to ASTM D2487 but still covers a range of contents from fine ($2.9\% < 0.1$ mm diameter) to coarse ($15.9\% > 2$ mm diameter). (Figure 2). The bio-cementation protocol involved 11 days of successive cycles injecting hydrolyzed carbonate solution followed by immediate cementation solution injection. *Sporosarcina pasteurii* was used for urea hydrolysis, and both solutions were prepared in 1000 L batches. (Table 1) The protocol aimed for full pore volume coverage, and drainage tubes simulated a draining bottom surface, collecting effluents for monitoring. This setup enabled control over injection pressure and offered insights into the bio-cementation process.

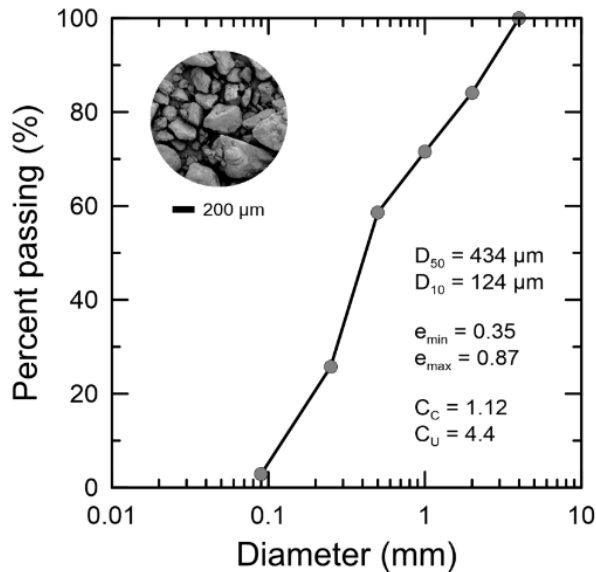


Figure 2 Particle size distribution and properties of the chosen sand

In-situ testing utilized an instrumented dynamic penetrometer (Panda, Solsolution) across treated and untreated sand volumes. Tests at various locations, chosen for spatial heterogeneity representation, assessed the bio-cementation effect on cone tip resistance. The results provided a detailed understanding of the treatment's impact on the soil's mechanical properties.

Mechanical performance assessment involved Unconfined Compressive Strength (UCS) tests on lab-treated samples. The samples

were compared to conventionally bio-cemented ones. Testing included controlled axial strain rates and subsequent determination of calcium carbonate contents, offering insights into the treatment's impact on mechanical behavior.

Table 1 Treatment solution constituents

	Bioreactor Solution (BS)	Cementation Solution (CS)
Peptone (kg/m ³)	1	-
Lyophilized bacteria (L/m ³)	1	-
Urea (kg/m ³)	50	-
Calcium chloride (kg/m ³)	-	200

Samples from both laboratory and large-scale experiments underwent Scanning Electron Microscopy (SEM) for textural and geometrical characterization of the precipitated calcium carbonate. This microscopic analysis provided visual insights into the structure and morphology of the cemented material.

RESULTS

The precipitation efficiencies were computed based on the excess reactant species found in each injection cycle, using either calcium or TIC excess. For each injection cycle, two effluent samples were taken. Remarkably, high completion rates exceeding 70% were achieved with reaction times as short as 4 hours between injections. The effluent predominantly exhibited a richness in calcium in five out of eight injection cycles. The use of pre-hydrolyzed urea played a pivotal role in this efficiency, as calcium carbonate precipitated immediately upon the contact of BS and CS solutions. This process demonstrated comparable efficiency to conventional Microbially Induced Calcium Carbonate Precipitation (MICP). Notably, it boasted faster precipitation times and the advantage of introducing a solution into the ground with known characteristics, concentrations, and stability.

UCS (Figure 3) values increased with rising bond contents, reaching as high as 2.1 MPa for 10% CaCO_3 . Young's modulus (E) varied between 180 MPa and 480 MPa, increasing with higher bond content. Despite some observed scattering associated with the measured heterogeneity in gravimetric contents within the same sand column, the results were consistent. Generally, higher CaCO_3 contents resulted in a brittle response, characterized by distinctive peak strength followed by sharp softening.

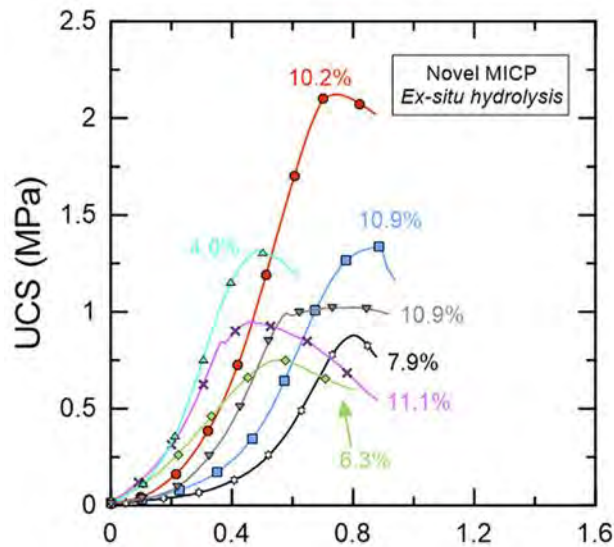


Figure 3 UCS determination on samples produced using ex-situ hydrolysis

SEM observations of precipitates formed in liquid batches after mixing the ex-situ BS and CS solutions are provided in Figure 4. The crystals exhibited a typical rhombohedral calcite morphology, reaching a size of 20 μm when precipitated without a sand matrix.

When ammonium concentrations exceed the water's natural self-purification capacity, it triggers various environmental concerns, notably eutrophication. Eutrophication, in turn, leads to the decline of aquatic life, such as fish, degradation of water quality, and disturbance of biodiversity. Therefore, it is crucial to monitor and mitigate ammonium concentrations arising from the bio-cementation process, to maintain them below acceptable thresholds. The recirculation of ammonium, combined with the levels produced with every new hydrolyzed batch, led to an increase in influent NH_4^+ concentrations throughout the treatment, as illustrated in Figure 5. At the end of the

20,800 L treatment, the effluent NH_4^+ concentration stabilized at around 1 mol/L.

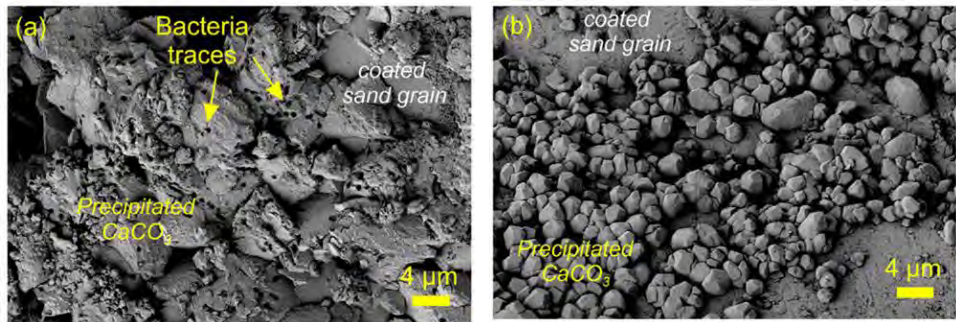


Figure 4. SEM observations of calcium carbonate precipitated on the surface of sand grains using (a) traditional MICP with in-situ hydrolysis and (b) novel MICP with ex-situ hydrolysis.

Considering the average NH_4^+ concentration in the 11 cycles, accounting for dilution, the residual recorded ammonium aligned with the theoretical value. This suggests that to bring NH_4^+ within acceptable limits, substantial volumes and pumping times are needed in large-scale projects. The potential retention of NH_4^+ on soil particles and dissolution in groundwater observed in previous literature works further underscores the challenges associated with NH_4^+ elimination through MICP-induced ureolysis.

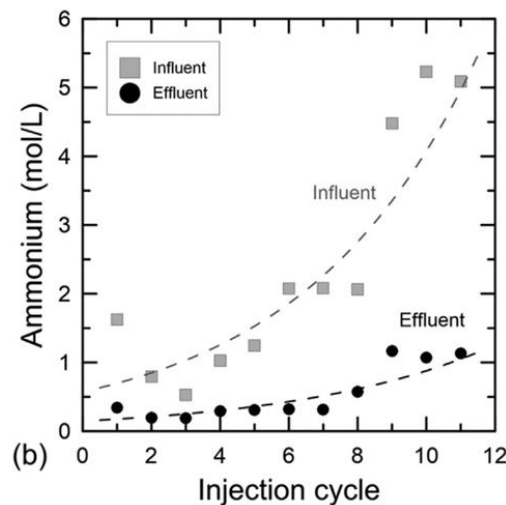


Figure 5. NH_4^+ concentrations during recirculation of enriched treatment batches

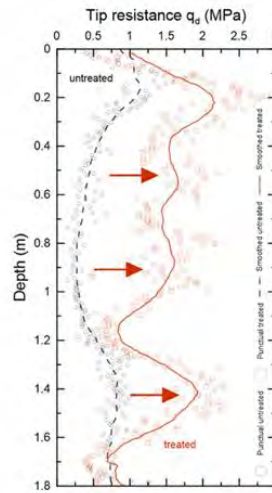


Figure 6. Penetrograms for both treated and untreated states using a sleeved tube

Hence, the study postulates that byproduct extraction from treatment solutions prior to injection into the ground using ex-situ hydrolysis remains a promising alternative. This approach aims to enable NH_4^+ elimination and minimize operational constraints, ultimately producing soil admixtures or grouts for bio-cementation that align with environmental norms. The results of dynamic penetrometer tests revealed varying heterogeneity in the treated section. Figure 6 presents a typical penetrogram result with a discrepancy at a depth of 1.2m attributed to the well's outlet. All tests in the untreated compartment demonstrated overall homogeneity in cone resistance (q_d) across depth, with values ranging between 0.5 and 1 MPa.



Figure 7. Various types of injection tubes utilized in subsequent experiment

To evaluate injection methods for homogeneously treating the soil, a follow-up experiment was conducted. The same basin was equipped with six boreholes. Four different types of injection pipes were used, (Figure 7 & 8) and the obtained results were compared. The pipe types included a conventional tube-a-manchette pipe, a tube-a-manchette pipe with softer sleeves, a steel injection rod with ports on the tip, and a tube-a-manchette pipe with horizontal slits between main ports. During injections, a double inflation packer was used to ensure the same amount of injection was delivered through each port at different depths. The amount and pressure of the injection flow were controlled by flow meters and pressure meters, respectively. The most effective results in achieving homogeneity throughout the depth were obtained using the slit pipe (Figure 9).



Figure 8. General view after exhuming biocemented sand columns

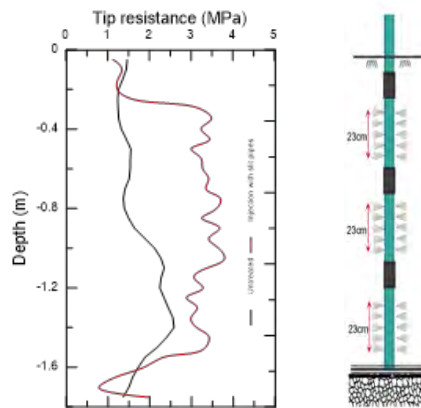


Figure 9. Uniform treatment of sand throughout its depth using slit pipes

CONCLUSIONS

The ex-situ hydrolysis approach demonstrated high precipitation efficiencies and comparable mechanical properties to traditional MICP. The microscopic analysis revealed changes in mineralization processes at the pore scale. Chemical monitoring confirmed the efficiency of the precipitation reaction, with implications for the treatment of residual ammonium. The dynamic penetrometer campaign provided spatial insights into the effectiveness of bio-cementation, highlighting areas of significant improvement. Overall, the study contributes to bridging laboratory advancements with large-scale implementation of bio-cementation, providing a comprehensive multi-level quality monitoring system for real-world applications.

ACKNOWLEDGEMENT

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L-SHAPED GRAVITY WALLS. RECOMMENDED PRACTICE FOR ULS

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KEYWORDS

Gravity walls, non-associated flow versus associated flow, water pressures.

ABSTRACT

The scope of the present paper is to identify key issues that should be considered during the design of L-shaped gravity walls. Results from the finite element method have been combined with a tailor-made software code to investigate relevant mechanisms to be included in the recommended approach. The main elements addressed are the failure mechanism along the rear side wall and added robustness in the design. The paper suggests a simple design approach that will provide safe structures without undue conservatism.

INTRODUCTION AND BACKGROUND

When designing L-shaped gravity walls in the ultimate limit state (ULS), cf. Figure 1, equilibrium shall be ensured considering passive earth pressures in the front of the wall, active earth pressures along the rear side of the wall, water pressures and bearing capacity of the base of the wall. The failure mode of L-shaped gravity walls may be characterized by horizontal sliding or lack of bearing capacity caused by overturning moment; examples of the latter are illustrated on Figure 1.

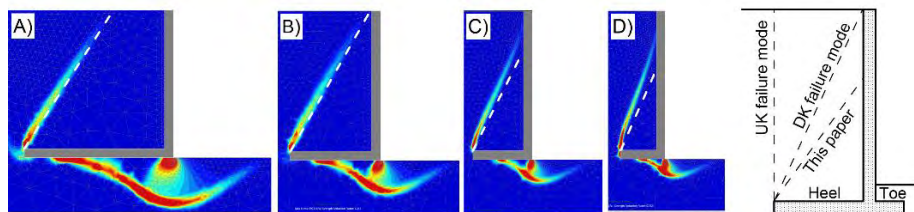


Figure 1 Contour plots of mobilised shear stress as found by the finite element method in coarse grained soils. The white dashed lines are inclined corresponding to the statically admissible angle, $45^\circ - \phi'/2$ (active failure boundaries).

Most countries agree on the passive pressure, but active pressures are applied differently from country to country and deviations are also observed

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A computer code is developed to conduct the calculations using a geometry as defined on Figure 2, Part 1. When the DK approach is investigated, the rear side rupture line goes along the points ABD, assuming a completely rough interface. For the UK approach, the bounding rupture line extends vertically from point A (smooth interface). A third failure mode is investigated where the rupture line runs along AB, and then along a line inclined θ with vertical so it may intersect the wall (e.g. point C) or the soil surface (e.g. point E), assuming a completely rough interface. An optimization routine is included in the third failure mode where the maximum overturning moment around point A is sought by varying θ and using this overturning moment to establish the required vertical reaction.

Figure 2, Part 2, shows the actions considered and the sum of actions shall be counteracted by reactions complying with vertical and horizontal equilibrium together with moment equilibrium in accordance with EN 1997-1 (reactions are marked in red on Figure 2). The overturning moment is evaluated with reference to point A. The symbol G_C refer to the weight of the wall, G_F is the weight of the soil on top of the toe and G_B is the weight of the soil at the rear side. The vertical surcharge load contributes with a vertical component provided that the value of θ causes the rupture line to intersect the soil surface to the right of point D. Each of these components (and the associated point of application) are estimated together with the total passive and active earth pressures (tangential components included) acting on the wall. Soil resistance relies on effective stresses while G_C , G_F and G_B should be based on total unit weights and a water pressure following Section 0 (not included in current calculations).

Projecting all actions on a horizontal plane leads to the horizontal reaction, H and similar for the vertical reaction, V . The estimated moment around point A, MoA is used to derive the eccentricity, e using $MoA = V \cdot (e + BB/2)$. The calculation procedure above may be wrapped in a zero-seeking procedure where the heel length, $B1$, is changed until the vertical pressure caused by V over the effective foundation width equals the computed bearing capacity or until the sliding condition ($\tan\phi' = H / V$) is met, whichever is most critical for the structure in question. The output from the suggested approach is therefore the minimum foundation width needed to ensure equilibrium. For the third failure mode, the output is also the location of the critical rupture line along the rear side wall, e.g. ABCD on Figure 2.

Earth pressure and bearing capacity

The total earth pressures discussed above are computed by integrating the unit earth pressures along the failure planes assumed, and where the associated earth pressure coefficients are derived in accordance with [3], Mortensen & Krogsbøll (2019). Note that [3] deviates from EN 1997-1, Appendix C.2 by introducing an earth pressure coefficient for the γ -case, K_γ using $K_\gamma = K_p$ for

the active case (K_p refer to surcharge case) while $K_\gamma = K_p/\cos\delta$ for the passive case, which is an approximation not deviating more than 4 % from the theoretically correct value provided that $\varphi' \leq 45^\circ$ and the soil surface is horizontal.

The drained bearing capacity formula follows DS/EN 1997-1 DK NA:2021, [4] and only the unit weight and surcharge contributions have been shown for plane strain corresponding to normal eccentricities, $e/BB \leq 0.30$:

$$\frac{V}{b_{\text{eff}}} = 0.5 \cdot \gamma' \cdot b_{\text{eff}} \cdot N_\gamma \cdot i_\gamma + q' \cdot N_q \cdot i_q \quad (2.1)$$

where $b_{\text{eff}} = BB - 2 \cdot e$ (effective foundation width), γ' is the effective unit weight, $N_\gamma = 0.25 \cdot ([N_q - 1] \cdot \cos\varphi')^{3/2}$, $i_q = (1 - H/V)^2$, $i_\gamma = i_q^2$, q' is the effective surcharge load at foundation level, $N_q = \tan^2(45^\circ + \varphi'/2) \cdot e^{\pi \cdot \tan\varphi'}$. When the normalised eccentricity, e/BB exceeds 0.30 (strong eccentricity), the bearing capacity failure mechanism is a rotational mechanism below the foundation and Equation (2.1) turns into $V/b_{\text{eff}} = \gamma' \cdot b_{\text{eff}} \cdot N_\gamma \cdot i_\gamma$.

RESULTS

Main tendencies from the results analysing a series of different geometries have been included below. Figure 3 shows the failure mechanisms developed around an L-shaped gravity walls with the height $HH = 5.0\text{m}$ and foundation width varying between 5.0 and 1.0m. The L-shaped walls are placed directly on the soil surface, so the bearing capacity of the walls is a pure γ -case (no soil cover on the front side of the wall).

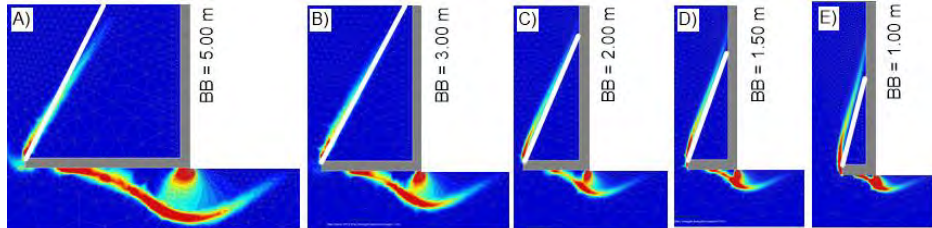


Figure 3. Contour plots of mobilised shear stress as found by Optum^{G2} in coarse grained soils. The white line from the rear side heel refer to the inclination of the critical rupture line found from the third failure mode in the Limit Equilibrium Method.

Failure of the structures in Optum^{G2} was introduced by gradually reducing the friction angle until a mechanism has developed, and the corresponding failure value of φ' was then used in the Limit Equilibrium Methods to estimate the required heel length B1 on Figure 2, Part 1. Figure 4 is established as Figure 3, but the wall is embedded into the soil and a toe has been introduced.

Main results from 12 different cases covering walls placed at the soil surface, embedded walls, and walls with and without surcharge loading are: The sum of foundation widths from the 12 cases in Optum^{G2} adds up to 33.2m. The

corresponding value from the DK approach leads to 37.9m (+12% relative to Optum^{G2}), the UK approach sums up to 37.5m (+14%) and the third Limit Equilibrium Method arrives at 37.0m (+11%). The three Limit Equilibrium Methods uses the same computational approach when estimating the bearing capacity and the passive earth pressure so any difference in results must reflect a difference in the way the active earth pressure is estimated, and no practical difference is observed. The estimated value of the total active earth pressure from the three different Limit Equilibrium Methods is not affected by the degree of sophistication applied in the various models. It is therefore suggested to use the UK model when evaluating the active earth pressures as this involves rather simple calculations.

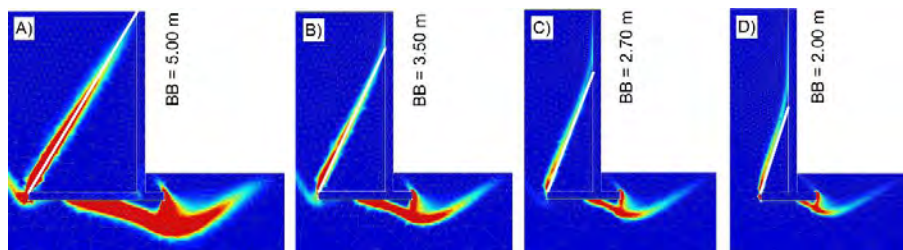


Figure 4 Contour plots of mobilized shear stress using Optum^{G2} in coarse grained soils. The white line from the rear side heel refer to the inclination of the critical rupture line found from the third failure mode in the Limit Equilibrium Method.

Figure 4, Case A) represents a rupture line from the rear side heel to the top of the wall and the inclination of this rupture line with horizontal is close to the statically admissible angle being $\pi/2 - \phi'/2$. There is a vague tendency in the results obtained that the DK method leads to shorter heels than the UK method when the angle between the heel and the line from the rear side heel to the top of the wall exceeds the statically admissible angle and opposite if it doesn't. This tendency is, however, not very strong and it will not change the recommendation about the UK method being the preferred method.

Figure 3 refer to cases where the wall is placed at the soil surface and the bearing capacity is therefore a pure γ -case. The active pressures from Figure 3, Case A) and B), when estimated by Optum^{G2}, must be almost identical to the values found using the UK method so any difference in results comparing the heel length from Optum^{G2} and the heel length from the UK method must be found in the bearing capacity formula, Equation (2.1). The bearing capacity factor from Optum^{G2} was therefore estimated for a centrally loaded foundation placed on a coarse grained soil with no surcharge using associated flow to check the validity of the program. Optum^{G2} found $N_\gamma = 14.8$ for $\phi' = 30^\circ$ while the value from Equation (2.1) is 14.6 showing a close match. The Finite element method and Equation (2.1) therefore seems to deviate on the inclination factor, i_γ . Steenfelt (2003), [5], suggested that $i_\gamma = (1 - 0.7 \cdot H/V)^5$

should be used in Equation (2.1) and Table 1 illustrates the effect of the different inclination factors.

Table 1 Main results for Figure 4 case A) using Optum^{2G} and the UK method with two different inclination factors.

	BB [m]	b _{eff} [m]	V/b _{eff} [kPa]	H [kN/m]
Optum ^{G2}	5.00	4.39	131	117
UK, $i_\gamma = (1 - 0.7 \cdot H/V)^5$	5.38	4.72	117	106
UK, $i_\gamma = (1 - H/V)^4$	5.67	5.05	116	106

Optum^{2G} results in Table 1 are found by integrating effective normal stresses and shear stresses along the interface between the soil and the bottom part of the foundation. A similar approach has also been used on rupture figures depicted in Figure 3, Part E) and Figure 4, Part D) revealing that $b_{eff} / BB > 0.30$. This ratio reflects a strong eccentricity, cf. Equation (2.1), and the failure mode in the soil below the wall should reflect a clockwise rotation of the wall, which is not observed from the contour plots (but found in the Limit Equilibrium Methods).

WATER PRESSURES AROUND THE WALL

The presence of water around the structure will influence the design. Drains may be used to control the water pressure but as the lifetime of a drain is typically shorter than the lifetime of the structure, drains cannot be assumed to be fully operational. Differential water pressures will cause a steady state flow situation where water will typically flow from the rear side of the wall towards the front side. This effect will cause a destabilising horizontal force from water combined with hydraulic gradients influencing the effective unit weight and thereby also influencing the active and passive earth pressures. Piping/erosion may develop on the front side of the wall and as the base plate act a membrane, water pressures below the plate will act upwards, cf. Figure 5 (Left), and these pressures will influence the vertical reaction, V on Figure 2 together with the overturning moment.

DISCUSSIONS AND CONCLUSIONS

Discussions, Associated flow versus non-associated flow

The finite element analyses included in this paper has consistently been conducted assuming associated flow; that is to use the same angle of dilatancy as the friction angle, $\psi = \varphi'$ and it deviates from non-associated flow where $\psi < \varphi'$. Using finite element analysis to verify the earth pressure coefficients from EN 1997-1 Appendix C.2 reveals that associated flow must be used,

which is also the case for the bearing capacity factors in Equation (2.1). From testing in the geotechnical laboratory, it is known that real soils behave as non-associated materials, and it is also known that non-associated analyses will lead to a lower resistance than associated material when using the same friction angle. The designer must therefore account for these aspects when deriving characteristic values of the shear strength properties if simple design tools are to be used.

Figure 5 (Right) has been based on a series of 2D Plaxis analysis where the ULS resistance has been computed using associated and non-associated flow ($\psi = \varphi' - 30^\circ$ [industry practice]). The figure allows for transforming friction angles from non-associated flow to associated flow for active earth pressures, passive earth pressures and for the bearing capacity.

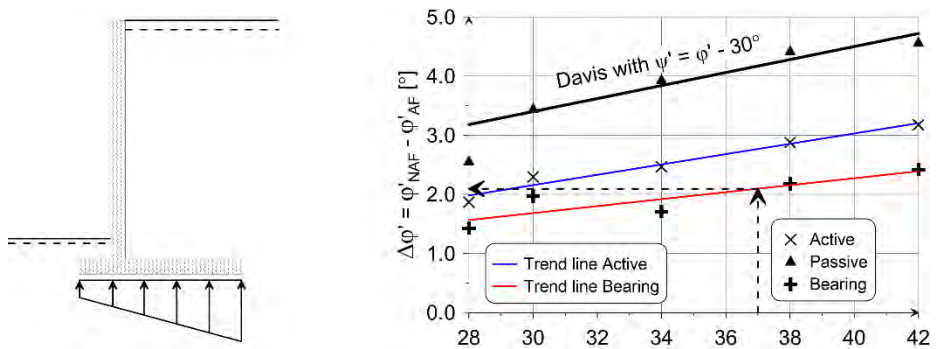


Figure 5 Left: Vertical water pressures acting on the base plate caused by differential water pressures acting on the wall. Right: Simplified method to evaluate the effect of associated flow versus non-associated flow.

The x-axis on Figure 5 (Right) represents the friction angle from non-associated flow, φ'_{NAF} while the y-axis represents $\Delta\varphi' = \varphi'_{NAF} - \varphi'_{AF}$ where φ'_{AF} is the friction angle for associated flow. It is seen that $\varphi'_{NAF} = 37^\circ$ (and thus $\psi = 7^\circ$) leads to the same bearing capacity as $\varphi'_{AF} = 37 - 2.1 = 34.9^\circ$. The line “Davis” follows $\tan\varphi'_{AF} = \sin\varphi'_{NAF} \cos\psi / [1 - \sin\varphi'_{NAF} \sin\psi]$ using $\psi = \varphi'_{NAF} - 30^\circ$ and has been suggested by [6], Davis (1968). The Plaxis analyses with passive pressures in Figure 5 (Right) are based on the pure γ -case without surcharge. The active pressures and the bearing capacities in the same figure have been computed using $\gamma' = 10 \text{ kN/m}^3$, a foundation width (or wall height) of 3.0 m and a surcharge of 15 kPa.

Discussions, Robustness

A structure is robust when the safety is only slightly sensitive to unintended effects and defects or, when there is no extensive failure of the structure if a limited part of the structure fails, cf. [7], DS/EN 1990 DK NA.

A critical aspect for most gravity walls is a reduced soil resistance caused by flowing water on the front side of the wall and below the base plate. Such events must be mitigated by establishing a sufficient soil cover in front of the wall and ensuring that the design water tables have been set conservatively. The roughness of the front side wall and/or along the foundation base should be re-considered if the wall fails in sliding. If the ALS or robustness analysis reveals a wall failing by bearing pressures combined with a strong eccentricity, extreme care should be exercised as all partial safety factors are unity (no safety) and because a possible failure will likely develop without a detectable warning, e.g. settlements / displacements.

Conclusions

For design of gravity walls, the UK approach (active failure boundary as vertical line from heel extreme) should be the preferred design approach as it is very easy to use, and it does not imply cumbersome evaluations of triangular stiff soil bodies and earth pressures along inclined lines; both aspects shall be re-computed for every heel length evaluated to establish the final design.

The cases studied implies that the largest difference between finite element analysis and the limit equilibrium methods originate from the bearing capacity equation applied. The Danish bearing capacity equation, as it appears in EN 1997-1 DK NA:2021, should thus be re-evaluated.

Characteristic values of the soil properties shall account for differences between associated flow and non-associated flow depending on the tool applied in the design. Using a design tool reflecting associated flow, the soil properties on the passive side should be carefully evaluated.

Water tables must be selected conservatively taking the lifetime of the structure into account. The effect of flowing water shall be considered carefully.

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MASWAVESPY: A PYTHON PACKAGE FOR ANALYSIS OF MASW DATA

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KEYWORDS

Surface wave analysis, MASW, Shear wave velocity, Open-source software

ABSTRACT

This paper presents MASWavesPy, an adaptable, open-source Python package to retrieve S-wave velocity (V_S) profiles from MASW-type active-source surface wave registrations. The data processing and forward modelling tools of MASWavesPy were validated by comparison with existing software. The analysis approach was then assessed by measurements at four geotechnical benchmark sites, where the retrieved V_S profiles were found to be consistent with existing in-situ and laboratory measurements.

INTRODUCTION

MASW (multichannel analysis of surface waves) is a non-invasive approach for in-situ evaluation of soil S-wave velocity (V_S) profiles. In the past two decades, surface wave methods (SWM), including MASW, have become increasingly more common in civil engineering practice as tools to retrieve the V_S distribution of soil sites down depths of a few tens of meters.

MASWavesPy is an open-source, adaptable Python package to process and analyze MASW-type active-source surface wave registrations and evaluate soil V_S profiles [1]. It presents an advancement of an earlier MATLAB tool created by the same authors [2,3] with more refined data processing and analysis methods for improved code usability and performance. Computationally intensive parts of the software are written in Cython for increased computational speed. The main aim of this paper is to present the new package and explain its main modules. The data processing and forward modelling tools of MASWavesPy are verified by comparison with existing software. The performance of the package as a whole is then assessed by measurements at four geotechnical benchmark sites in Norway, where V_S profiles obtained with MASWavesPy are compared with existing in-situ and laboratory data.

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METHOD

For MASW field testing (Fig. 1), a linear array of equally spaced receivers is placed on the soil surface and a vertical impact load is applied in-line with the receivers. The interval V_S profile of the underlying soil deposits is then retrieved by inverting the observed Rayleigh wave (R-wave) dispersion curve (DC). The inversion is conducted by modelling the subsurface as n homogeneous and isotropic linear elastic layers over a half-space. Each layer is described by its V_S , P-wave velocity (V_P) (or Poisson's ratio, ν), mass density (ρ) and thickness (h). In this work, the V_S and thickness of each layer are established by inverting the fundamental mode DC. Hence, the term 'dispersion curve' generally refers to its fundamental mode. In line with common practice [4], the values of V_P (or ν) and ρ for each layer are estimated based on available data and assigned fixed values during the inversion process.

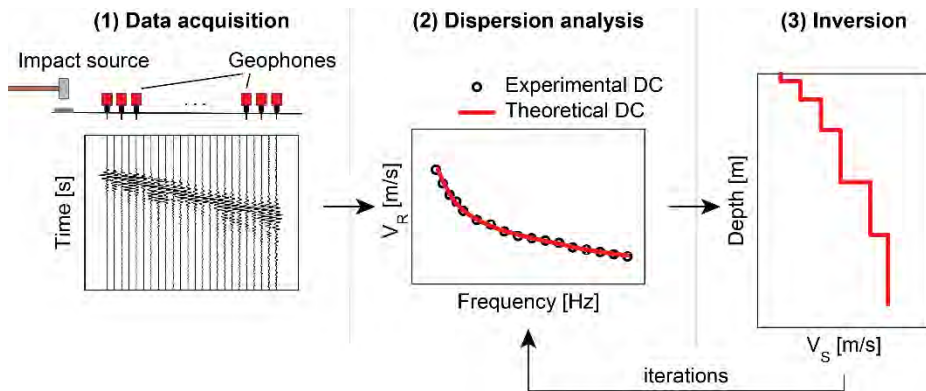


Figure 1. MASW surveying and analysis procedure. (1) Data acquisition: An impact load is applied on the soil surface and the wave propagation is recorded by an array of geophones. (2) Dispersion analysis: Data processing to retrieve the R-wave DC for the site. (3) Inversion: Evaluation of the V_S profile by inversion of the experimental DC.

A single shot gather may be sufficient to retrieve the V_S -depth distribution at a given site. However, to get a quality assessment of the V_S profile it is, in the authors' experience, important to collect multiple records with several impact locations (source offsets) and, in some cases, two or more receiver array lengths. This aligns with prior studies that advice collecting repeated forward and reverse shots to better assess the experimental DC and, consequently, the V_S profile [4-6]. The benefits of using multiple source-receiver configurations include mitigation of near- and far-field effects, the possibility of assessing DC uncertainty, and an extended DC frequency range providing both an improved survey resolution close to surface and an increased surveying depth.

MASWAVESPY PROGRAM STRUCTURE

MASWavesPy is designed for processing and analyzing surface wave datasets that consist of multiple shot gathers. The package contains four main analysis modules (`wavefield`, `dispersion`, `combination`, and `inversion`) and two supplementary modules (`dataset` and `select_dc`).

MASWavesPy can be installed from the Python Package Index (PyPi) using the command `pip install maswavespy`. Its code, together with a quick start guide and sample data, can also be downloaded from Github (see further <https://pypi.org/project/maswavespy/>). The implementation described below refers to version v.1.0 (released 02.24). A more comprehensive description of the computational procedure is given in Olafsdottir et al. [1].

The `wavefield` module imports shot gathers as `RecordMC` objects (one object for each shot gather) and transforms each seismic record into the frequency-phase velocity ($f-V_R$) domain by using the phase shift method [7]. A DC obtained from a single shot gather is here referred to as an elementary DC. The supplementary `dataset` module can be used to batch import surface wave datasets, containing multiple shot gathers, as a `Dataset` object.

The `dispersion` module includes methods to visualize the $f-V_R$ spectrum and identify the corresponding elementary DC. An `ElementDC` object stores the $f-V_R$ domain representation of a given `RecordMC` and the identified elementary DC. In the $f-V_R$ domain, the propagation of the fundamental R-wave mode (and higher modes, if excited) is revealed by the spectral amplitude maxima. Hence, the experimental DC is extracted by picking the relevant amplitude maxima over a range of frequencies. The `select_dc` module provides an easy-to-use graphical user interface (GUI) to aid the DC extraction. It also includes a function to automatically identify the absolute amplitude maxima at each frequency. It should, however, be noted that manual inspection of the spectral image, and the trend shown by the amplitude maxima with frequency, is a crucial aspect of the DC identification. Sometimes, the fundamental mode may be associated with a local maximum of the spectral image or masked by a higher mode or disturbances in the seismic data.

The `combination` module provides methods to combine elementary DCs, obtained from multiple shot gathers, into a composite DC with upper and lower boundaries [8]. It is, therefore, specifically intended for processing surface wave datasets. It further provides tools to assess the variation within the set of picked elementary DCs with wavelength or frequency. A `Dataset` object contains multiple `RecordMC` and `ElementDC` objects (one pair for each imported shot gather) and provides a routine for initializing a

CombinedDCs object for the set of records or a given subset of processed records.

Lastly, the **inversion** module provides routines to assess the V_S profile of the surveyed site by inverting the composite DC (as is recommended) or a particular elementary DC. The inversion methods, including methods for post-processing and visualizing the inversion results, are defined on an **InvertDC** object that is initialized using a given DC. The fast delta matrix algorithm [9] is used for forward modelling (i.e., computation of theoretical DCs) and a Monte-Carlo global search algorithm [3] for searching the solution space for the optimal set of model parameters (V_S and h for each layer). A detailed description of the inversion process and recommended practices, e.g., related to model parameterization, is provided in Olafsdottir et al. [1,3].

EVALUATION OF MASWAVESPY

Comparison with other computational methods

The wavefield transformation and DC extraction methods of MASWavesPy were verified by comparing their results with those of the Active FK toolbox of Geopsy [10]. Figure 2 shows spectral images that were obtained with the two programs using data from three locations in Iceland (one 24-channel shot gather for each site). The three sites are characterized by sediments of Holocene granular materials but have different surficial grain size distributions (silty sand to sandy gravel) and surficial compaction levels.

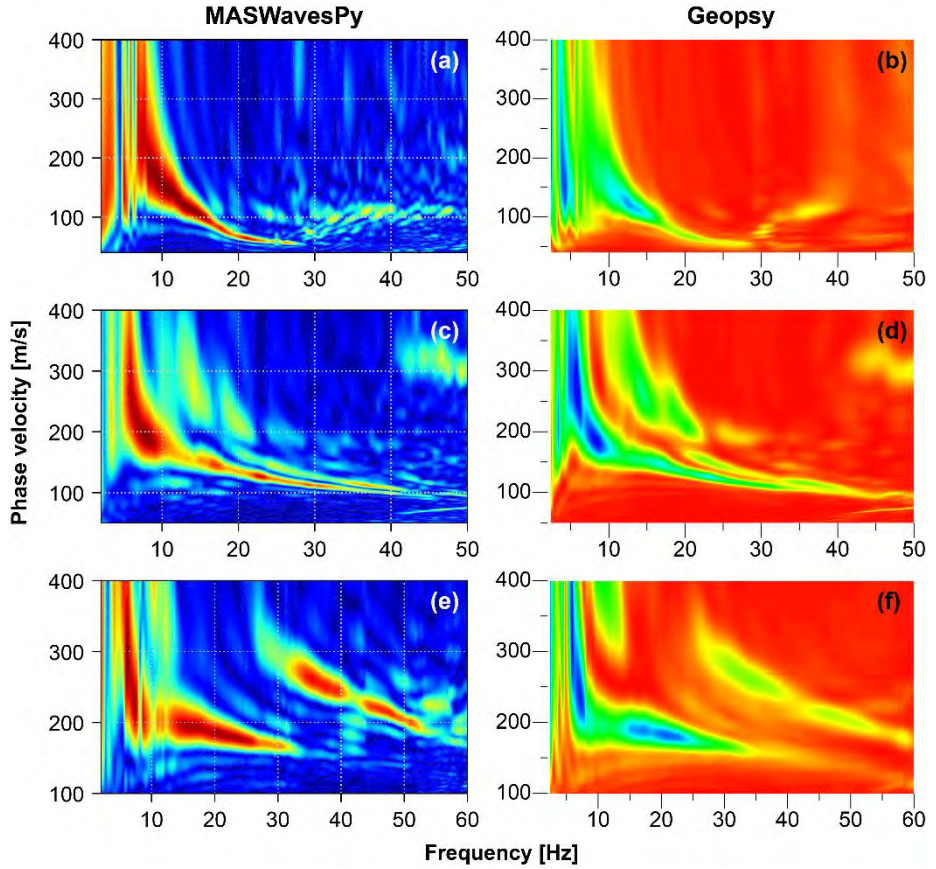


Figure 2. Spectral images obtained with MASWavesPy (left column) and Geopsy (right column). (a,b) Site I. (c,d) Site II. (e,f) Site III.

The corresponding elementary DCs are given in Fig. 3, where they are presented in the phase velocity-wavelength ($V_R-\lambda$) domain. Also shown is the composite DC for each site with experimental boundaries defined as one standard deviation of the mean curve. Each composite DC was retrieved using a variety of receiver array lengths and source offsets. Therefore, its wavelength range is wider than that of the two elementary DCs.

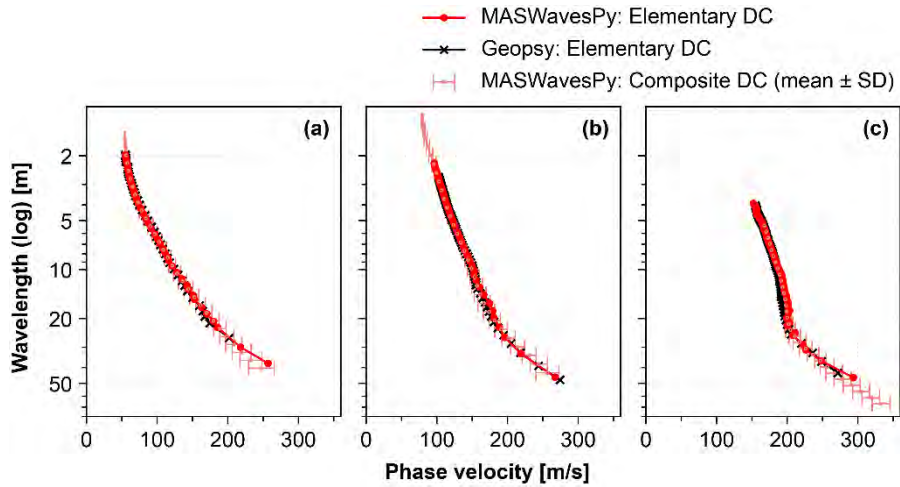


Figure 3. Comparison of experimental DCs obtained with MASWavesPy and Geopsy for (a) site I, (b) site II, and (c) site III. The elementary DCs are extracted from the spectral images shown in Fig. 2. The composite DC (obtained with MASWavesPy) for each site is shown with upper and lower boundaries corresponding to one standard deviation (SD) of the mean curve.

As shown in Figs. 2 and 3, there is a good agreement between Geopsy and MASWavesPy. Each pair of spectral images shows the same dispersion characteristics. The extracted DCs also agree well, with the observed inter-code differences being smaller than, or comparable to, the estimated uncertainty of the composite DC. The fundamental R-wave mode dominated the surface wave signal at sites I and II. At site III, a higher mode was found to dominate at frequencies exceeding 30 Hz (Figs. 2ef), therefore limiting the maximum frequency (minimum wavelength) of the extracted DCs (Fig. 3c). For site II, the spectral imaging routines of both programs revealed higher mode propagation between approximately 12 Hz and 40 Hz (Figs. 2cd). The effects of this on the identification of the fundamental mode DC were though minimal (Fig. 3b).

The forward modelling tool of MASWavesPy was evaluated by comparing its results to those of the gpdc module in Geopsy [10]. Figure 4 shows theoretical dispersion curves (TDCs) that were obtained using the two programs for three soil layer models of varying complexity (Table 1). The models, initially defined by Tokimatsu et al. [11], all present values of V_S that are consistent with those commonly measured in the upper-most 15–20 m at granular soil sites. Model A represents a normally dispersive soil profile. The S-wave velocity varies more irregularly in models B and C, with a stiff surficial layer in model B and a low-velocity layer at depths of 6–14 m in model C. The TDCs were computed over a frequency range of 3–100 Hz, as commonly retrieved in active-source surveys. As shown in Fig. 4, the results show

excellent agreement between the two programs, with the TDCs in all cases being nearly identical.

Table 1. Definition of soil layer models A, B and C.

Layer	h [m]	V_P [m/s]	ρ [kg/m ³]	V_S [m/s]		
	(All)	(All)	(All)	Model A	Model B	Model C
1	2.0	360	1800	80	180	80
2	4.0	1000	1800	120	120	180
3	8.0	1400	1800	180	180	120
4	(infinite)	1400	1800	360	360	360

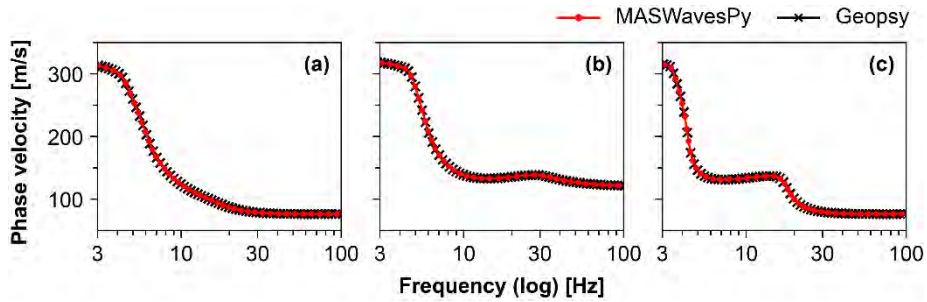


Figure 4. Comparison of theoretical dispersion curves computed with MASWavesPy and Geopsy for (a) model A, (b) model B, and (c) model C.

Comparison with invasive and non-invasive measurements of V_S

For further evaluation of the package, velocity profiles obtained using MASWavesPy were compared with results of invasive, non-invasive, and laboratory-scale measurements of V_S at four well-established geotechnical research sites in Norway. The Halden, Øysand and Tiller-Flotten sites were developed in the Norwegian GeoTest Site (NGTS) project [12]. The research site at Onsøy was established in the late 1960s [13]. The four sites are characterized by deposits of silt, soft clay, silty sand, and quick clay [12-16]. They are, therefore, considered representative of soft soil conditions commonly encountered in engineering practice.

The MASW data was acquired using a set of 24 vertical 4.5 Hz geophones with the receiver arrays placed as close as possible to the relevant invasive measurements. However, some differences may be expected in the V_S assessments as the invasive tests are point measurements whilst SWMs average the soil stiffness properties over a larger area. A certain degree of

variability is further associated with inverted V_S profiles, e.g., resulting from the non-uniqueness of the R-wave DC inversion and the choice of layering parameterization. The MASWavesPy data processing and analysis is described in Olafsdottir et al. [1]. An example illustrating the analysis process is given in Fig. 5, showing data and results from Tiller-Flotten.

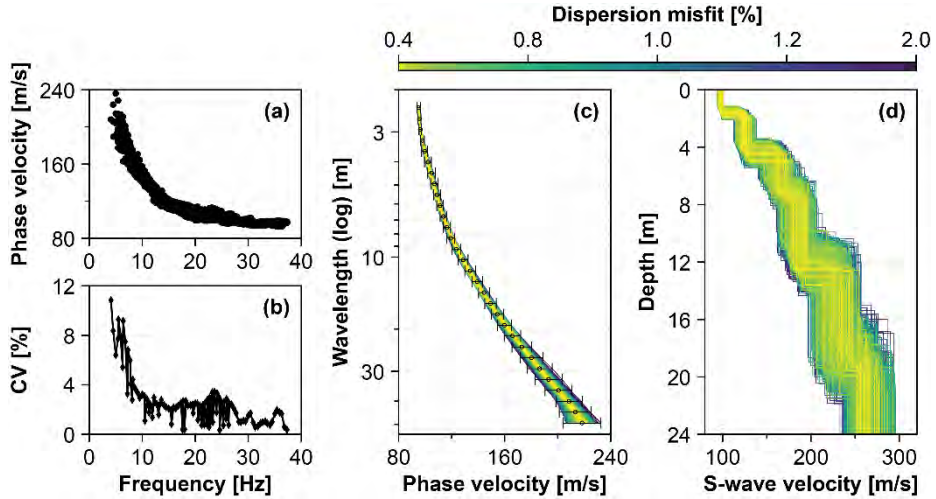


Figure 5. MASW survey at the Tiller-Flotten geotechnical research site. (a) Set of elementary DCs retrieved by processing repeated shot gathers collected at the site. (b) Coefficient of variation (CV) of the identified elementary DCs. (c,d) Sampled V_S profiles whose TDCs fall within one standard deviation (SD) of the composite DC at all wavelengths. The V_S profiles and associated TDCs are color-coded by dispersion misfit values. In (c), the composite DC (mean $\pm$ SD) is shown in black.

Figure 6 compares the resulting velocity profiles with the independent assessments of V_S for each of the four sites. The MASWavesPy results are summarized by the median of the set of sampled V_S models whose TDCs fall within one standard deviation of the composite DC. The lowest misfit V_S profile for each site is also shown. The comparison in Fig. 6 reveals that the V_S profiles obtained with MASWavesPy are, overall, consistent with those established with invasive techniques (SCPT and SDMT) and MASW surveys conducted with other hardware and software. They further show comparable values as the laboratory assessments of V_S available for the Halden, Onsøy and Øysand sites.

CONCLUSIONS

This paper presents MASWavesPy, an adaptable, open-source Python package to establish soil V_S profiles from MASW-type surface wave data. It is based on an earlier MATLAB tool created by the same authors, with more refined data processing and analysis methods for improved code usability. Furthermore, efforts were made to improve the computational performance of

the forward modelling to decrease the run-time of global search approaches in the inversion.

The wavefield transformation, DC extraction and forward modelling tools of MASWavesPy were verified by comparing their results with those of Geopsy, a widely known software for active-source and ambient vibration processing. The performance of the package as a whole was subsequently assessed by measurements at four geotechnical benchmark sites in Norway, where velocity profiles obtained with MASWavesPy were found to be consistent with those previously obtained at the same locations with a variety of in-situ and laboratory techniques.

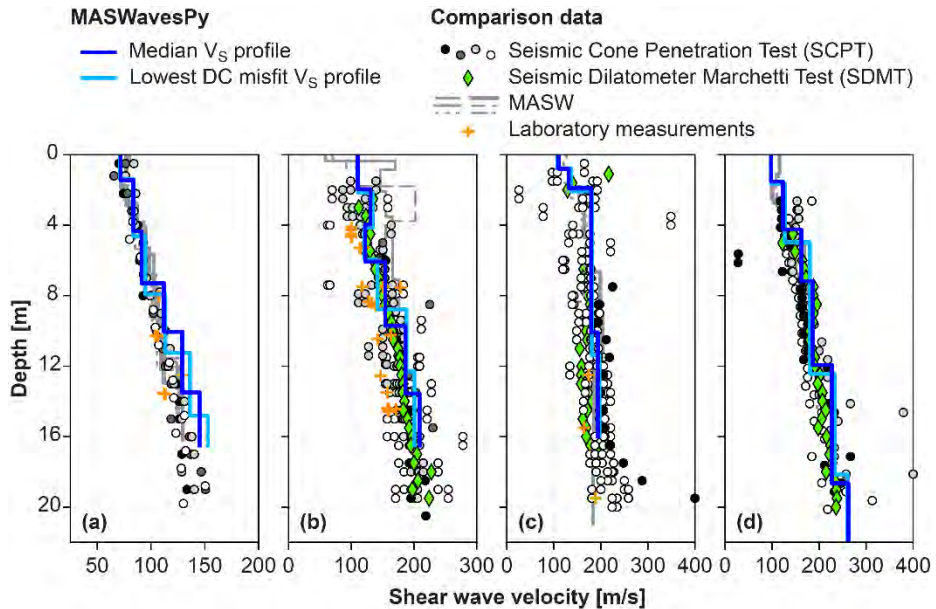


Figure 6. Comparison of inverted V_s profiles (MASWavesPy) and results of invasive, non-invasive and laboratory measurements of V_s for (a) Onsøy, (b) Halden, (c) Øysand, and (d) Tiller-Flotten. SCPT, SDMT, MASW and laboratory results from Blaker *et al.* [14], Quinteros *et al.* [15], L'Heureux, Lindgård, and Emdal [16], Long and Donohue [17], Bazin *et al.* [18], NGI [19], and NGTS data. Figure altered from Olafsdottir *et al.* [1].

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MEASURING STRENGTH INCREASE OF REMOULDED SENSITIVE CLAY FROM THE GJERDRUM LANDSLIDE, NORWAY

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KEYWORDS

Landslide, Sensitive clay, Undrained strength, Remoulded, Reconsolidation

ABSTRACT

The Gjerdrum landslide occurred in 2020 and involved highly sensitive clay (quick clay). After the landslide, a large deposit of slide material remained in the slide area. The remoulded clay initially had very low undrained shear strength, but strength increased due to consolidation, helped considerably by the installation of plastic vertical drains (PVDs). Determination of the strength of the remoulded slide material was vital to design the remediation measures. The objective of this work is to characterise the slide material and examine the changes in index properties over time.

The properties of the slide material are well documented; cone penetration testing (CPTu) and soil sampling were conducted in the slide material at two different times after the landslide. Undrained shear strengths based on fall cone tests and unconfined compression tests are reported in this paper. The undrained shear strength (c_u) increased with time for both testing methods.

INTRODUCTION

After the Gjerdrum landslide there remained a large volume of remoulded slide material in the slide area. The slide material in many places had extremely low remoulded strength (quick clay with $c_u < 0.5$ kPa [1,2]) and responded to loading in a liquid like manner. This remoulded clay made it difficult to access the slide area and to start remediation measures.

Given sufficient time and allowing for drainage of water, remoulded clay consolidates and gains strength. This increase in strength must be quantified to design the remediation measures.

This paper aims to characterise the Gjerdrum slide material and show the strength increase with time using results from standard index tests.

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GJERDRUM LANDSLIDE

Landslide event

In the middle of the night on 30th December 2020, a quick clay landslide occurred at Ask. The town of Ask is located in the municipality of Gjerdrum, approximately 30 km northeast of Oslo, Norway. The landslide killed eleven people, destroyed several homes and significantly damaged local infrastructure.

The slide release area had a length of 630 m and a width of 240 m. In total, the release volume was estimated to be 1.35 million cubic meters. The run-out area of the slide extended 2 km to the south due to the presence of quick clay [3].

Cause of the landslide

The cause of the landslide was later investigated by an independent commission [4]. The landslide was found to have initiated in a tall, steep slope approximately 400 m south of the residential area (Figure 11) and was triggered by erosion in the stream. The initial slide was probably small and triggered a series of larger retrogressive slides that moved up the slope and developed northwards and sideways, eventually reaching the residential area. Much of the slide material consisted of quick clay, and the soil structure collapsed upon failure, becoming liquid flowing out of the slide area.

Mitigation measures

The first mitigation measure was unloading of slope by removing soil from the landslide scarp. When the landslide area was assessed as safe enough to enter and slide debris from houses etc. were removed, plastic vertical drains (PVD) were installed in the landslide area south of the residential neighbourhood. PVDs were installed over an area of approximately 23 000 m², depicted in Figure 12, between April to August 2022. The purpose of the PVDs was to speed up consolidation of the slide material.

After installation of PVDs in the landslide area staged counterfills, consisting of crushed stone and clay, were constructed in the landslide area, allowing time for the remoulded slide material to consolidate between stages.



Figure 11. Aerial photograph of the landslide area in 2020 before the landslide occurred (source: Kartverket). The approximate location of the initial (triggering) slide is shown in green. The extent of the backscarp and run-out area on 5th January 2021 is shown in red, and the backscarp on 28th January 2022 is shown in blue. The runout area extends 2 km further to the south; thus, the full extent is not shown here.

Geotechnical site characterisation

The Ask area consists of a thick deposit of marine clay, with an upper layer of dry crust. Some areas have an upper layer of old fill material, either from levelling of farmland or due to the housing developments. Some limited site investigations (SI) had been conducted in the residential area prior to the

landslide, but none were conducted in the slope where the landslide was triggered.

Figure 12 presents the area of SI results presented in this paper. Multiple rounds of SI have been conducted [5-8], both in the surrounding vicinity and in the landslide area itself.

The slide material from the northern and eastern parts of the slide area were of most interest to engineers, as the landslide left tall scarps (up to 15 m) close to the remaining residential areas. The maximum thickness of the slide material in this area was around 10 meters.

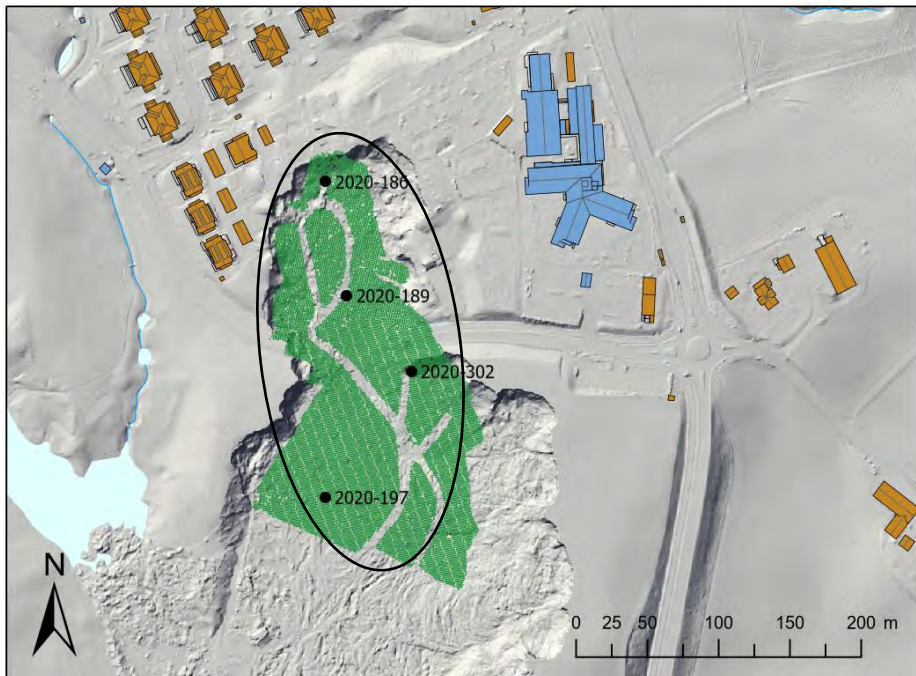


Figure 12. Digital terrain model created from laser scan conducted on 14th January 2021, two weeks after the landslide. The area with site investigation results included in this report is indicated with a black oval. Precise locations are provided for four specific boreholes that are discussed later in this article. Plastic vertical drains were installed in the green area in 2022.

In-situ testing and soil sampling within the slide area were conducted between August 2021 and February 2023. There are large inherent uncertainties in the response of slide material, e.g. how the properties of the remoulded material develops over time, as excess pore pressure dissipates. Changes in the properties of the slide material has been documented by conducting CPT and laboratory testing of the material at the same location at different times. The focus of this article is on the properties of the slide material and how they

develop over time, documented with index testing. Results of advanced testing and CPTu will be presented in later publications.

For brevity, only results from slide material are reported in this paper, where slide material is defined as soil that has been moved or remoulded during the landslide event. The slide material is a heterogeneous material that consists of remoulded clay/quick clay, dry crust and some intact blocks of clay/dry crust. The number of site investigations in slide material, within the area indicated in Figure 12 are as follows: 15 total soundings, 26 rotary pressure soundings, 60 CPTu and 17 series of piston samples.

Soil description

Soil samples were described based on visual inspection. The slide material was most often described as:

- clay,
- consistency varies from very soft to firm,
- colour is dark grey,
- often described as silty,
- some samples have inclusions of dry crust (Figure 13a),
- some samples have silt layers or layers of fine sand,
- some samples are described as laminated (Figure 13b).



Figure 13. Piston samples. Examples of (a) dry crust intrusions, (b) laminated clay.

Only one analysis of particle size distribution was conducted for the slide material, where the material was classified as clay: 35 % clay sized particles and 65 % silt sized particles, as measured by the falling drop method.

Index parameters

In general, there is a large spread in the results for index parameters, which is attributed to the heterogeneity of the slide material. The trends for index parameters are summarised in Table 4.

Table 4. Summary of index parameters for slide material

Index parameter	Figure number	Range	Comment
Water content (w)	4 (a)	21-40.7%	Decreases with depth, large variation at same depth
Unit weight (γ)	4 (b)	15.9–21 kN/m ³	Increases with depth
Atterberg limits and Plasticity index	5	PL: 17-24 LL: 24-40 PI: 6-20	
Undrained shear strength from fall cone (FC) tests (c_{ufc} and c_{urfc}) *	7		Large spread in undisturbed strengths (c_{ufc}). Remoulded strengths (c_{urfc}): 4 samples had strengths lower than 2 kPa, and were classified as <i>brittle</i> [†] . Sensitivity varied between 1 -80.
Undrained shear strength from unconfined compression (UC) tests (c_{uuc})	6	7-73 kPa	Increases with depth [‡] .

Note:

* Fall cone tests were conducted in accordance with NS 8015 [1].

† Norwegian regulations define stain softening clays or silts, with brittle failure mechanism as “brittle”, these materials have $c_{urfc} < 2$ kPa [2].

‡ These results were affected by the dilatant response of the slide material; in about 60 % of tests “failure” was defined at 15 % axial strain, because the sample continued to dilate and increase in strength past this point.

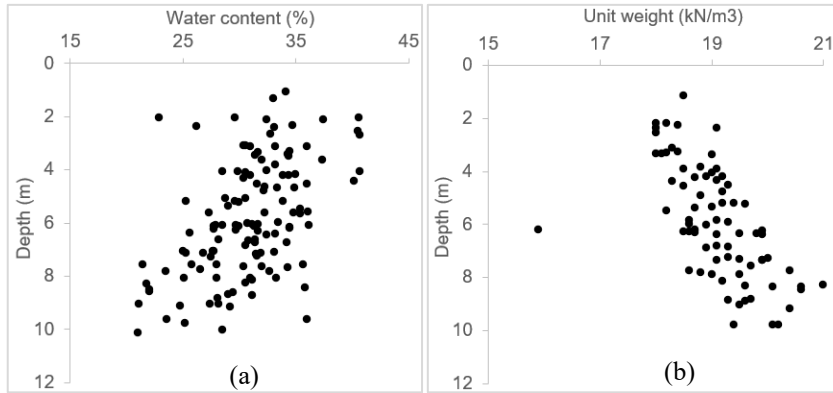


Figure 14. a) Water content (w) and b) unit weight (γ) for samples of slide material

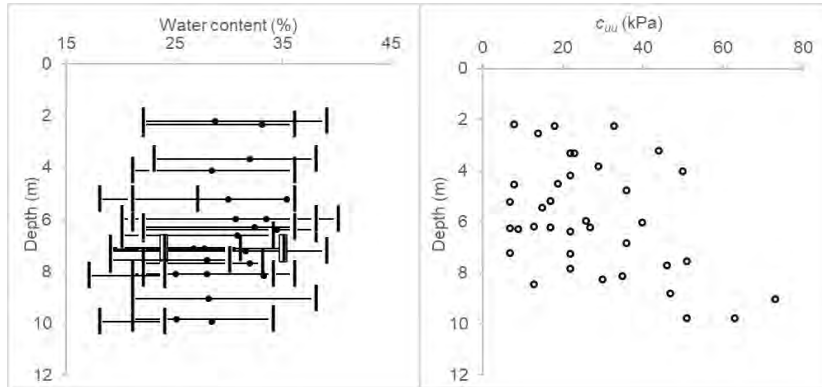


Figure 15. Atterberg limits and natural Water content for the slide material.

Figure 16. Unconfined compression test results (c_{uu}) from the slide material.

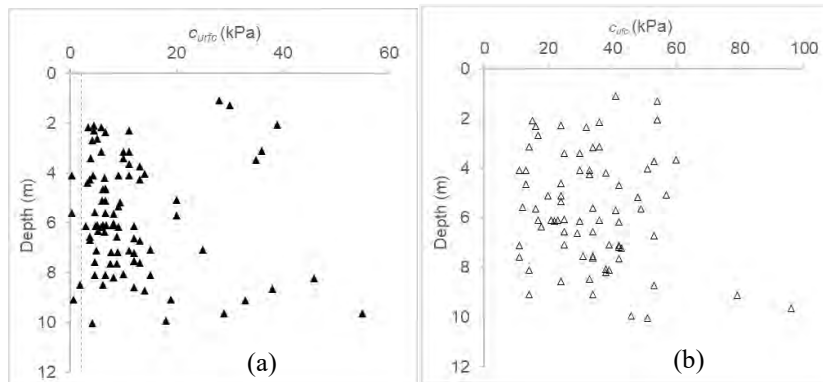


Figure 17. Undrained shear strength from fall cone tests in the slide material a) results for remoulded soil, b) results for undisturbed soil specimens in slide material. The

dashed vertical line in a) represents the definition of “brittle” material in the Norwegian regulations ($c_{urfc} < 2 \text{ kPa}$).

METHODS

Determination of slide surface depth

To evaluate the properties of the slide material, it was first necessary to determine where the slide surface was located. It was possible to localise the transition between slide material and intact clay level by interpreting CPTu results. Figure 18 shows an example of interpreted layering, where the grey layer is interpreted as slide material. Only the pore pressure response (u_2) is shown, and from the figure it is clear that the intact clay has a higher excess pore water pressure response than the remoulded material. This pore pressure response was also reflected by a sudden increase in the pore pressure parameter, B_q . Similar interpretation of CPTu-data has been used to determine the interface between intact and remoulded material, e.g. [9].

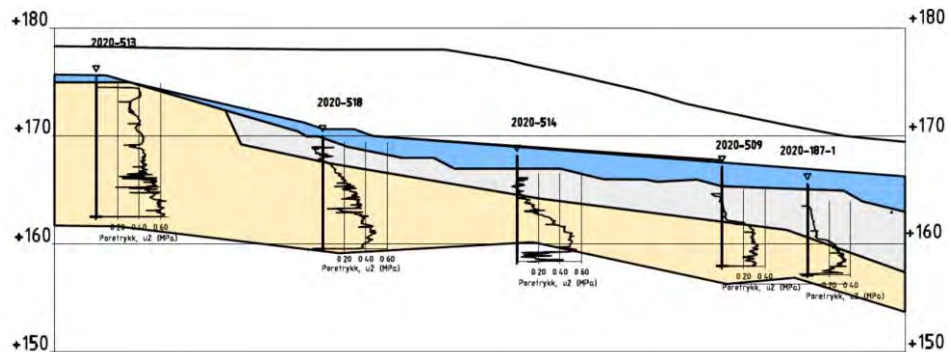


Figure 18. Example of terrain profile in the slide area. The yellow layer is clay (not quick clay, but still sensitive). The grey layer is the slide material. The blue layer is either counterfill or intact dry crust.

Interpretation of undrained strength

Interpretation of the pore water regime and the in-site effective stresses were necessary for the interpretation of CPTu. The on-site effective stresses were calculated based upon the closed available piezometer station, or a station at a similar elevation.

RESULTS AND DISCUSSION

The results for boreholes that have multiple rounds of sampling in the slide material are presented and discussed in this section. Suffixes to the borehole name signify a new round of site investigations and that the terrain level has changed, due to stabilisation measures such as the addition of counterfill.

Locations of the boreholes are presented in Figure 12, whilst more

information about the change in terrain level and date of installation of PVD is provided in *Table 5*.

Water content decrease with time

A decrease in water content is apparent in borehole 2020-186 (Figure 19a). This decrease in water content is expected, as the slide material consolidates, and excess pore water pressure dissipates. The other locations with repeat sampling, boreholes 2020-189, 2020-197 and 2020-302 (Figure 19b-d), do not exhibit a consistent reduction in water content. The most probable reason for this difference is the installation of PVD: only borehole 2020-186 had samples taken prior to installation of PVD, and thus the material in borehole 2020-189 was still under-consolidated for the samples in July 2022. For the remaining boreholes, the PVD have sped up consolidation, and by the time of the first round of sampling, primary consolidation was most likely already well underway or completed.

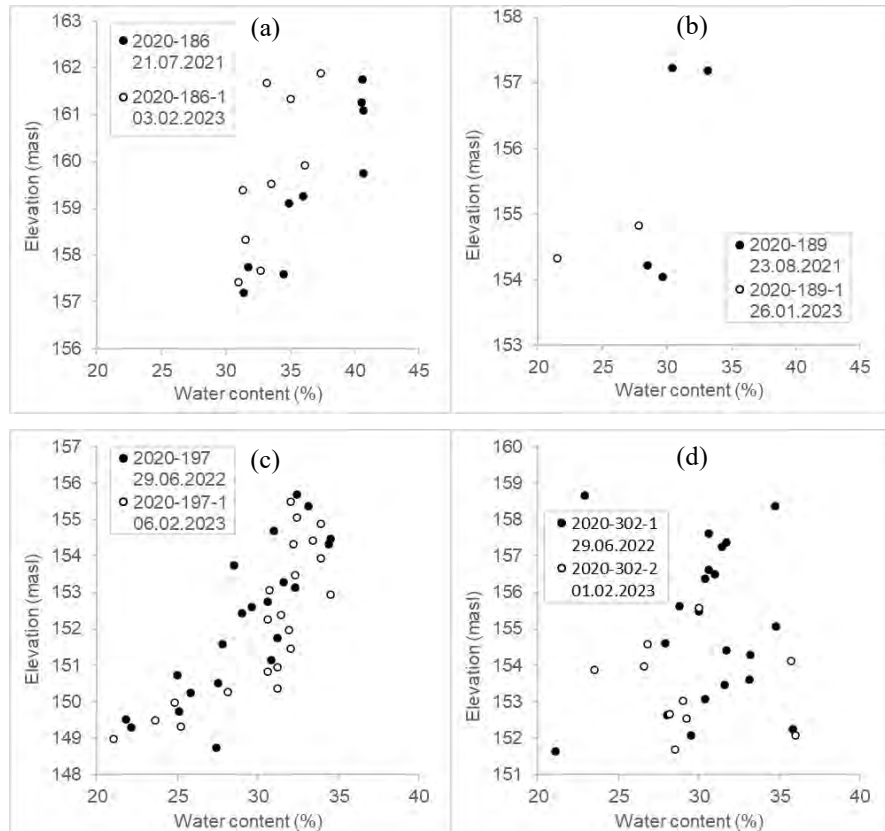


Figure 19. Change in water content with time for four borehole locations: a) 2020-186, b) 2020-189, c) 2020-197, d) 2020-302. The date of sampling is indicated for each sample.

Strength increase with time

The results of unconsolidated undrained (UC) tests are examined to check the increase in undrained strength with time. Repeat data was available at three boreholes: Figure 20.

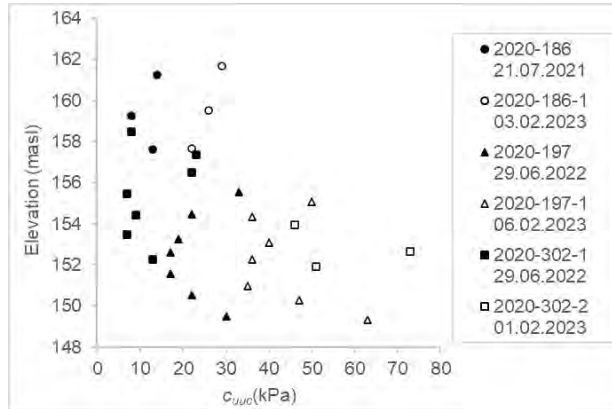


Figure 20. Undrained shear strength from unconsolidated, undrained tests on slide material against elevation. The date of sampling is indicated for each sample.

All samples displayed a strength increase of at least 10 kPa over time. Surprisingly, the samples with the longest time between samples (sampled at borehole 2020-186) resulted in smaller changes in undrained shear strength than samples with a longer time interval. Samples from borehole 2020-186 had a clear reduction in water content during this time period (Figure 19a), therefore it is unexpected that they exhibited the lowest increase in undrained shear strength. Possible reasons are a shorter time period with PVD, later application of the counterfill or differences in material type.

The change in undrained strength for boreholes 2020-197 and 2020-302 were larger, even though the time interval between sampling was smaller. These boreholes were located further south in the slide area and had the PVD installed earlier. Borehole 2020-302 also had a larger counterfill applied before the second round of sampling (Table 5). This indicates that the effective stress conditions should also be considered, and this hypothesis could be further strengthened by assessment of the pore pressure measurements at the time of sampling, and investigating when each counterfill was placed. Note also that borehole 2020-197 is the only borehole that is totally surrounded by PVD. Other boreholes were located on the edge of construction roads, where it was not suited to install PVDs.

Borehole number	Difference in terrain elevation between sampling dates (m)	Approximate date of PVD installation
2020-186 to 2020-186-1	1.10	29.08.2022
2020-189 to 2020-189-1	1.60	02.07.2022
2020-197 to 2020-197-1	1.30	04.05.2022
2020-302-1 to 2020-302-2	2.40	11.06.2022

Table 5. Changes in terrain level between sampling dates, and date of installation of PVDs in the vicinity of each borehole

CONCLUSIONS

This paper presented results for index tests conducted on slide material from the quick clay landslide at Ask, Gjerdrum. The soil samples demonstrated that the slide material was quite heterogeneous and varied between boreholes. The samples consisted mostly of clays, with some few samples having inclusions of dry crust. There were also some unexpectedly high fall cone (c_{ufc}) results: which indicated the presence of intact and undisturbed “blocks” within the otherwise remoulded quick clay. Only four samples displayed a typical quick clay response, with remoulded fall cone strengths (c_{urfc}) under 2 kPa. Whilst the slide material had a liquid-like response immediately post-slide, the remoulded slide material was no longer very brittle after reconsolidation.

Closer analyses of tests conducted at the same elevation and borehole, at differ times indicated that all samples displayed an increase in undrained shear strength (c_{uuc}). Although the unconfined compression (UC) tests dilated, the UC tests are fast and cheap, and carried out on all piston samples, therefore it is fast and readily available way to assess the undrained shear strength. Probably the largest changes in water content occurred prior to, and just after the installation of the plastic vertical drains (PVD): only the borehole with samples taken prior to the installation of PVD exhibited a clear decrease in water content between different sampling periods.

This paper presents a small portion of the site investigation data gathered from the slide material at Gjerdrum. Future publications will include results from advanced laboratory testing and interpretation of CPTu.

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METHODOLOGY FOR SAMPLE PREPARATION FOR QUALITY CONTROL OF STABILIZED DREDGED SEDIMENT

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KEYWORDS

Stabilization/Solidification, Dredged sediment, Quality control, Calorimetry, Electrical resistivity

ABSTRACT

Periodic dredging of harbors and other waterways is carried out to ensure sufficient depth for navigation. The Stabilization/Solidification method (S/S) is the global approach for improving the geotechnical characteristics and stabilizing pollutions in the low-compressive-strength dredged sediment (DS) for land reclamation. For this, different binders, such as cement, fly ash, and slag, are mixed with DS. The quality of mixing influences treated DS directly; therefore, this study investigated the effect of mixing time on the physical properties of treated DS, such as the unconfined compressive strength (UCS). Moreover, the potential for using electrical resistivity (ER) measurements and isothermal calorimetry (IC) tests to evaluate the mixing quality at the early stage were examined. Dredged sediments from the harbor of Stavanger, and Oslo in Norway, were mixed with binders using different water-binder ratios (w/b), and free-free-resonant (FFR) and UCS tests were performed to evaluate mixing time effects on the treated sediments. The results indicate that the higher the water content is, the higher the mixing time to reach the maximum compressive strength needs to be. The potential of ER and IC for quality control of treated DS at early stages was tested on one DS. It was found that these techniques have the potential to evaluate early-stage DS quality. The correlation between ER, CL, and UCS tests will be investigated in the future.

INTRODUCTION

Dredging is performed periodically in channels, ports, and rivers worldwide to maintain adequate depth for navigation, and consequently, large masses of

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sediments need to be taken care of annually. These sediments have high moisture content, low strength, high compressibility, and contain toxic compounds. Stabilization/Solidification (S/S) has been widely established as an effective procedure to manage contaminated dredged sediment.

In stabilization/solidification (S/S) projects, the procedure for mixing binders in the laboratory varies between different countries. In Japan and some other countries, it is recommended to mix for 10 min to have a homogenized mixture, while in Portugal, it is 3 min. In Sweden, 5 min of mixing is recommended to homogenize a mixture [1]. Yang et al. show that the unconfined compressive strength (UCS) of cement paste backfill (CPB) increases from 1 min up to 4 min of mixing time and then decreases for longer mixing times than 4 min [2]. Yaghoubi et al. showed that by increasing mixing from 5 min to 15 min, the UCS of stabilized soil was enhanced [3].

Although several studies have shown the effects of mixing time on mechanical properties and homogeneity of stabilized soil, there is still a need to standardize and harmonize mixing methods for stabilized soil applications.

The quality of treated DS is evaluated by both destructive and non-destructive tests that can be used both for laboratory and field samples. The 28-day unconfined compressive strength (UCS) is one of the destructive tests that can be performed on samples prepared at the laboratory or on the core samples that are taken from the site. During the 28-day waiting time before the results from the laboratory large amounts of stabilized soil may have been produced on-site. If the compressive strength of the 28-day samples fails to meet project specifications, the subsequent costs of re-stabilization or removal of the affected layers can become very high. Seismic-based testing methods such as the free-free resonance test, that categorized as a non-destructive method, have been employed for evaluating the quality of treated DS during the curing period [4], [5], [6]. However, a drawback of this approach is the time required for the treated soil to become sufficiently hardened to obtain the initial quality assessment results. Therefore, there is a need to develop more rapid and non-destructive methods to evaluate the quality of treated DS when it is still fresh.

Electrical resistivity (ER) and isothermal calorimetry (IC) measurements are two alternative non-destructive tests. The former measurement can be employed both in the field and laboratory to measure the electrical resistance of the treated DS, and the latter method measures the heat generated by the binder reactions. Some studies have shown that ER and IC could be correlated with compressive strength for cement mortar [7], [8].

This study focused on examining the influence of mixing time on the unconfined compressive strength of stabilized soil. The primary objective was to determine the optimal mixing time for laboratory procedures and examining two non-destructive tests (ER and IC) as alternative QC/QA control at early stage of stabilizing DS.

MATERIAL AND TEST METHOD

Materials

Two batches of dredged sediment with different water content were sampled in two harbors. Before the determination of water content and density, an electric paddle mortar mixer was used to homogenize each batch. Then, 8 samples were taken from each batch. The results are presented in table 1.

Batch number	Dredged sediment collection site	Water content		Density	
		Mean%	COV%	Mean (kg/m ³)	COV%
1	Stavanger harbor, Norway	349	1.98	1130	0.79
2	Oslo Harbor, Norway	88	0.9	1510	0.73

Table 1. Dredged sediment water content and density

The binders used for the mixing process were CEM III B for batch 1, a combination of 40% Portland limestone cement (PLC) and 60% ground granulated blast-furnace slag (GGBS) for batch 2.

Sample preparation and testing method

Mixing of DS and binder was by an electrical hand mixer for batch 1, and a KitchenAid Aristan stand mixer with a flat beater for batch 2. For each mix, 1 kg of the DS was weighed and mixed with the binders at different durations. To ensure thorough mixing with the KitchenAid mixer, the mixing was paused after 1 min, when material adhering to the flat beater and the inside of the bowl was scraped off, similarly to what is prescribed in EN 196-1 and ASTM C305. Batch 2 was sieved to eliminate grains with a diameter exceeding 4 mm to ensure that the particle size of the DS would be less than 1/10 of the inner diameter of the mold to provide more homogeneous raw material. Batch 1 was not sieved. The quantity of binders used to strengthen a DS can differ significantly based on the soil's condition and the project's needs. Typically, the required amount of binder falls between 80 and 200 kg/m³ for treated DS. Therefore, tests are needed to find the best binder dosage for stabilization with the target compressive strength. For batch 1, the calculated binder dosage was 100 kg per m³ of sediment, and for batch 2 we used 107 kg per m³ of sediment. Mixing times were 4, 9, and 14 minutes. Samples were prepared by pouring the treated dredged sediments into plastic tubes measuring 50 mm in diameter and 170 mm in height. The plastic tubes were filled in three layers, with each layer being tapped against the floor to

ensure all entrapped air came out. All specimens were placed in a water bath at 20 °C for one week. Following this, to conduct the 7-day FFR test, all specimens were removed and trimmed to achieve the height-to-diameter ratio of 2. Each specimen was placed in an individual plastic bag along with a moist tissue to prevent drying. The FFR test was conducted after 7, 14, and 28 days of curing, while the UCS test was performed after 28 days.

Electrical resistivity measurement was conducted to monitor the hydration process and assess the quality of samples during the curing phase of treated DS from batch 2. Accordingly, three water-to-binder ratios (w/b) were used: 4, 6, and 8. For each w/b ratio, two cylindrical samples, each with a diameter of 50 mm and a length of 170 mm, were prepared. These samples were subjected to electrical resistivity measurements using an instrumentation system developed by Dahlin et al., wherein the resistance of each sample to the flow of electric current was determined [9]. *Isothermal calorimetry* is a method used to quantify the thermal power generated by the hydration reactions of cementitious materials [10]. After the mixing of DS from batch 2 with binders, the samples were transferred into 20 mL plastic vials and sealed with plastic lids. Then, the vials were charged into the calorimetric equipment (TAM Air, Thermometric AB) to measure the heat production rate (thermal power) of the samples, from which the produced heat can be calculated for example 2 days or 1 week of curing to assess the quality of the treated DS.

RESULTS AND DISCUSSION

Effects of mixing time on compressive strength

Tables 2 and 3 show the *P*-wave velocity and compressive strength in relation to the mixing time for each batch. The finding indicates that by increasing dredged sediments' water content, to obtain higher compressive strength, it is needed to mix longer than in the situation where the dredged sediment has low water content. In batch 1, the water-binder ratio is 8; therefore, regarding the existing lower binder content in admixture, mixing less than 9 minutes decreased the compressive strength. In previous literature, the study showed that in the case of lower binder content, mixing time shorter than 10 min decreases the unconfined compressive strength [1]. As the mixing time increases, the mixing torque increases [11]; furthermore, the effectiveness of the mixing force transmitted by the blade depends on the viscosity of the material being mixed [12], and the viscosity of the material depends on water content. Thus, in materials with higher viscosity and low water content, less force is required from the blade to disperse the material because the DS can transmit most of the forces. Conversely, in DS with high water content and low viscosity, a portion of the force from the blade compensates due to the less stiffness of the mixture. Therefore, a higher force from the blade is needed to disperse the material effectively.

Mixing beyond 9 min decreases the compressive strength. One possible description for this phenomenon is that segregation between materials occurs by mixing for more than 9 minutes. In the case of batch 2, with low water content, 4 min mixing is enough to achieve the highest compressive strength.

The results in tables 2 and 3 also show that the coefficient of variance for UCS reduces when sieving raw material and mixing with a Kitchen Aid mixer with a flat blade and has a more homogeneous mixture compared to a kitchen hand mixer.

Table 2. Batch 1 FFR and UCS test results

Mixing time (minutes)	<i>P</i> wave velocity			Unconfined compressive strength		
	Days after treatment	Mean value of FFR (m/s)	Coefficient of variance (%)	Days after treatment	Mean value of UCS (kPa)	Coefficient of variance (%)
4	7	156	4	28	206	14
	14	239	2.3			
	28	329	4.9			
9	7	171	3.2	28	277	11.6
	14	264	3.5			
	28	346	3.5			
14	7	169	1.6	28	260	8.9
	14	263	2.2			
	28	338	2.1			

Table 3. Batch 2 FFR and UCS test results

Mixing time (minutes)	<i>P</i> wave velocity			Unconfined compressive strength		
	Days after treatment	Mean value of FFR (m/s)	Coefficient of variance (%)	Days after treatment	Mean value of UCS (kPa)	Coefficient of variance (%)
4	7	321	6.3	28	453	0.8
	14	500	3			
	28	721	1.6			
9	7	318	6.9	28	450	0.2
	14	485	1.3			
	28	709	0.4			
14	7	295	2.1	28	435	0.5
	14	461	2			
	28	684	1.2			

Potential of using calorimetry and electrical resistivity for early-stage quality control

Figure 1 shows V_p and 28-day compressive strength, respectively, with different water-binder ratios; as was expected, with an increase in the water-binder ratio, both the *P*-wave velocity and UCS decrease. Furthermore, the differences in compressive strength between a water-binder ratio of 4 and 6 are greater than the differences between a water-binder ratio of 6 and 8. The IC and ER measurements showed the same trend (fig. 3) as the UCS and FFR tests.

Figure 2 shows the Pearson's correlation heat-maps between FFR, UCS, ER, and heat release to examine the correlation coefficient through linear

regression between these measurements. The correlation coefficient, ranging from -1 to +1, indicates the strength and direction of the relationship between two variables. A value of +1 suggests a perfect positive correlation, -1 signifies a perfect negative correlation, while 0 indicates no correlation between the variables. For calorimetry, it is recommended to perform a correlation between measurements taken after 48 hours of curing or more. On the other hand, the coefficients suggest that it might be possible to correlate 48 or 72-hour measurement data from ER with UCS. The correlation between FFR tests at 7, 14, and 28 days and UCS tests is clearly strong. Consequently, there is a strong and positive correlation between V_p , ER, and heat release.

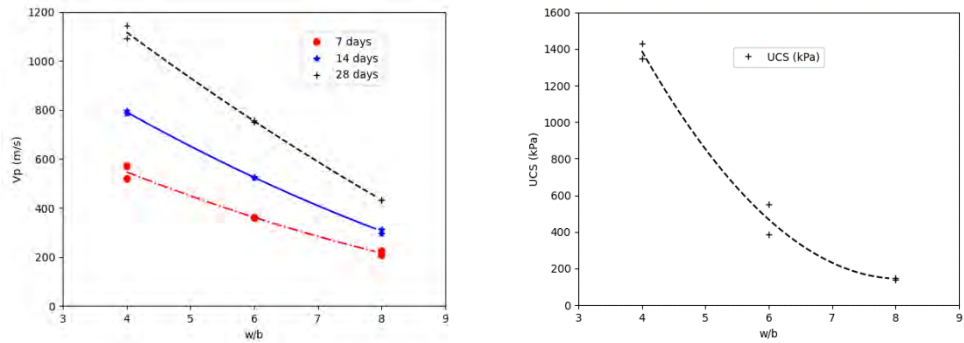


Figure 1 Left: V_p (m/s) against w/b , right: UCS (kPa) against w/b

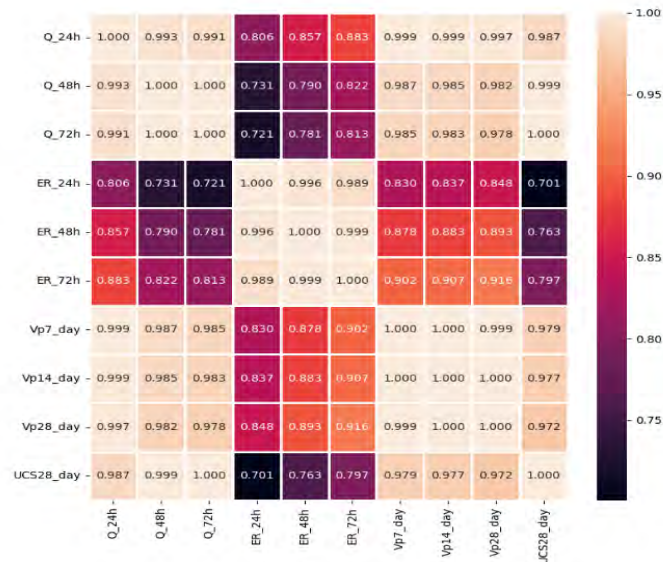


Figure 2 Pearson correlation coefficient

Figure 3 illustrates that with increasing curing time, both ER and heat release exhibit an upward trend. Conversely, as the water-to-binder (w/b) ratio increases, both ER and heat-release measurements decrease. This is attributed to the lower amount of binder, resulting in reduced reaction and consequently less heat release. Conversely, samples with a higher w/b ratio have more water content and less tortuous pathways, leading to a more conductive behavior.

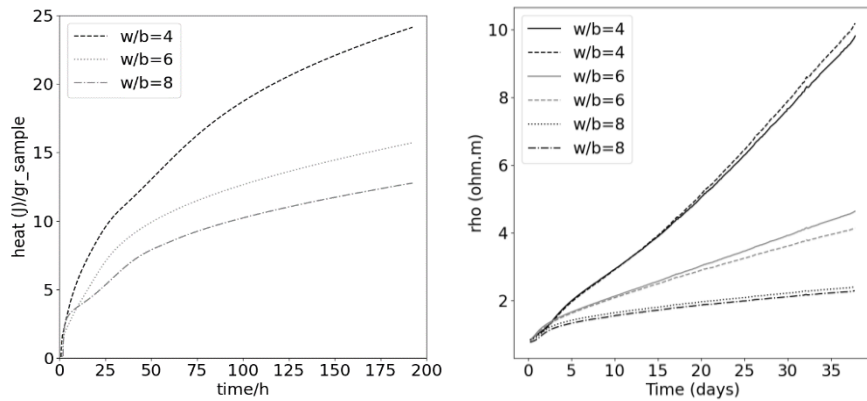


Figure 3 Left: Cumulative heat release per weight of sample versus time, right: measured electrical resistivity as a function of time.

CONCLUSIONS

In this study the effects of mixing time on the 28-days compressive strength of stabilized dredged sediment were examined, moreover two methods for quality controls of stabilized dredged sediment in early-stage were. The following conclusions were drawn from the analysis.

Mixture with high water content needs more mixing time to reach maximum compressive strength.

The use of a laboratory mixer with a flat beater blade results in less variability compared to a kitchen hand mixer with a smaller blade, resulting in reduced result variability.

To obtain less scattered results, it is recommended to sieve the dredged sediment (DS) using a sieve size that is 1/10th the diameter of the sampler. In this study, the sampler diameter was 50 mm, and the DS was sieved using a 4 mm sieve prior to mixing.

The results obtained from calorimetry and electrical resistivity tests demonstrate that these two methods can be employed on-site during the early

stages of material production to evaluate the quality of the treated material before it hardens, thereby enabling timely interventions and preventing costly repairs.

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MITIGATING VOLCANIC HAZARDS ON THE REYKJANES PENINSULA. LAVA FLOWS AND PROTECTION OF INFRASTRUCTURE, THE GRINDAVÍK SAGA

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KEYWORDS

NGM, Volcanic hazards, mitigation, lava flows

ABSTRACT

Geological evidence shows that volcanic activity on the Reykjanes peninsula follows a periodical pattern with several decades of dormant periods intercepted by shorter volcanic episodes. In 2021 the first eruption in approximately 780 years marked the onset of a new active period. The greatest activity has been near the town of Grindavík and the geothermal field in Svartsengi. To protect the town and power production installations a vast network of lava deflection barriers has been constructed. The barriers extend over 12 km with a fill volume in the range of 2,4 million m³. Since November 2023 lava flow from a series of five eruptions have successfully been deflected. Over 10 km of roads have been laid out on fresh lava fields only few days old and a network of central heating hot-water pipes, cold-water pipes and power lines have been successfully restored under very demanding conditions. This paper aims to tell the story of the eruptions and measures undertaken to either protect or reinstate the vital infrastructure, rather than to provide a full scientific account of the situation at hand. As of August 2024, the event is still ongoing, the next eruption is to be expected and the future remains uncertain for the people of Grindavík.

INTRODUCTION

Iceland is situated on the Mid-Atlantic ridge that forms the divide between the North American and Eurasian continental plates. The ridge is a divergent plate boundary where the two continents spread apart by roughly 2 cm per year. As the continents drift apart magma flows to the surface feeding a belt of volcanic systems stretching diagonally across the island. Iceland also hosts an additional uplift by a mantle plume that coincides with the divergent plate boundary. The oldest rock formations are about 15 million years old on the west and the east coasts. The youngest rocks follow the active volcanic belt

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coming ashore on the Reykjanes peninsula, crossing the central highland to the northeast coast where it connects the Jan Mayen ridge.

Iceland is in a way an infant with very active geology and associated earthquakes and volcanic eruptions, as well as other natural hazards such as landslides, avalanches, floods, storms, and recently, a variety of climate-change induced hazards. Since 2021 a total of 8 volcanic eruptions have occurred on the Reykjanes peninsula affecting greatly the population, especially in the town of Grindavík.

REYKJANES PENINSULA

The Reykjanes peninsula is a direct continuation of the Mid-Atlantic ridge. Series of volcanic systems lie along the peninsula in an echelon pattern under a 40-45° angle to the main rift axis. That angle is partly caused by the curvature of the Mid-Atlantic plate divide through the island and is a combination of a rift zone and a transform fault, or a leaking transform zone. It is a matter of an opinion how many volcanic systems are to be defined as individual systems along the peninsula and how well they are connected. For the current paper the systems are regarded to be six in total. The volcanic systems appear to follow a periodical behavior showing active periods of roughly 400–500 years and intersected with substantially longer dormant periods of 600–800 years [5],[8]. For the past three volcanic cycles at least four systems have been active, while the remaining two behave more sporadically. The latest active period on Reykjanes peninsula was from AD ~800–1240, and for the system closest to Grindavík from 1210 - 1240. Large portion of the peninsula are covered by lava formations that have been produced over the last 14,500 years, after the glacial retreat.

Inhabitants of the Reykjanes peninsula are ~30,000, with the majority living in several towns on the northwestern part of the peninsula. Grindavík is the only major settlement in the southwestern part with ~3,600 inhabitants prior to the evacuation of 2023 and is the second most valuable fishing harbor in Iceland. The hosts the Keflavík international airport and the well-known Blue lagoon geothermal spa. Two major power plants are located on the peninsula utilizing geothermal power to produce electricity and hot water for central heating for the region, one of them in Svartsengi close to Grindavík.

Reykjanes awakening

After 780 years of volcanic quiescence on the Reykjanes peninsula, an episode of seismic and later volcanic unrest started in December 2019 that is currently ongoing. Most of the activity has been concentrated around the mountain Fagradalsfjall and Svartsengi geothermal field. The seismic unrest started near Svartsengi 4 km north of Grindavík and later moved towards Fagradalsfjall where the first eruption occurred in 2021, [7]. This first eruption went on for over six months with rather moderate or small magma

flow of 7–15 m³/s and became a major tourist attraction. Petrographical analysis showed that the magma originated from a substantial depth of 10–15 km, [2], while the seismicity around Svartsengi was related to an accumulation in a potential magma reservoir at shallower depths, or some 5 km.

After 14 months of earthquake swarms associated with magma intrusions, possibly at shallow depth and with the general knowledge of the periodical volcanic behavior of the Reykjanes peninsula, the Department of Civil Protection and Emergency Management of the National Commissioner of the Icelandic Police (NCIP) (hereafter referred to as the Icelandic Civil Protection) put together a team of engineers and scientists to map out vital infrastructure that might be threatened or affected by potential eruption and to portray the possible mitigation methods. The work started in February 2021 with immediate analysis of previous eruption sites, extends of older lava flows, potential lava volume, flow rate and general behavior of the lavas. Narrowing down the most likely eruption sites, series of lava-flow simulations were carried out by several different methods and a plan laid out for barrier system that could compensate a selection of scenarios. The constructability, availability of material and suitable machinery had to be analyzed to assess realistic mitigation actions with regards to large uncertainty and most likely a very restricted time frame.

The eruptions at Fagradalsfjall were located well away from any infrastructure and the non-violent behavior of the first eruption provided a unique opportunity for Icelanders and tourists to witness a relatively small and confined eruption. Although not causing immediate threat to infrastructure, the vast number of visitors caused serious strain on the Icelandic emergency response system as thousands flocked towards the volcano in all weathers. The eruption also provided an opportunity to study the lava flows and to calibrate lava-flow modelling by real-time event. Furthermore, a series of trial barriers were constructed that gave immensely valuable experience for the upcoming events in 2023.

The first eruption at Fagradalsfjall lasted for 6 months. In 2022 and 2023 similar but smaller and shorter eruptions followed. Those required a significant response due to massive tourist interest and widespread moss fires that followed the 2023 eruption.

Increased activity near Svartsengi and Grindavík

On October 24th, 2023, yet another major earthquake swarm started north of Grindavík, with the center slightly NW of the power plant area in Svartsengi and the Blue Lagoon resort. This area was quite active prior to the volcanic eruption in Fagradalsfjall in 2021, but as events unfolded near Fagradalsfjall in 2021-2023 the Svartsengi area showed much lesser activity. Simultaneously fixed GPS stations showed increased horizontal and vertical

displacements indicating ground inflation that were confirmed by InSar satellite data. These events were soon interpreted by the Icelandic Meteorological Office (IMO) and other scientific parties as a magma intrusion that progressed with continuous surface inflation and varying seismicity, [3],[4]. On November 10th the seismicity increased significantly along with extensive surface fracturing, subsidence and horizontal displacement in the town of Grindavík. An evacuation was implemented that afternoon although many of the inhabitants of Grindavík had by this time already left town due to the constant unrest. The town has been formally evacuated ever since (in August 2024). In few households the inhabitants have chosen to stay in Grindavík and some workplaces, mainly around the harbor, are still in function.

It has been suggested that these events on November 10th were caused by a major dyke intrusion, extending over 15 km in length and at depth between 1 and 5 km, [6]. The dyke intrusion caused widespread tectonic displacement in Grindavík, at least ~6-7 fault zones intersect the town, some with a vertical displacement of 1,0 m and horizontal displacement of 1,2 m. A network of open fractures cut through the town, destroying tens of houses, streets and affecting all vital utilities.

During the following weeks following November 10th a continuous surface inflation associated with varying seismic activity was monitored through a vast network of fixed GPS stations that have been in operation on the Reykjanes peninsula. On December 18th, the first of many subsequent eruptions along the Sundhnjúkar crater row began.

Drawing from the earlier preparation work in 2021 and the intensity of the uplift rate and a series of three very recent eruptions in Fagradalsfjall, construction work on lava deflection barriers was commenced on November 10th, the very same day as the major rupturing event in Grindavík. The first and foremost priority was to secure, if possible, the power plant location at Svartsengi which is essentially the only source of geothermal extraction for central heating for the towns spread out on the Reykjanes peninsula. The power plant is set up with geothermal turbines as well with 75 MW electrical generation along with 150 MW of geothermal water. The spill water from the power plant is furthermore the main geothermal water source for the Blue Lagoon geothermal resort situated few hundred meters away. Both installations are sitting in a topographical depression or shallow basin surrounded by lavas formed during the last two volcanic episodes on the peninsula. The magnitude and hastiness of this construction exceeded all conventional constructional and planning laws, requiring a special bill to be passed through the Parliament with great urgency in the days before.

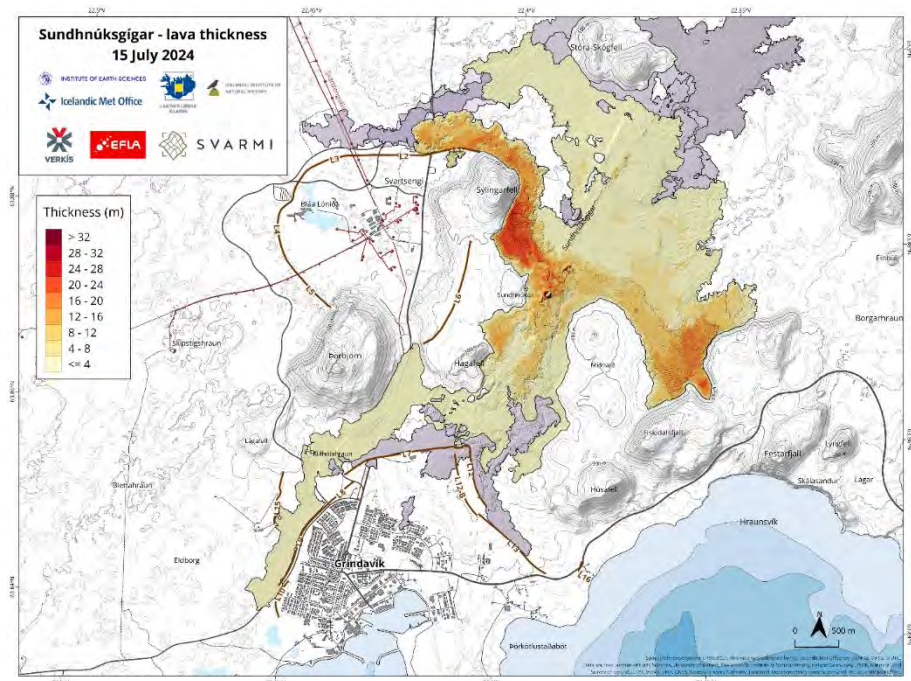


Figure 1. Overview of Svartsengi and Grindavík, status of lava flows in July 2024.

The Svartsengi barrier system

As the focal point of uplift and earthquakes coincided with the power plant location it was not known if the mitigation plans had to consider both previous eruption sites located at Eldvörp and Sundhnjúkar crater rows west and east of the Svartsengi basin, respectively, or each one of them individually. The Eldvörp site was active with at least three eruptions in the Middle Ages from 1210-1240 and the Sundhnjúkar crater row that produced vast lava fields surrounding the town of Grindavík and the power plant site was active some 2,400 years ago.

It became quite evident that the Svartsengi basin had to be defended by series of deflecting barriers covering an almost circular envelope around the basin. As both literature study, lava flow modelling and work on the trial barriers in 2021 had shown, the plan had to be to convey or deflect the lava flow to a lower topographical level where it could spread out and accumulate without affecting the heart of the power plant. By doing so it had to be accepted that other important utilities as roads, power lines, hot-water and cold-water pipes would potentially be overrun by new lava. The barriers around the Svartsengi basin, named L1-L6 extended over 5.5 km with varying height from 8-13 m above the surrounding ground level, see figure 1. Estimated volumes for initial design involved some 600,000 m³ of earth to be piled up. The “lava” side of the barriers were formed with a slope of 1:1.5 (vertical:horizontal),

with a top width of 4 m and an “air” side (leeward) of 1:2. Later this basic form was modified as many barriers were elevated or adjusted to suit the ever changing topography.

The first phase of the construction involved immediate barriers that are crudely piled up to provide a first line of defense along the entire barrier system. The fill material was sourced by large bulldozers from the older lava formation immediately along the “outside” of the barrier line and hauled in from a gravel mine some 15 km away by regular lorries. The initial bulldozed fill extended generally some 3–5 m above the intact ground elevation providing a base for the top fill to be hauled in by dump trucks and placed by large excavators shaping the barriers. The first line of defenses was largely completed on December 18th, apart from several gaps or passes in the system where the barriers crossed major roads and utility paths.

December 18th eruption - 2023

This first major eruption on the evening of December 18th proved to be quite dramatic in the opening phase compared to the previous eruptions at Fagradalsfjall. In a matter of minutes, a 4-km-long eruptive fissure opened. The crater line extended from Stóra-Skógfell in the north and towards Hagafell in the south, crossing the tectonic plate boundary but following the same orientation as the Sundhnjúkar crater row which was active 2,400 years ago. In the first few hours the outermost ends receded quite fast and after the first day the lava flow was confined only to center part of the crater row slightly north of the old Sundhnjúkar main crater. The immediate response was to close the road passes on the barriers closest to the eruption, which was achieved within few hours. But as the first night went off with receding magma outflow it became evident that a direct contact with the barriers or some vital infrastructure was not imminent. On December 21st the eruption ceased with new lava covering an area of 3,5 km². Estimated lava volume was 20 million m³, which is rather small in relation to older lava formations in the vicinity. After an evident ground deflation at Svartsengi during the eruption a distinct uplift was noted immediately, concurring with continued inflation in the magma domain resting underneath Svartsengi, indicating that these events were potentially only the beginning of a larger and more persisting event.

Towards Grindavík – January 2024

Work on the Svartsengi barriers continued towards the Christmas holidays. By that time no decision had been made about the protection of the town of Grindavík. Preparation work with lava-flow simulation and layout design for Grindavík continued along with the barrier construction at Svartsengi. The first plans involved a U-shaped envelope around the town, close to 6.5 km long with a fill -volume of close to 1 million m³.

With ongoing inflation in the magma domain and the December eruption situated close to the town an order was given to commence construction of the first Grindavík barrier named L7 on January 2nd, 2024, at the northern border of the town. By that time the uplift at Svartsengi had reached similar levels as for December 18th eruption. The barrier started at the watershed northeast of the town and extended 1.7 km diagonally towards southwest crossing the main road to Grindavík. All major utilities as central heating, cold water, power cables and communications followed the same road, resulting in a 150 m open gap in the barrier where it crossed the road. The barrier layout aimed to lead the potential lava flow under moderate gradient towards west, where the road would later be moved to allow for the road to swing by the tip of the barrier through an overlap with next barrier, L8 and a topographical threshold heading towards the town.

An old gravel pit resting under the south slopes of Hagafell had been put in use by this time allowing for much shorter hauling distance and greater fill capacity. With the continued pressure increase in the magma domain a great urgency was put on the work and by the first week of January an immediate first crude barrier line was in place. The northeastern part was substantially higher than the immediate plan called for as the bulldozers could pile up that part easily, whereas fill material to the middle and westernmost part of the barrier had to be hauled in.

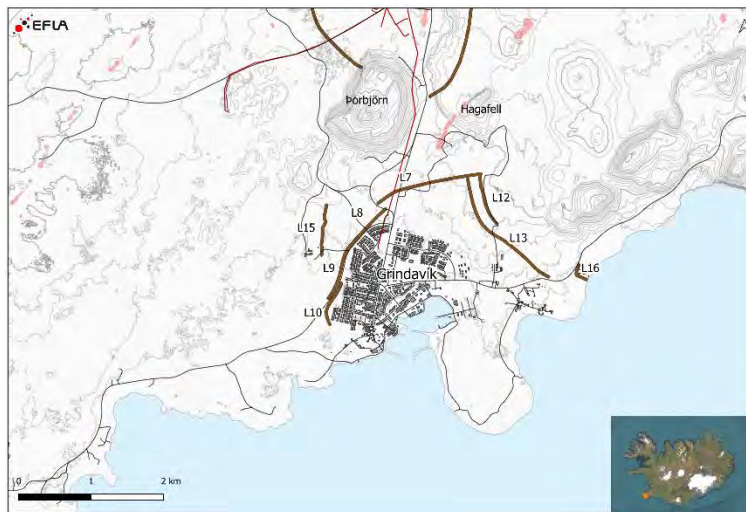


Figure2. The Grindavík barrier system, old volcanic craters shown with red polygons.

The road and pipe crossing called for some special measures as the hot water main to Grindavík was a surface pipe. Since the preceding summer a new hot-water main and cold-water main had been under construction. Both pipes were planned as underground pipes in the same trench, to replace the older surface pipe and a fragile cold-water pipe. As the situation unfolded in

October–November 2023, work on the pipes was abandoned due to safety concerns by the privately owned utility company. Work on the pipes was again well underway, the cold-water main had been put in use, while the new hot-water main lay still in the open ditch with only the final touches remaining before the trench could be closed. That was the situation north of Grindavík in the second week of the new year.

January 14th eruption

On Sunday morning January 14th the second eruption started, preceded by early warning signs by increased seismicity, tectonic displacement and increased surface fracturing in Grindavík. This time the eruptive vents were located south of Hagafell on a fissure system slightly east to the fissure system associated with the crater row active in December. The new crater row extended through the L7 barrier southward, with one of the main vents located directly in the barrier. As the barrier defined a flow path with sufficient gradient, most of the lava followed the barrier to the west instead of taking the otherwise more direct path towards the town.

As the eruption started on a Sunday morning the construction team had a rare day off. The construction machinery stood parked on top of the barrier few hundred meters away from the craters, including all the heaviest bulldozers and excavators, which are the most vital components of the machine park. The lava flow was rather moderate compared to what would later be seen in this series of events and the main lava front moved only with a speed of brisk hiking pace. The contractors were obviously eager to get in and save their machines. The Regional Command Center situated outside the area and with a limited view of the situation denied that request. After a prolonged communication between the construction site supervision and the Regional Command, and helicopter fly-over to assess the situation a permission was granted to save the equipment and start to fill in the gap over the main road, that would otherwise provide an open path for the lava into the town. Work on closing the road gap started immediately with all available machines operating in a confined area with great haste. The final buckets were being shoveled as the lava front reached the road. Again, drawing from the work on the trial barriers in 2021 when work on actively steering the lava front along the main deflecting barrier near Stórhóll gave important lessons in working around the hot moving lava. Soon afterwards the surface hot water pipe broke, but as the pumps had been turned off, there were no steam explosions. To save the cold-water pipes, fire hydrants in the town were opened to allow the water flowing along the pipe to cool off the skin and hopefully save the pipe. Work in the road pass was abandoned when the lava front passed by on the request of the Regional Command, leaving the hastily piled up fill, substantially lower than the remainder of the barrier. As combined result of the lava front interaction with the hot-water pipe and significantly lower fill

over the road a minor overflow occurred that could otherwise be most likely be avoided.

During the eruption a small eruptive vent opened inside the town border producing a small lava field that engulfed one house and set two other houses on fire. It has been suggested that the lava travelling laterally along open surface fissures at shallow depth and emerging to the surface only hundred meters from the northernmost houses in Grindavík.

As the day and next night went on, emphasis was put on to pile up an immediate barrier along the northwestern and western border of the town along a road following the outskirts of the town. After some 12 hours the flow rate declined and on the following morning the lava front had come to a rest. The eruption continued, gradually decreasing and on January 16th the eruption was over.

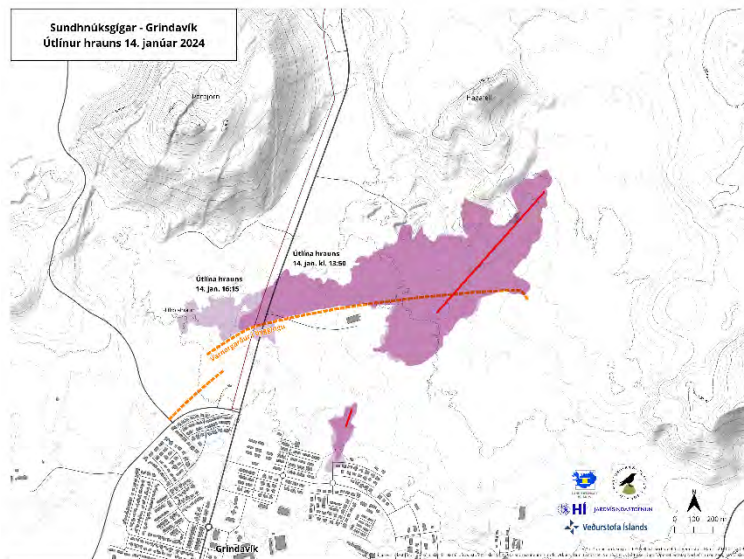


Figure 3. The January lava

January eruption aftermath

This short-lived eruption caused a significant havoc although being rather small in all comparison, covering only 0.7 km^2 and with volume of 2.8 million m^3 . The main road to Grindavík was covered by new lava, the town was without central heating and in three days after the eruption the power cables submerged by lava gave away. The cold-water pipe was conveying water, but the minor lava overflow overran the control station for the water distribution destroying it completely. One week later an overhead transmission line had been erected. The new hot-water pipe was connected, and conveying water, but without pressure buildup caused by leaking heat expanders. Few days after the eruption, work on extracting the cold-water supply from underneath

the lava started. There were some successes as the pipe conveyed water, but some damages had occurred. In the end an old asbestos pipe lying on the ground underneath the lava was used as a conduit after being cooled down and new pipe fitted through. Work on trying to repair the leaking hot-water pipe was abandoned and a section of older steel pipe was brought in and placed over the surface of the new lava.

The latter half of January entered with rather strange mood for the people on site. The town had now been completely evacuated by the State Police Commissioner under a special clause in the Civil Protection laws, following a fatal accident in the town and a great uncertainty on the status of fault systems and possible hidden fractures og bedrock weaknesses that could potentially collapse and open without warning. There was also uncertainty regarding the continuation of the barrier construction work, if further action to prevent lava from flowing into Grindavík were to be justified and approved by the government. In the meantime, work on completing the Svartsengi system continued along with repair work. Preparations for barriers along the eastern part of the town were made along with plans for elevating the L7 barrier that took on the January lava flow. That involved installing construction roads and loosening materials from the gravel pit by Hagafell.

On February 4th an order was given for continued barrier work with elevation of L7 north of Grindavík and adjusted barrier along the eastern part of the town, named L12 and L13. The eastern barriers followed slightly diagonally and partly through the watershed formed by an 8,500 years old Hópsnes lava formation. The easternmost tip of L7 was cut off to allow the northern tip of L12 to connect into the eastern boundary of the January lava and facilitate a natural pathway for future lava flows that would likely follow the outskirts of the January lava.

Geotechnical issues crossing tectonic fractures

Associated with the January 14th eruption a significant tectonic displacement affected Grindavík. That involved especially the main fissure line which the eruptive vents followed into the town, forming a small graben in the eastern part of the town. The previous displacement associated with the November 10th dyke intrusion formed a 4.5 km wide graben in the western part of the town. Between those grabens is a 200–300 m wide horst formation [1]. Prior to the January eruption measures had been taken to repair some of the most important streets within the town. The most significant faulting system following a known fracture by the name Stamphólsgjá was activated severely during the November event with vertical displacement of 1.2 m, horizontal displacement of up to 1 m and several tens of meter deep. The conventional way in Grindavík and mostly elsewhere in Iceland has been to fill in the fractures after seismic events. As a long-term measure with expected further ground movements that was seen as rather short-lived method. Instead, the

most critical crossings were repaired with the aid of geotextiles and reinforced with geonet. After rounding off the bedrock and filling in the bedrock fracture as practicable a layering of reinforcement was placed over the fractures providing a ductile pass way. The main function of the geotextile is to keep the fill material in place and thus providing an arching effect over the fractures. Several of these had been constructed prior to the January displacement and all performed very well providing a safe passage through the town.



Figure 4. Grindavík and the fracture systems, yellow and orange.

February 8th eruption,

At 5:30 on Thursday morning February 8th an intense seismicity started northeast of Sýlingafell along the Sundhnjúkar crater row. About 30 minutes later an eruption started on approximately 3 km long fissure following similar location as in December. The lava flow quickly formed a distinctive front towards west and monitoring through web cams and drone surveillance showed the lava front moving quite fast along towards Stóra-Skógfell where it accumulated for a bit before taking a sharp turn towards Svartsengi. The new lava followed a defined channel in a depression formed by the boundaries of two older lava formations. The lava front progressed at a pace of 600–700 m/hr and by 10:17 it had reached the main road to Grindavík, north of Svartsengi. By 12:00 it reached the utility corridor from Svartsengi connecting all towns on the northern part of the peninsula. By 13:30 the lava front reached the maximum extension towards west, then it had travelled some 4.5 km in 6 hours. Simultaneously the eruption receded with lava fountaining confined to three active vents, and the following day, February

9th, the eruption ceased. The February lava covered 4,2 km² and with a lava volume of approximately 15 million m³.

Work on protection measures for the very important hot-water pipe from Svartsengi, which provides geothermal water for central heating for the entire Reykjanes peninsula, started in January. The main pipe was as for Grindavík a surface installation and as such very susceptible to lava flows. As the project stood in February, some 500 m of new Ø700 mm pipe had been laid down in a trench crossing the most obvious depression and potential flow path. Work on the pipe had not been completed and the trench was still open when the eruption began. The construction teams started immediately to cover the pipe with sand and protective gravel fill. As the lava front progressed work on closing the road passages in the barriers also went on. The surface pipe ruptured almost instantly as the lava hit and carried the remainder of the pipe downstream with a great plume of steam rising from the lava. Condition on the new Ø700 mm pipe remained uncertain. Consequently 30,000 inhabitants on the Reykjanes peninsula were without central heating in subzero winter conditions.

The main cold-water supply for Svartsengi power plant is located 3.5 km north of Svartsengi and the pipes follow the same corridor as the hot-water pipes north of Svartsengi. This water supply is essential to operations in Svartsengi as it is the only source of cooling water and after running through heat exchanger it is returned as hot water for central heating, while the remaining brine is discarded. To protect the cold-water pipes under the new lava a continuous flow was maintained to allow proper cooling of the pipes.

A major high voltage overhead transmission line also follows the same corridor, connecting Svartsengi power plant to the main grid. Two of the most exposed transmission towers had been protected by piling up fill material against the towers. To avoid uncontrolled electrical outage the transmission line was shut off temporarily to reduce sagging as the heat flux from the running lava affects the conductors significantly. These measures were essential as by 17:00, when the heat flux had receded, the transmission line was connected again to the main grid without interruption although 6–7 m thick fresh lava surrounded two towers.

Restoring infrastructure

The largest blow to the infrastructure was the loss of geothermal power supply to the towns on the Reykjanes peninsula, the international airport included. The main road to Grindavík was also cut off resulting in 1 hour long diversion along the nearest roads. As the lava front slowed, work on a construction road began immediately to connect the work sites on either side of the new lava. It was anticipated that the new Ø700 mm pipe had survived as the pipe had been mostly covered before being overrun with new lava. When the protruding northern end of that pipe was inspected a significant

lengthening of the pipe had already occurred buckling the end through the new lava. Geothermal pipes are constructed with heat expanders to facilitate pipe expansion when heated to 80°C. The first alternative was to connect the part of this new pipe, now covered by lava with remains of the older surface pipe. The connection was completed on Friday afternoon, but when water started to run through the pipe the water temperature rose towards 100°C at the north side of the lava, while being pumped in at the other end at 80°C. The flow maintained for few hours until the pipe broke in the evening, rendering hope for a quick repair.

In the subzero temperatures it was foreseen that it would only be a matter of 2–3 days before a significant and widespread frost damages would set in as electrical installation in the towns were not capable of running all households by electrical heating. With ~30.000 inhabitants without central heating, widespread actions were taken to safeguard public and private installations against frost damages as plans were drafted on how to reconnect the hot-water connection over the new lava. On Saturday morning, only 48 hours after the eruption started, a suitable Ø500 mm steel pipe material had been located. Welding and auxiliary crews were mobilized to facilitate a 24 hrs operation on two welding platforms. The pipe would be welded into length of two 250 m sections to be moved over the new lava if possible. The lava crossing was very uncertain as by this time there were only 48 hours since the lava crossed the pipe corridor. After careful inspection and drone surveying a small 17-ton bulldozer started to level the scoriated surface crust. The crust was about 50 cm thick; surface temperature of the lava was ranging from 200–250°C and through cracks in the crust temperatures over 650°C could be measured at 50 cm depth. The lava surface was quite level, thickest at 7 m and gradually thinning towards either side, with this thickness solidification of the lava takes many months up to a year. The surface crust has a certain bearing capacity, but a liquid core remained underneath. Despite the general believe, the levelling went quite smoothly and after only 2 hours on the new lava the bulldozer was through. Dump trucks waited on the other end ready to place fill material over the levelled crust scoria. Upon completing a “road” connection over the new lava it became certain that the new temporary pipe section could be pulled over the new lava. By Sunday afternoon the new pipe section was installed and connected to the still surviving parts of the main pipe and run up of central heating commenced.

On the main road to Grindavík a 300 m long section was now covered by new lava. Drone surveying showed that this part of the lava was only 2.5–3.0 m thick, and therefore expected to solidify in few days. Work on levelling the top started on Saturday and on Sunday a useable construction road was in place. Work on the road continued through the next days, by levelling side areas and finalizing surface course for public use. On Wednesday afternoon, only six days after the eruption started, the road was opened for public use.

March 16th eruption

Along with the infrastructure restoration after the February eruption, work on the Grindavík barriers continued. Inflation of the magma domain showed the same tendency as before; therefore, a new eruption was to be expected within few weeks. The priority was to elevate the L7 barrier north of Grindavík and the barriers at the eastern border. On March 2nd a dyke intrusion occurred without eruption and without affecting magma accumulation significantly. Two weeks later, on a Saturday evening the March 16th, a fissure eruption started at a similar location as in December and February. During the first hours the lava flow was estimated to be 1100–1200 m³/s. After 3–8 hours the lava flow had receded to 100 m³/s before settling down to 15 m³/s on the second day, gradually decreasing thereafter. The lava fronts followed similar paths as before, a western lobe running towards Svartsengi, covering the main road to Grindavík again before coming at rest close to the utility corridor. Another and more significant front flowed south towards Grindavík. That front followed previous formation but were directed by the January crater row and then by the recently constructed L12 and L13 barriers away from the town before halting some 400 m from the coastal road east of Grindavík. The main road to Grindavík was restored 5 days later again over the new lava.

After the initial phase the newest lava covered 6 km² and expanded only slightly over the next weeks although its volume increased. The eruption continued until May 8th and was the longest eruption this winter with estimated lava volume of 34 million m³. When the first intense phase was over and the eruption settled into a similar pace as seen during Fagradalsfjall 2021–23 events, the fluid lava started to accumulate in the near vicinity of the craters. The lava accumulated in lava ponds and large open channels with a gradual building up on top of earlier flows. Lower magma flow from the crater does not sustain lava flow over long distance as with reduced effusion the temperature of the running lava is lowered and with more exposure it loses gases resulting in a more viscous mass. After few days of lava accumulation south of the main vent, a thick viscous and slowly moving lava front approached the north corner where barriers L7 and L12 met. This slow flowing mass carried large blocks of solidified lava, up to 5 m in diameter. On average the crust appeared to be in the range of 3–4m thick. As the lava touched the barriers it stood 3 m above the top and rolled like a belt on a bulldozer along the barriers. Blocks touching the barrier came at rest, while there was a continuous movement further out in the main flow, some 15–20 m away from the barrier. The following days the flow continued by filling completely the precious gravel pit under Hagafell. During Easter holidays another change happened as small lava outlets began to squeeze out higher up in the system along with rising of the surface crust. That development continued downward along the system, indicating that the lava was flowing slowly in connected tube system under the crust. During the second week of

April the expansion of the lava had reached the L12 barrier and lifted the few weeks old surface lava even higher than already seen, building up a pile of large blocks some 5 m above the barrier. Still only a one minor overflow resulted by squeezing. At the same time the first geotechnical deformation of the barrier occurred, when a large flat block was lifted by the expansion, scraped the inside slope until standing vertical near the top of the barrier pushing the top outwards. This deformation only affected the topmost 1 m of the 4 m wide barrier top with an expanding lava approximately 5 m higher than the barrier, thrusting the now vertical block outwards like a giant bulldozer blade.

May 29th eruption

On Wednesday May 29th the fifth eruption on the Sundhnjúkar crater row started. The initial phase of the eruption was even more intense than the previous ones. Some estimations have been put forward for lava flow up to 1.500-2,000 m³/s (Icelandic Met Office, 2023-2024). The lava flow followed similar patterns as before since the main vents were close to the December, February, and March eruption sites. The main flow headed towards Grindavík and was led by the older lava formations and barriers along the northern and western border of the town, while less significant fronts flowed towards Svartsengi and east of Grindavík. The lava flow was extremely liquid and fast flowing and reached the maximum length at 5.6 km west of Grindavík in ~4–5 hours. The immediate effect of this initial phase was that all roads connecting Grindavík from north and west were covered by the new lava, leaving only one major road connection functioning towards east.

As for the March eruption this initial phase receded fast and on the second day of the eruption all fronts were standing still, and the lava started to accumulate close to the craters. Eventually on June 8th the lava accumulation northwest of the main crater succumbed, and a viscous slow-moving front developed towards Svartsengi, taking out the Grindavík main road for the third time on this specific location. As the viscous lava front reached its terminal length, signs of inflation within the lava body higher up in the system appeared. During the next two weeks the lava expanded from within via tube system and rose to an elevation 7 m higher than the barrier it lay against. Eventually the lava boundary gave in, and multiple small overflows squeezed over the barrier. Those were kept at bay with active plugging and steering by fill material and with the aid of water cooling. Luckily the eruption ceased soon after or by June 22nd. Even though the crater went quiet, the lava continued to move gradually over the next days, not coming fully to arrest until June 28th.

In July and August 2024 inflation has continued in the Svartsengi magma domain with expected eruption within the first three weeks of August. Compared to the Krafla eruptions in North Iceland from 1975–1984 it appears

that the Grindavík fires can possibly be reaching the final stages. That might be a wishful thinking, but as long as magma is accumulating more eruptions must be expected. How many and for how long remains uncertain. The infrastructures are still functioning and holding out and with each event the mitigating measures need to be adjusted accordingly.

CONCLUSIONS

The theme of this meeting is “next generation meeting” and the subthemes of sustainable foundation, digitalization, challenges for the future, construction in urban areas and handbooks or guidance. What can we draw from the past months of our operations on the Reykjanes peninsula that may have relevance to some of these subjects.

Digitalization. This project has been realized mostly without traditional drawings when transmitting the design to construction. The lava flow modelling is highly dependent on accurate topographical models. The topographical models have mostly been sourced by drone surveying both photogrammetric and by Lidar scanning. After finding a suitable barrier elevation and form of the barriers, the model is delivered to the contractor’s cloud service where the operators have instant access to new or updated models. This seamless transfer allows for fast adjustments and adaptability that is badly needed in a constantly changing environment. With the ever-changing topography with each eruption the topographical models need to be updated fast and with great accuracy, mainly by drone surveying. During inflation periods of the viscous slow-moving lavas in the March and June eruptions, the drone surveying has also been vital to recognize the slow movements in the lava fields.

Volcano monitoring, eruption prediction and management of response is obviously very reliant on automation, interpretation of vast data quantities, and remote surveillance. However, there is the danger of remoteness associated with this approach as monitoring operators and command centers are at great distance often lacking communication with the ground, that is both ground personnel and the physical ground conditions. As personnel on the ground may be noticing important signs that are not incorporated in the assessments made at distance. Thereby we are at the risk of missing the trust and understanding between parties that is a crucial part of effective teamwork. Increased compartmentalization in the science community and within the response system may also contribute to misunderstanding as different entities are putting their narrow understanding forward that might shadow a more generalized view of the situation at hand.

Challenges for the future and construction in urban areas. In Iceland we already see present and future challenges related to climate change in both rural and some urban areas. with increased frequency and size of eruptions,

landslides, and glacier outburst floods, as well as sea-level changes due to retreating glaciers on local and global scales. In addition, we have now a very definitive situation of a new volcanic episode that has started on the Reykjanes peninsula where we have the highest population density. We have large urban areas placed on lava fields that are relatively young. If we were to compare frequency of lavas flows to landslide and avalanches, we would most likely limit or not allow inhabitation on those areas. The great question is how do we move on? Do we need to restrict further urban development within such areas, or are we going to accept the situation as is and rely exclusively on monitoring and mitigation measures as have been carried out for Grindavík and Svartsengi in the past months? It all come down to the value of acceptable risk which is at presently being evaluated. Central heating for the region is obviously too reliant on one source, alternative sources must be incorporated, either by low temperature harnessing or strong connection to the Reykjavík area.

Handbook and guidance. Familiarize yourself with the terrain! Engineering is in many ways based on a trial and error, to be molded and adapted into practical theories and methods. There are no applicable handbooks on how direct lava flows or to interact with running lava or how to lay roads or other utilities on a fresh lava. In a way we must gain that experience in the field through the sole of our boots and have the courage to pioneer something that would look absolutely ridiculous in the eyes of the general public or safety managers in industrial complexes used to a very controlled work environment. At the same time, we need to study previous historical undertakings and experiences achieved, constantly learning and adapting. In the end, fresh lava is just a material that we need to learn to work with and around.

We also must maintain a strong focus on the task ahead and not loose sights in all the digital and physical noise surrounding us. There is a good chance that Grindavík and Svartsengi can be protected. If we were to compensate for all possible outcomes that have been suggested, the situation would be near unrealistic. We can only cover the most likely scenarios, but those must be fully executed, any half-hearted attempts are not likely to secure a positive outcome.

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MODELLING THE PERMEABILITY OF A JET-GROUTED CUT-OFF WALL

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KEYWORDS

Jet grouting, cut-off wall, groundwater control, permeability, shaft

ABSTRACT

Jet grouting is a practical method to construct a sealed cut-off wall beneath a retaining wall when the excavation needs to be carried out in dry conditions. The method is frequently used in Scandinavia to seal the area between a steel sheet pile wall and the bedrock. The retaining wall subsequently has a hydraulic interface between the soil and the sheet pile, the soil and the jet grouted cut-off wall, and with the bedrock, respectively. However, the efficiency of the cut-off wall is often not possible to assess from hydraulic tests, and empirical methods are used in practical design. In the current paper, the permeability of the cut of wall is modelled from the installation and soil parameters, and the risk of encountering larger cavities in the cut-off wall is demonstrated.

INTRODUCTION

Development of building and infrastructure in urban environments frequently entails large excavation carried out beneath the groundwater table. Sealing the excavation from the surrounding groundwater is frequently one of the main challenges in geotechnical design. The jet grouting method was initially developed in Japan and has subsequently found worldwide use in practice, [1]. Jet grouted members of different shapes and properties can be developed at specified depths, giving the method great flexibility, [2]. In Scandinavia, excavations are frequent executed with sheet pile walls which are driven to the bedrock and supported with dowels. Geological conditions for this excavation method are common especially in Eastern Sweden. A schematic outline of the retaining wall and the surrounding soil is shown in Figure 1. To prevent changes to the groundwater levels in the area surrounding the

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excavation, a hydraulic seal is frequently added by jet grouting between the sheet pile wall and the bedrock, also shown in Figure 1. After the jet-grouted wall is constructed, the resistance to groundwater flow into the excavation, Q_{Total} , consists of the different seals: the sheet pile wall, the jet grouted wall, and the flow through the bedrock, as shown in Figure 2.

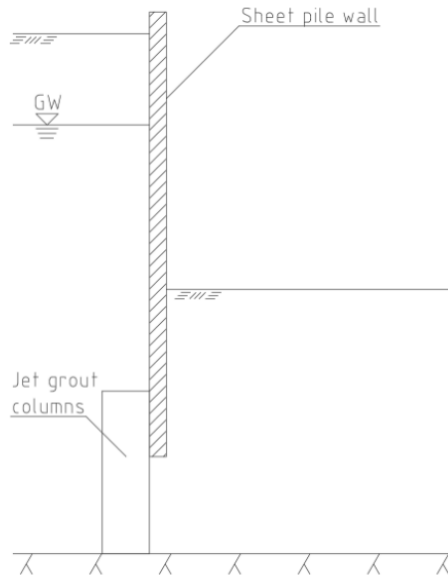


Figure 1. The typical design for jet-grouted columns for a hydraulic seal between the toe of the sheet pile wall and the bedrock

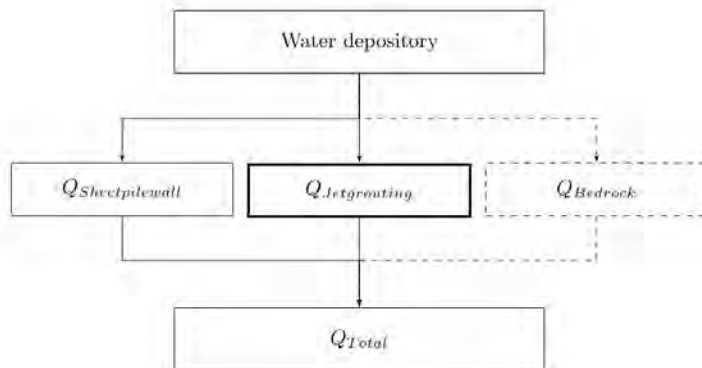


Figure 2. The parallel system with resistance for the flow of water, consisting flow through the sheet pile wall, the jet-grouted wall and the bedrock.

For the design of the jet grouted structure, the main parameter of interest for the hydraulic seal or cut-off is column diameter, as well as the geometric precision of the jet-grouted columns, [3]. The interaction between the grout

and the soil during jetting has been examined in detail in [4], outlining the mechanisms by which the soil properties influence the diameter of the jet grouted structure. Methods for the prediction of the jet grouted column diameter have been developed in [5], and a comparison between different prediction methods has been demonstrated in [6].

However, the prediction of the diameter is still associated with large uncertainties. Hence a statistical approach to assess how the performance of the geotechnical structure depends on the properties of the structure. In [7], the Monte Carlo method is used to assess the structural bearing capacity of a jet grouted structure. A similar method methodology is demonstrated in the current article to assess the remaining gaps in the jet grouted seal, taking account of the variability of the jet grouted columns.

METHODOLOGY

The geometric representation of the jet grouted columns in the Monte-Carlo method consists of a series of discs which extended along the diameter of the jet-grouted column. Since the columns is created by a rotational motion with the jet monitor, this representation can be considered to capture some of the main mechanism of the jet grouting process: namely the dependence of the local soil layer and the rotation and elevation speed of the monitor, [4]. The basic geometrical properties of the discs are shown in Figure 3. The statistical formulation consists of the description of the diameter of the discs with the normal distribution, but a correlation between the diameter of each depth is included to represent the influence of the specific soil layer.

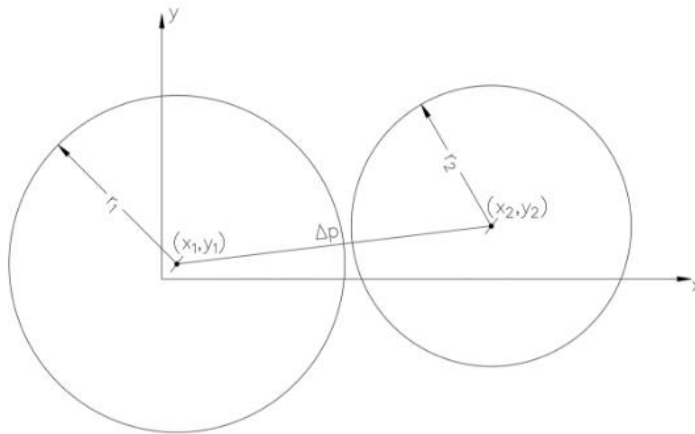


Figure 3. The geometry of the jet grouted columns in the Monte-Carlo simulation.

The following parameters are included in the statistical model:

- The number of columns

- The center-distance between the columns
- The depth to the top of the columns and the depth to the bottom of the columns
- The delta height of the discs
- the statistical distributions for the deviation in position on the ground surface
- The azimuth for the position deviation, the deviation in vertical inclination and its azimuth

To simulate the correlation between the discs at each layer, a matrix containing the discs diameters is generated. An additional correlation parameter is needed together with the mean value, and the standard deviation for the diameter of the discs. A correlation matrix, P , is constructed with the size $n \times n$, the diagonal consisting of ones and the rest ρ . The indices implies that the diameter in the same column is perfectly correlated with itself, and the ρ 's determine the correlation factor between columns m and n which in this case indicates the row and column in the matrix. The uniform distributions are then transformed to normal distributions. The diameter matrix now consists of columns with normally distributed series with mean-value and standard deviation that are correlated with each other with the correlation factor ρ .

Figure 5 shows the diameter of the discs in a scatter diagram, and Figure 5 shows the graphical representation of the discs for $\rho = 0,85$.

A simulation can subsequently be carried out with a specified set of parameters, in which the wall is simulated graphically, and the area from hollow holes emanating from the jet grouted walls is calculated. The model is implemented in the Monte-Carlo model and the resulting number of holes and the hole area are calculated, resulting in a distribution for statistical inference.

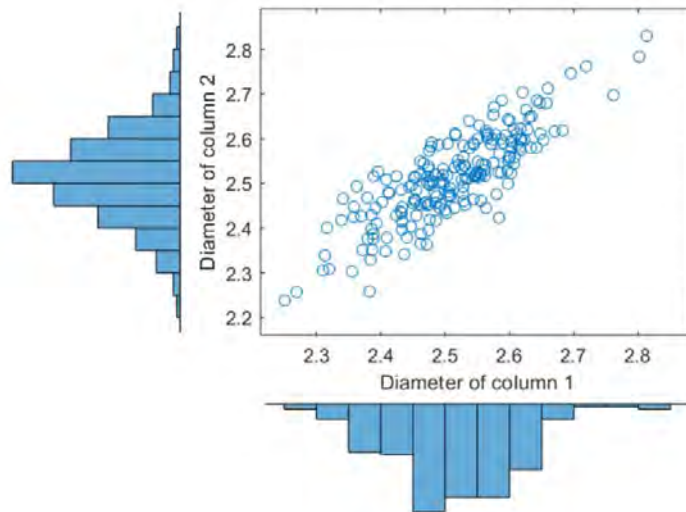


Figure 4. The correlation between the diameter of each disc at each layer for two columns with a correlation coefficient of 0,85.

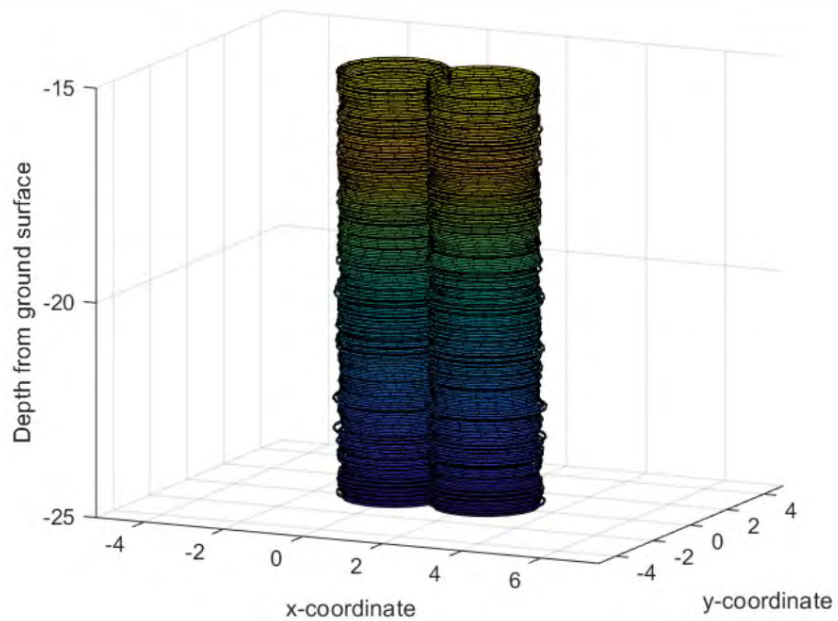


Figure 5. The graphic representation of the jet grouted columns as discs for two columns with a correlation coefficient of 0,85.

RESULTS

The described methodology for the Monte Carlo simulation of a jet-grouted cut-off wall allows a statistical analysis of the main parameters included in the design. The risk of groundwater flow through the jet-grouted cut-off can subsequently be estimated from Darcy's law with an estimation of the hydraulic conductivity of the surrounding soil.

A simulation was carried out for a typical jet-grouted cut-off, in which the total length was 42 m, with 2 m long jet-grouted columns. The column diameter, the center location of the column, and the inclination of the column were included in the Monte Carlo simulation.

Figure 6 displays the probability density function from the Monte-Carlo simulation. The normalized hole area, defined as the hole area divided by the total area of the wall, is shown on the abscissa on the graph. For the particular simulation, in which the column diameter was normally distributed with $N(1, 0.12)$, the probability of at least one hole is 98 %, and it is quite likely that the total hole area is between 0,05 to 0,1 m².

If the hydraulic conductivity is known, the probability of a groundwater flow into the excavation can subsequently be calculated.

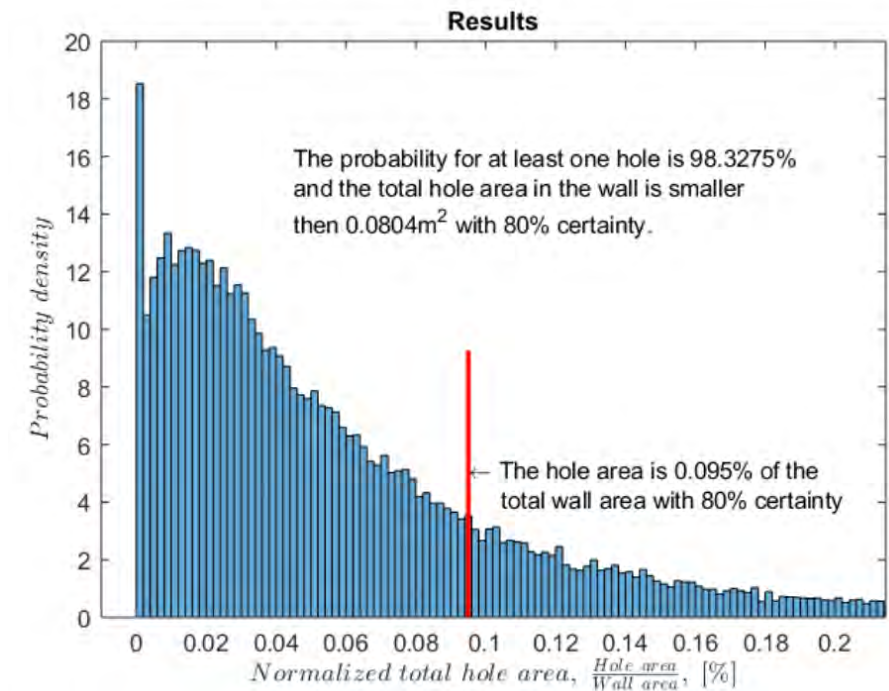


Figure 6. The probability density function for the normalized total hole area from the Monte-Carlo simulation.

CONCLUSIONS

A numerical procedure to carry out a Monte-Carlo simulation on a jet-grouted cut-off wall has been developed and demonstrated. The method allows the simulation of the hydraulic performance of the cut-off wall, and the influence of the main parameters in the design can be estimated, especially the influence of the column diameter.

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MONITORING REQUIREMENTS FOR LARGE SCALE EXCAVATIONS IN GOTHENBURG

Matthias Ziegert¹, Anders Bergström¹

KEYWORDS

Accuracy, Deep Excavation, Jet Grouting, Monitoring, Railway, Retaining Walls, Soil Movements, Surveying

ABSTRACT

The excavation and foundation work of Västlänken lot E05 Korsvägen is performed in conditions characterized by varying geology, neighboring buildings and public infrastructure along the alignment. The deep excavation pits do not only need to meet structural requirements but often also require watertightness. Several different special foundation methods are used to address the respective technical and economic constraints, e.g. drilled and driven steel piles, secant bored piles, diaphragm walls and sheet piling. In general, the very soft clay within the excavations needs to be stabilized to enable foundation works and the subsequent excavation of the naturally highly thixotropic soil.

To prevent critical settlements and movements in adjacent buildings and infrastructure that remain in operation throughout the construction period, strut systems are used for bracing the excavations. Wall deflections and settlements are monitored in an extensive control program using levelling and surveying stations. In addition, multiple inclinometers, distance meters and extensometers are monitored. For load monitoring in struts, different sets of strain gauges are used. With a system of wells, the ground water level is monitored, evaluated, and controlled by infiltration. Monitoring results are presented together with a discussion on accuracy and relevance in requirements, in relation to expectations of different actors, based on experiences from the project.

INTRODUCTION

For the development of the public transport infrastructure of West Sweden, the Swedish Transport Authority Trafikverket builds an 8 km long double lined railway track in the city of Gothenburg. The southernmost 3.2 km of the alignment are executed in the lot “E05 Korsvägen” by the Joint Venture

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“WLC – West Link Contractors” consisting of the Swedish contractor NCC and the German contractor Wayss & Freytag.

As part of the lot “E05 Korsvägen” the excavation pit of Almedal is located within the vicinity of an existing live railway on one side and the highway E6 on the other side connecting Gothenburg with Malmö/Copenhagen (Figure 21). Due to the sensitivity of the existing infrastructure a robust retaining structure along with extensive ground movement monitoring is required. Besides a brief display of the given soil conditions and chosen retaining structures, predicted deformations and ground movements are to be discussed in comparison with the monitored movements and stresses. Along with this comparison, challenges connected to the operation of monitoring systems within the construction field and the interpretation of the retrieved monitoring data are examined.



Figure 21: Live railway next to the strut supported deep excavation for the Västlänken-tunnel.

DESIGN BASE: HYDROGEOLOGY AND BOUNDARY CONDITIONS

The soil stratigraphy in the Almedal tunnel area mainly consists of soft clays and an underlying frictional material laying on bedrock. The clay is mostly sensitive, or even quick, to varying depths between 7 m to 20 m. The frictional material consists of grain sizes from fine sands up to coarse

boulders, located in a layer of up to 5 m thickness on top of the bedrock (Figure 22).

The hydrology is mainly characterized by two aquifers, one close underneath ground level and the other one confined underneath the clay layer. Due to the sensitivity of the neighboring infrastructure and buildings and the immediate impact of ground water level changes related to the foundation and excavation works, strict tolerances are to be maintained.

The design is based on finite element modelling in Plaxis 2D. The untreated clay on the active side is modelled with the NGI-ADP material model. The input for the undrained shear strength s_u^{DSS} is 16 kPa and the stiffness $E \sim 10$ MPa, both increasing by depth. The clay stabilized with lime cement columns (LCC) on passive side, and the frictional soils in fill and bottom layers, are modelled as Mohr-Coulomb material. The input for the LCC's undrained shear strength c_u is 100 kPa and the stiffness $E \sim 40$ MPa, both constant by depth and time [1].

The connection of the sheet pile to the bedrock through rock dowels is modelled as a fixed point at top of rock, with no moment restraint [2].

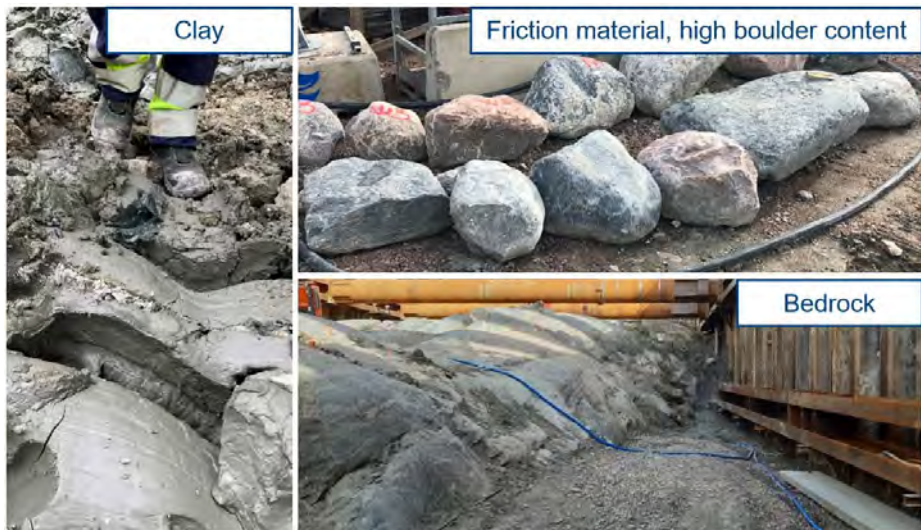


Figure 22: Illustration of the soil stratigraphy in Almedal

EXECUTION OF GROUND AND FOUNDATION WORKS

Prior to the foundation and excavation works and related utility relocation in Almedal, three railway tracks are to be demolished and two of them are rebuilt next to the planned work site.

After the final excavation level is reached, the concrete tunnel for the later railway is built in the bottom-up method. In between the different casting sections of the tunnel elements, struts and waling beams are removed in stages. The lowest strutting layer is removed after the tunnel bottom slab is installed, middle and upper most strutting layers are removed in connection with the installation of the tunnel walls and roof slab, while the gap between the tunnel walls and the sheet pile walls is backfilled with heavy concrete to counteract uplift and to enable load transfer from outside earth and water pressure into the tunnel's bottom slab.

MONITORING

Monitoring of buildings and infrastructure

The working site area has been rigidly monitored in purpose to verify the response in the installed supporting structures during construction as well as to monitor surrounding ground and structures, e.g buildings, highway, railway and bridges, to prevent exposure to harmful or unexpected deformations.

The surrounding structures and ground surface are monitored by using total stations, digital levels, monitoring prisms, level markers etc. Railway curvature and relative distance between the tracks is verified from daily performed wheeled “crab”-monitoring results. In addition, satellite monitoring is also available to discover ground level movements in random points and general pattern of settlements and heave.

Total stations, Leica Nova TM60, are used for monitoring of the prisms. The monitoring accuracy of the total stations is stated by the manufacturer to be 0,6 mm +0.0001 %. This applies for ideal conditions, however at the working site it is clearly seen that the practical accuracy is in the range of ± 2 -3 mm at the best. Most monitoring noise is evaluated to be caused from uncertainty in the reference monitoring net and from refraction caused by heat flow e.g. around vehicles or simply from sun or rain. As most of this monitoring is performed around-the-clock, trends and systematic behavior can be concluded, e.g. thermal response on daily or yearly basis. Monitoring during night hours is found to have less monitoring noise than during daytime, which is likely related to the reduced impact of the sun and construction activities during daytime. The total stations deliver more accurate monitoring results in length than transversally, which is caused by the geometrical difficulty of projecting angle measurements over distance. A variety of different prisms is used, depending mainly on flexibility needs and risk of damaging from production. For example considering the requirement of replacing prisms due to unforeseen impacts from the activities with heavy machinery, more economical prisms with less accuracy are used on the retaining structure.

Digital levels, e.g. Leica LS15, are used for levelling of markers, sheet pile top, railway tracks and other objects in the vicinity of the pit. The product

data sheet of the digital levels state monitoring results with the accuracy of parts of a millimeter. Measurements within the working site confirm this accuracy with a high reliability.

Due to the complexity of predicting ground movements caused by the execution of foundation works, also in the project in Almedal, only deformation limits related to the excavation process are considered in the design. Movements caused by e.g. the forces, pressures and vibrations from sheet piling or jet grouting are to be handled during the execution phase on site. The most demanding limits are in this case connected to the adjacent railway tracks, allowing only deformations of up to 12 mm over 6 m length equaling (1/500). The maximum limit values for vertical ground movements in the vicinity of the excavation pit retrieved from the design calculations range between 10 – 20 mm. As preventive measure the foundation works along the railway are performed during night times with certain hours of line closure. Due to the very sensitive clay underneath the railway tracks ground works are challenging to execute safely, especially jet grouting works, where pressures of more than 400 bars are induced into the soil. Depending on grain size and thickness of the frictional layer sudden heaves of up to 100 mm within minutes have to be controlled (see Figure 24). Besides the extensive surveying, various mitigation measures are in place, such as a tamping machine, to secure safe railway tracks after the line closure.



Figure 24: Risk zone of 50 m, 100 m and 150 m distance from the location of groundwork execution. During installation of jet grouting heave may appear fast and with significant impact.

Monitoring of retaining structure

Horizontal displacements in supporting structures are monitored by using prisms and inclinometers mounted on the sheet pile walls. Loading effects in the steel struts from different steps of loading and unloading during excavation and constructions works are monitored by using loading cells and strain gauges mounted on the struts.

Inclinometer sensors are MEMS Digital In-Place Inclinometer System from RST instruments. The data is handled by a digital logger LS-G6 in a wireless monitoring system from Worldsensing. The accuracy of each sensor is stated as 0.003 mm/m in the manufacturer's data sheet. The sensors are installed with internal distance of 1.0 m in depth. As the sensors are installed in a chain, there is likely an error propagation by depth of one or a couple of millimeters. However, in practice the main interest is in relative deflection and consequently impact of error propagation may be neglected.

Comparing the FEM-calculated deflections of the sheet pile walls with the actual field measurements of the inclinometers a general qualitative consensus is noted. However, evaluating deflections in a specific cross section of the excavation box in a three-layer braced section with one inclinometer on each side, differences in absolute values can be observed, see Figure 25. While on one side the predicted values are exceeded (IM S49), the corresponding opposite stays below the expected values (IM S277). This may be related to locally varying soil properties and taken assumptions during design phase.

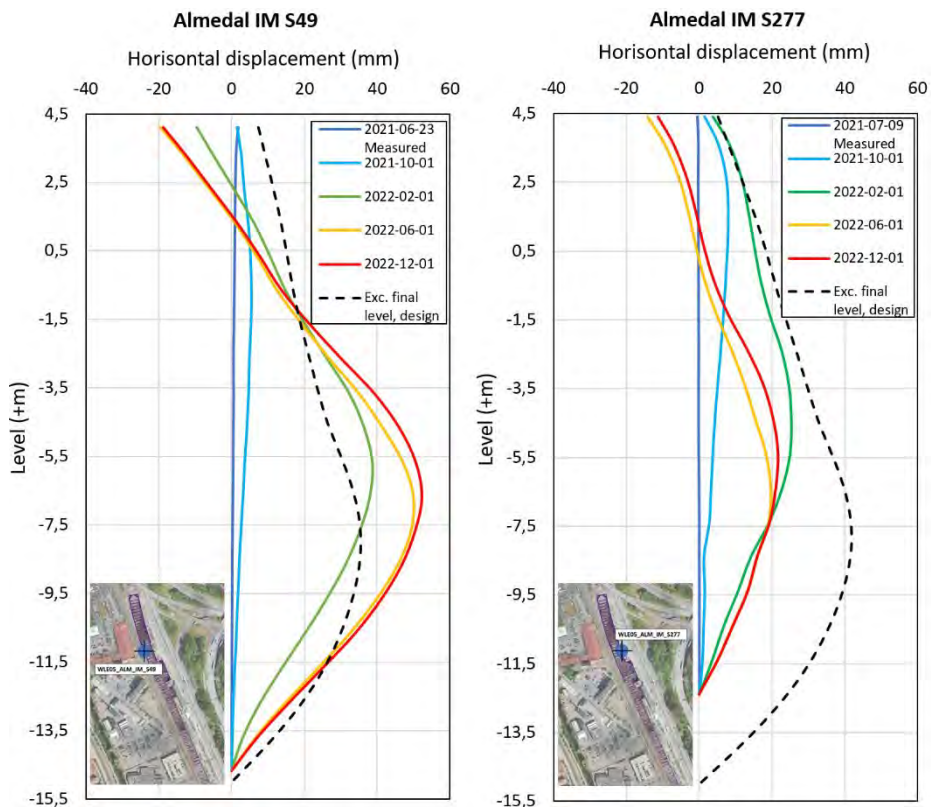


Figure 25: Inclinometer monitoring result from sensor installation to final excavation and installing of all supporting steel. Calculated deflection at full excavation is indicated in black dashed line.

Strut loading is monitored by using strain gauges fixed to the steel struts in four positions in each monitoring section. The strain gauges are VWSG-A sensors from RST instruments. The monitoring accuracy of the strain gauges is stated as 0.05 %, indicating $< \pm 25$ kN for a loading of 5000 kN. In a few positions at site, the sensor layout has been calibrated with acceptable accuracy when pre-stressing the strut, however this has not been verified in the struts presented in this text. In sections with struts provided from Ground Force, the loading is monitored in a loading cell in the end joint of the strut.

The large amount of data monitored provides the engineering opportunity to evaluate behavior both on a local and on a global level. For example, the strain gauges setup provides data in different positions around the steel strut tube; top, bottom and both sides. The mean value of these sensors is used in the general strut evaluation to fulfil the Control plan requirements. However, from the monitoring results of the individual sensors the stresses may vary significantly. This is evaluated to be from bending moment but also from the fact that the contact surface between the large size strut tubes and the waler beams will never be perfectly performed, resulting in eccentricity. Another observation from the extensive monitoring is the response on temperature. Depending on the stiffness and depth of the supporting sheet pile wall, the thermal expansion of the struts induces loads and deflection in the wall differently. Further, the loading response from temperature also seems to vary more extensive on short term basis (daily) than on long term (seasonal), see Figure 26.

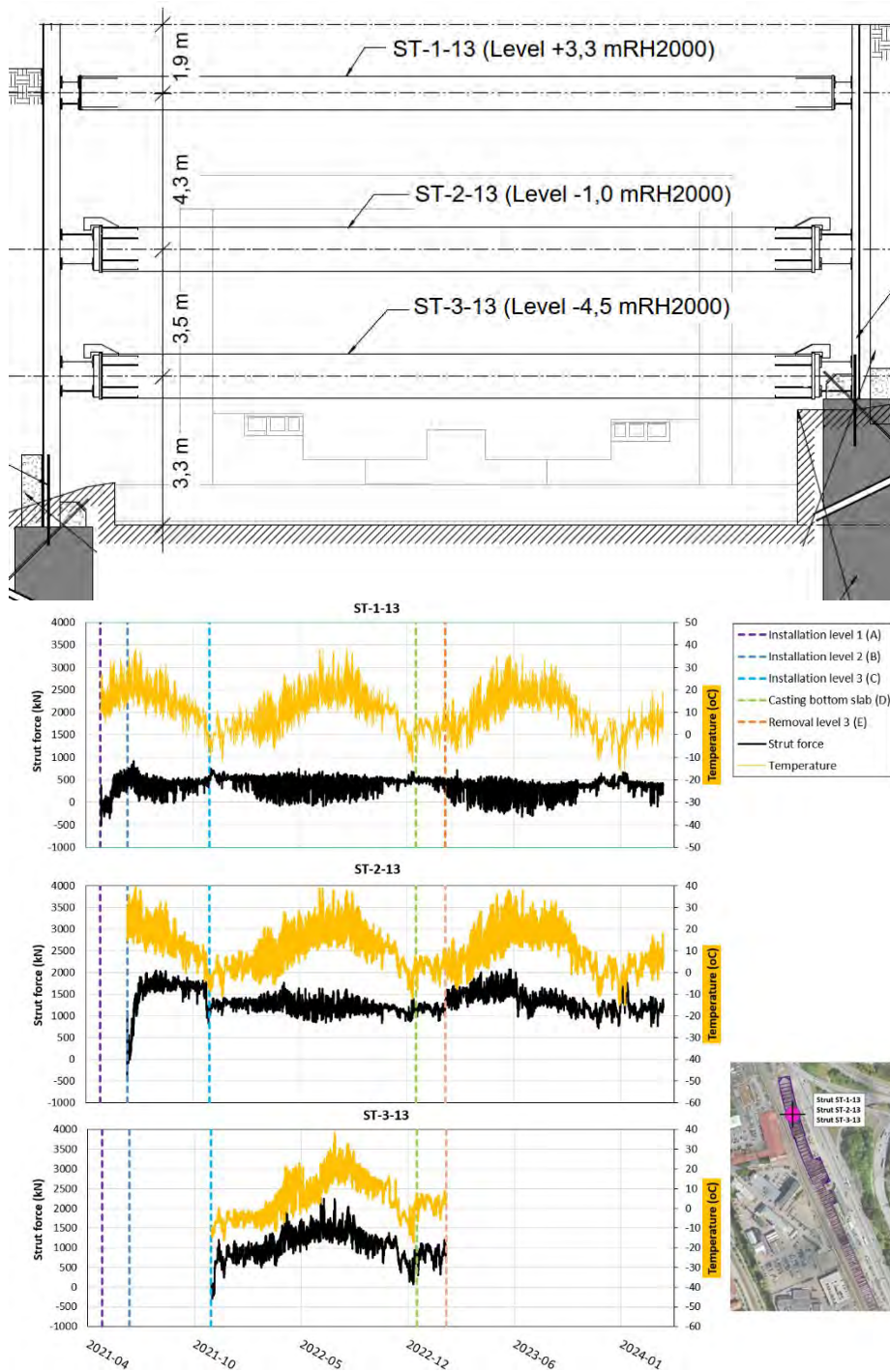


Figure 26: Section of the excavation, and plot on strut force and temperature vs time. The strut forces as per design is in the range of 4000 to 9000 kN.

LESSONS LEARNED

To monitor is to know! Properly used, the monitoring results can be used to verify design assumptions, optimize or adjust working sequences or quantify deformations. Monitoring brings information to evaluate and make knowledge-based decisions. Yet it is crucial to understand what to monitor, why, and not least how.

Noting that the accuracy of monitoring instrumentation is most often less in field conditions than in laboratory environment or in data sheets. Considering the accuracy limits of surveying and the impacts of the construction site, a sort of critical approach on produced monitoring data is recommended.

Digital levels are found to deliver monitoring results with very high accuracy, <1.0 mm. Automatic total stations with prisms delivers regularly continuous monitoring results, that are useful for evaluation of trends and general behavior, but with a lot of monitoring noise and less accurate results. To increase the accuracy the automatic monitoring requires daily hands-on service.

A sensor always delivers monitoring data if it is not broken. A full understanding of the way a sensor is installed and responding to its zero measurement is essential. For example, strain gauges are applied in groups of four around a steel strut; damage of one of the four sensors may lead to reporting of faulty mean values.

Sensors are most often sensitive to temperature and water, and in practice the sensors and connecting cables can be broken by construction machinery or simply being moved because they conflict with other works. Maintaining monitoring points on ground surface in working site environment is extremely difficult and shall be considered during the design phase.

An extensive monitoring system including a huge number of monitoring points also generates an enormous amount of data. To use the potential of this data, the project requires proper staffing and competence to evaluate data and distinguish between relevant movements and monitoring noise. Also, it is essential to understand potential consequences, while holding the mandate to act in short notice if required.

Monitoring can be used with the purpose to detect the development of settlements or rapid phenomena like the failure in a retaining wall. Inclinometers are accurate enough to detect both long term and short-term phenomena. Inclinometers are found to be less affected by monitoring noise than automatic total stations and therefore being preferred for the detection of rapid failures.

Installation of LCC and jet grouting often generates heave and soil movements, still this is regularly not modelled in FEM-based designs. This is

likely because the behavior is extremely complicated to predict as it depends on a range of parameters such as soil structure and installation methods. The movements of the installation works are often in the size of centimeters or even decimeters. Thorough risk assessments with effective mitigation measures are therefore vital to handle occurring movements and deformations during the execution phase.

Proper and relevant monitoring increases the understanding of the supporting structures and it gives an opportunity to evaluate behavior and detect danger and risks. To achieve these relevant aims, it is necessary that the measurements and the Control program have the trust of the organization. Communication between different actors is a key factor for success. There is occasionally a believe that monitoring, additional monitoring points or very specific and narrow limit levels is a straight and easy fix to make a design more robust or the construction works safer. However, experience proofs differently: Oversized monitoring creates unmanageable amount of data, likely connected to a lot of monitoring noise and the subsequent loss in confidence from the working site. Proper monitoring is difficult and comes with a high cost. The cost benefit must be analyzed in relation to other actions creating a robust working site. Non-specific, too generous, limit levels make the control monitoring irrelevant or even dangerous, but too narrow limits force focus to problem in monitoring accuracy rather than on verifying design or worse, not discovering the unexpected incidents that occur when there are humans working in a complex environment.

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NEW FINNISH NATIONAL GUIDELINE: SAMPLING FOR GEOTECHNICAL INVESTIGATION

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KEYWORDS

Guideline, Standards, Soil Sampling, Rock Sampling, Sampling category

ABSTRACT

With the new standard EN ISO 22475-1:2021 for soil, rock and groundwater sampling, new requirements for the sampling equipment and sample quality classes were defined. As the soil conditions and state-of-the-practice sampling equipment vary country by country, there was a clear need to create a new national guideline about geotechnical sampling in Finland. In a work funded by the Finnish Transport Infrastructure Agency, a new sampling guideline was created and subsequently published by the Finnish Geotechnical Society. The guideline presents practical guidance on the sampling process from choosing a suitable sampler to storing the samples in a laboratory. This paper presents the most significant observations made during the creation and the main national recommendations regarding the sampling equipment and procedure. With the new guideline, the quality of samples is expected to become higher in the future, thus ensuring reliable and accurate geotechnical investigations.

INTRODUCTION

Finnish Geotechnical Society SGY has published a series of national guidelines for various ground investigations, including soil sampling. With the renewal of European standard EN ISO 22475-1:2021 for geotechnical sampling [4], there was a clear need to revise the corresponding SGY guideline to align with the latest standard and state-of-the-practice sampling equipment used in Finland. In a work funded by the Finnish Transport Infrastructure Agency, new guidance was created and subsequently Ground investigation guideline III: Sampling for geotechnical investigation was published (in Finnish) by the SGY [3].

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This paper presents the most significant observations made during the creation of the SGY guideline and the main national recommendations regarding the geotechnical sampling equipment and procedure. Besides soil sampling, the guideline also describes sampling from rock and groundwater. Substantial work was done to define sampling categories (A/B/C/D/E) for specific equipment while considering varying soil conditions (soil type, depth of groundwater, etc.). All five sampling categories are considered in the guideline, even though the Finnish practice has considered only three sampling categories so far: undisturbed, disturbed, and imperfect sample. Sampling categories were discussed together with the ground investigation practitioners in a workshop, and comments were collected from a wider group of professionals, including geotechnical designers.

All five sampling categories are considered in the guideline, even though the Finnish practice has considered only three sampling categories so far: undisturbed, disturbed, and imperfect sample. Substantial work was hence done to define sampling categories for specific equipment while considering varying soil conditions (soil type, depth of groundwater, etc.). Sampling categories were discussed together with the ground investigation practitioners in a workshop, and comments were collected from a wider group of professionals, including geotechnical designers.

SOIL SAMPLING

According to the standard 22475-1 [4], “the aim of category A sampling is to obtain samples in which structure, texture, consistency and in-situ stresses are intact.” Hence, category A soil samples correspond to the “undisturbed” samples. Meanwhile, “category E sampling only obtains samples where all the initial soil properties have changed is wholly or partially separated due to the drilling process”.

The sampling categories A/B/C/D/E are connected to the quality classes in the standard’s Table H.1 [4]. The Finnish national guideline presents a revised version of this table: attempt was made to further clarify the classification and to modify it to better apply to Finnish geology and typical testing programmes (see Table 1). The classification itself was not modified, but some additional definitions were added; For example, testing for geochemical properties is not covered by the standard and was thus added to the national guideline.

Required quality class was assessed based on literature and interviews with environmental professionals. For instance, pH testing is relevant for sulphate bearing clays [1].

The guideline presents which sampling category (A/B/C/D/E) can be achieved with certain samplers in different soil conditions. The guideline also describes the most common equipment (which follow the standards’ requirements) with

photos or diagrams. Detailed guidance for category A sampling with piston samplers is provided as an appendix to the guideline.

Table 2 shows an example of the guidance provided for the selection of a suitable sampler (it should be noted that a shortened version of the table is shown in this paper for the sake of brevity.). The parentheses mean that such sampler is suitable in some specific conditions, referring to the properties listed for each soil type (e.g., relative density, depth of groundwater).

Based on the previous sampling experiences in Finland, silts without clay fraction are very challenging: the consensus among professionals was that thin-walled piston samplers (e.g., STII or Norwegian piston sampler Geonor), which usually corresponds to category A sampling, are suitable for category C sampling from silts. Category A or B sampling from silts requires block samplers or large-diameter special piston samplers described in Di Buò [2].

Table 1. Quality classes of soil samples and the sampling categories [3].

Soil properties that can be determined in the laboratory	Quality classes of soil samples in laboratory testing				
	1	2	3	4	5
Soil layer classification					
Soil type (based on visual inspection)	x	x	x	x	x
Layering order of soil types (approximate)	x	x	x	x	x
Boundaries of soil layers in rough accuracy	x	x	x	x	
Accurate soil layer boundaries (e.g., varved clay)	x	x			
Classification properties					
Grain size distribution	x	x	x	x	
Organic content, density of solid particles, Atterberg limits	x	x	x	x	
Proctor maximum dry unit weight	x	x	x	x	
Water content	x	x	x		
Geochemical testing ^{b)}	x	x	x		
Permeability properties and porosity					
Water permeability and capillarity ^{a)}	x	x			
Unit weight, bulk density, porosity, relative density	x	x			
Strength and deformation properties					
Undrained shear strength and sensitivity	x				
Shear strength (cohesion and friction angle) ^{a)}	x				
Stiffness and compressibility properties ^{a)}	x				
Sampling categories	A				
		B			
			C		
				D	
					E

a) Strength and deformation properties, water permeability and capillarity may be determined for coarse-grained soils and moraines using category C or D samples, which have been compacted into the desired dry unit weight with specific compaction energy.

b) Depending on the exact chemical properties, also soil quality class 4 (category D) may be sufficient. Note that the determination of pH should be performed as soon as possible after sampling (if needed, in-situ measurements should be performed to verify the results.)

Soil types	Conditions that affect the sampling	A	B	C	D
Dry crust	Stiffness Strength	Block sample ^{a)}	Block sample ^{a)}	Auger, Trial pit ^{b)} ,	Auger, Trial pit ^{b)} ,
Clay (and clayey silt)	Stiffness Strength Sensitivity Plasticity	Piston sampler (STI/STII/ Geonor), Block sample ^{a)}	Piston sampler (STI/STII/ Geonor), Block sample ^{a)}	Peat sampler, Open-tube sampler (double tube) Small piston sampler, Trial pit ^{b)} , Piston sampler with side intake, (Auger) (Open-tube sampler) (Flow-through sampler)	Peat sampler, Open-tube sampler, Auger, Flow-through sampler
Silt (without clay fraction)	Stiffness Strength Sensitivity Plasticity GW level	Block sample ^{a)}	Block sample ^{a)}	Piston sampler (STI/STII/ Geonor), Peat sampler, Open-tube sampler (double tube), Trial pit ^{b)} , (Auger) (Small piston sampler,) (Open-tube sampler) (Flow-through sampler)	Peat sampler, Open-tube sampler, Small piston sampler, Piston sampler with side intake, Auger, Flow-through sampler, Window sampler
Peat	Level of decomposition	(Block sample ^{a)})	(Block sample ^{a)})	Trial pit ^{b)} , (Peat sampler) (Auger)	Trial pit ^{b)} , Peat sampler (Auger)

Table 2. Guide for selecting a sampler for some soil conditions (modified after [3]).

Soil types	Conditions that affect the sampling	A	B	C	D
Sand	Grain size Relative density GW level	^{c)}	Block sample ^{a)}	Open-tube sampler (double tube), Peat sampler, Trial pit ^{b)} , (Auger)	Peat sampler, Open-tube sampler, Piston sampler with side intake, Small piston sampler, Trial pit ^{b)} , Window sampler, (Auger)
Gravel	Grain size Relative density GW level	^{c)}	^{c)}	(Trial pit ^{b)}) (Peat sampler)	Open-tube sampler (double tube), Test pit ^{b)} , (Peat sampler) (Open-tube sampler) (Window sampler)
Moraine	Grain size Relative density GW level	^{c)}	(Block sample ^{a)})	Trial pit ^{b)}	Open-tube sampler, Window sampler, Test pit ^{b)}
Fill	Grain size Relative density GW level	^{c)}	^{c)}	Trial pit ^{b)}	Window sampler

(Table 2. continues)

^{a)} Block sample refers to block sample taken from test pit, or a sample taken with block sampler or large-diameter piston sampler.

^{b)} Trial pit refers to bulk disturbed samples taken from trial pit.

^{c)} Category A and B sampling is rarely performed in coarse-grained soils and moraines.

According to Table 2, the peat sampler may be used in various soil conditions. It was observed that besides peat and clay, this sampler can also be used in

silts and sands. In Finland, a peat sampler with 60 mm diameter (developed by Ramboll Finland) has been used for sands and even some gravels. Meanwhile, the 40 mm version (see Figure 1) is used mainly for fine-grained soils.

For the most common samplers, the guideline presents a detailed description of the sampling category for different soil types and conditions (relative density and groundwater (GW) level). Table 3 presents such classification for two peat samplers: 40 mm and 60 mm models.



Figure 1 Peat Sampler with 40 mm diameter (also known as “Russian Corer”).

Table 3. Sampling categories for peat sampler in different conditions [3].

	Peat sampler 60 mm		Peat sampler 40 mm	
Soil types and conditions	Category	Notes	Category	Notes
Clay ^{a)}	C		C	
Silt ^{b)}	C		C	
Sand – over GW	C		C	
Sand – under GW	D (C)	C, if dense, fine to medium fine sand	D (C)	C, if dense, fine to medium fine sand
Gravel – loose, over GW	C	Only fine gravel	–	Not suitable
Gravel – dense, over GW	C	Only fine gravel	–	Not suitable
Gravel – under GW	D	Only fine gravel	–	Not suitable
Moraine – loose	–	Not suitable	–	Not suitable
Moraine – dense	–	Not suitable	–	Not suitable
Clayey gyttja, silty gyttja	C		C	
Peat	D (C)	C, if sapric peat	D (C)	C, if sapric peat

Mixed man-made fill	–	Not suitable	–	Not suitable

a) Fat clay, lean clay, organic clay and clayey silt, with clay-like consistency. b) Silt without clay fraction

ROCK SAMPLING

Regarding rock sampling, the definitions of quality classes of rock samples for laboratory testing were adopted in their standard form [4], and thus not repeated here. One clarification was added to the national guideline: it was noted that the rough determination of rock quality boundaries based on drilling mud is not possible in quality class 5.

Table 3. Examples of rock sampling methods and equipment [3].

Sampling method	Equipment	Sample diameter [mm]	Hole diameter [mm]	Category	Notes
Wireline, triple-tube	NQ3	45.1	75.7	A (B)	Especially for rock mass with faults. B, if water flushing causes sample disturbance.
Wireline, triple-tube	WL-66 triple-tube	46	67.1	A (B)	Especially for rock mass with faults. B, if water flushing causes sample disturbance.
Wireline, double-tube	NQ2	50.5	75.7	B (C)	Especially for rock mass with faults. C, if the structure of the sample is changed due to fracturing.
Double-tube core barrel	T2-46	32	46	B (C)	By default not recommended due to small diameter.
Double-tube core barrel	T2-56	42	56	B (C)	Commonly used to study the rock mass quality in projects with tunnels or drilled piles installed to bedrock. C, if the structure of the sample is changed due to fracturing.
Double-tube core barrel	T2-76	62	76	B (C)	May be used to study the properties of rock material (quality of aggregate materials for road structure and pavement layers). C, if the structure of the sample is changed due to fracturing.
Blasting/Hydraulic splitting	–	–	–	D	Boulder samplers, e.g. to define quality of aggregate materials.

Drilling mud / Cuttings by rotary open hole drilling	–	–	–	E	Sample retrieved with flushing medium (water/air)

Rock sampling categories were assigned for the most common equipment used in Finland, while considering the properties of the rock (see Table 3). Special attention was given to the typical Finnish conditions characterized by hard rock types such as granite. The listed equipment refers to the most common sampler models in Finland. The most common sampler type in Finland is the double-tube core barrel, in which the sample quality is affected by the flushing medium. Possible fault fillings may be removed by water flushing, which leads to the sampling category dropping from B to C. Sampling category A can be reached by using triple-tube samplers, which include a third inner split sample tube.

CONCLUSIONS

With the new standard EN ISO 22475-1 for soil, rock and groundwater sampling, new requirements for the sampling equipment and sample quality classes were defined. As the soil conditions and state-of-the-practice sampling equipment vary country by country, there was a clear need to create a new national guideline about geotechnical sampling in Finland. The new sampling guideline presents practical guidance on the sampling process from choosing a suitable sampler to storing the samples in a laboratory. With the new guideline, the quality of samples is expected to become higher in the future, thus ensuring reliable and accurate geotechnical investigations.

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NORDIC COOPERATION GIVES RESULTS

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KEYWORDS

Eurocode 7, NMGEC7, National Annex

ABSTRACT

The Nordic Mirror Group on Eurocode 7 was established in the year 2014, to influence the development of the second generation of Eurocode 7 on the European level and facilitate implementation within the Nordic countries. Our two-day meetings twice a year have been a well-attended forum for sharing national praxis and experience, creative discussions, finding the common base, and identifying the main items to influence. The goal has been to make sure that the second generation of Eurocode 7 is developed with consideration of Nordic geology, climate, and praxis.

The result? Ten years later, there is a long list of items that the Nordic team as a coordinated group has influenced, by assisting each other with arguments and input, for example, the content of FprEN 1997-2 Ground properties, the inclusion of a chapter on rock bolts and another chapter on groundwater control and an annexe on buckling of piles. The paper will give details on these items and others. In addition, the paper will give a first outline of the next challenge that the Nordic team has taken on – a common Nordic national annex for Eurocode 7.

EUROCODE 7 – A COMMON EUROPEAN TOOL FOR DESIGN

As geotechnical engineers, we are influenced by a broad spectrum of standards. However, the Eurocodes have a unique position with its mandate to serve as a reference design code, implying that all member states must accept the design according to EN Eurocode and withdraw any conflicting national standards. The Member States of the EU and EFTA have decided that these technical rules (Eurocodes) should serve as reference documents; 1) to prove compliance with the fundamental requirements (serviceability, safety, fire,

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robustness), 2) for contract specification, and 3) as a framework for creating harmonised technical specifications for building products.

2nd Generation of Eurocode

The first generation of Eurocode was published between 2002 and 2007, after a long process of developing the initial draft, pre-standards, and finally compiling the first generation. As a reference design code, Eurocode was expected to facilitate trade and provide a more consistent level of safety within the union.

Eurocode influences over 500 000 engineers' work and a large amount of civil engineering and building projects throughout Europe. It is a tool that needs to be up to date with the technical development (new technology and materials), and adapt to the fact that the demands from society have shifted over the years from the focus on safety to giving a larger consideration to adaptation to climate change and sustainability. In 2015, the first projects team within CEN TC 250 were established with the objective of 1) improving ease-of-use, 2) increasing harmonization, 3) covering aspects of the assessment, re-use, and retrofitting of existing structures, and 4) strengthening the requirements for robustness. For Eurocode 7, the scope of the standard was extended to include the application of rock. In addition, new aspects and geotechnical structures, such as ground improvement, soil nails and dynamic action were added to the scope.

NORDIC MIRROR GROUP ON EUROCODE 7

Is geotechnical design without borders possible? The Nordic countries are individually small countries with regards to inhabitants (4 per cent of Europe's population); however, together, our countries cover more than 30 per cent of the surface area of Europe. A surface area with specific geotechnical and climate conditions that the common European reference design code should account for. The question raised in the year 2013 was if we in the five Nordic countries had enough commonalities in our geotechnical design to work as a team on a European level. The aim was to recognize our differences and similarities, working to assist each other with arguments to ensure that the 2nd generation of Eurocode will be feasible to implement for use in Nordic climate and geotechnical conditions.

The first meeting was in August 2013 in Oslo, and since then, the team has met biannually, rotating the location between Norway, Sweden, Denmark, Finland, and occasionally Iceland. No meeting is alike. However, each meeting includes learning from each other's experiences, arguing for our views, laughing, better understanding our differences, and assisting each other to cover all the questions that the 2nd generation of Eurocode raises.

Geotechnical design in the Nordic countries—is it without borders? No, we still have our differences in geotechnical praxis and will always have. But the borders are easier to cross since we now at least understand each other's views.



Figure 1 Part of the NMGEC7-team in 2016 and 2024

NMGEC7 ACHIEVEMENT

The aim was to ensure that the 2nd generation of Eurocode would be feasible to implement and use in Nordic climate and geotechnical conditions. To accomplish this the team divided the work between them and ensured that as long as one from the team was present at the numerous meetings developing the Eurocode 7, the views that NMGEC7 had agreed on were expressed. During the work, we also developed a common support for each other in the debate, probably because all our discussions gave us a better understanding of each other's issues.

It is not possible to give a complete list of the items where our common work influenced the final draft, but let's look at some examples.

EN 1997-2 Ground properties

One of the first items recognized in NMGEC7 discussions was that if EN 1997-2 should be useful as a standard, it needed reorganization and updated content. A proposal was prepared to make EN 1997-2 a standard to be used by the designer to determine ground properties, in contrast to the 1st generation, which had more similarities with a guideline for performing ground investigation. A positive response was received at the European level, and the

final version now has a table of contents that focuses on the major groups of ground properties that are needed for design (strength, deformation, state, dynamic, thermal, and groundwater).

Ground investigation

From a Nordic point of view, extensive and high-quality ground investigation is necessary to design a complex geotechnical structure. However, for all projects, the engineer needs to have the flexibility to use engineering judgment to select the most appropriate type and amount of investigation. This was not in line with praxis in other parts of Europe, which asked for requirements on the number of required field and laboratory tests. Endless discussions resulted in that there is a national possibility to add the requirements, and the tables with values were transferred to an informative annexe.

Pile design

The pile design clause has had much debate since the praxis throughout Europe has significant differences. The Nordic tradition of pile testing is not obvious to the rest of Europe; however, in the end, the clause includes three different approaches and, in addition, one combination of two approaches, see Figure 2. The dynamic testing of piles is recognized as an alternative to static pile load tests, and the interpretation of test results corresponds with the current Nordic approach.

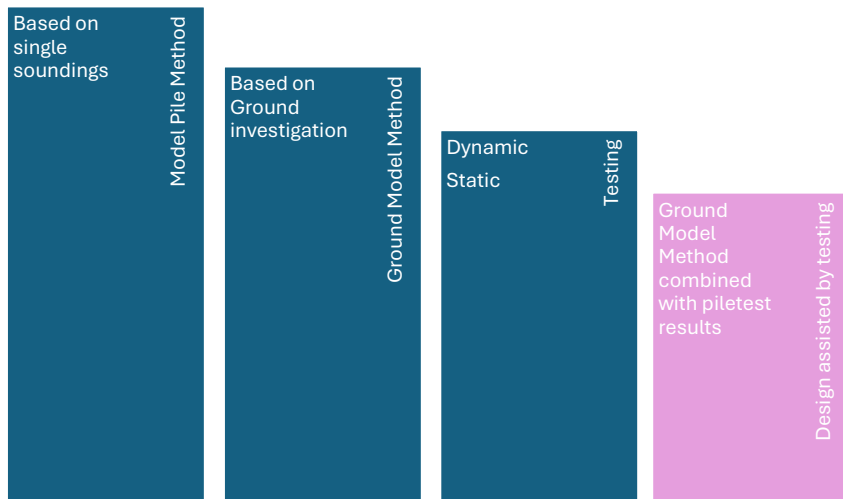


Figure 2 Pile design approaches according to FprEN 1997-3:2024

The structural design of piles requires that we consider the second-order theory determining the buckling of the pile. In EN 1997-3, a separate annex

has been included with recommendations on how to treat this phenomenon. This mostly has been developed by a group with members from the Nordic countries and Germany.

Rock engineering, including rock bolts and rock surface support

The mandate for developing EN 1997 stated that the standard should be equally applicable for rock and soil, and therefore the main word used in the standard is ground. 1st generation mainly considers soil; hence, it was a challenge to rewrite the code to make it generally applicable to rock with a specific focus on foundations on rock and rock slopes. The design of underground openings is still an item that will be developed in the coming years. The Nordic team contributed both to the general inclusion and the specific items, such as the inclusion of rock bolts and rock surface support.

Ground Model and Observational Method

Other items where the Nordic Team has made a substantial contribution are the development of the new sub-clause on the Ground Model (EN 1997-2) and the updated wording on the Observational Method (EN 1997-1).

CONTINUATION

The formal vote draft of EN 1997-1, EN 1997-2, and EN 1997-3 have been submitted, and NMGEC7 has contributed to the process. The next step is to implement the design code in our Nordic countries, and for that, we decided to continue our cooperation. We aim to share our views, analysis, and examples in an attempt to facilitate not only the implementation of Eurocode but also our cooperation on a day-to-day basis in civil engineering and building projects throughout the Nordic countries. From the work with NMGEC7, we learned that it is beneficial to work together, even though each country, in the end, needs to prepare its regulations and guidelines themselves, mistakes are avoided, and knowledge and new perspectives are added through the cooperation. At the latest NMGEC7 meeting in May 2024, it was concluded that the Nordic countries would have similar choices for the majority of the NDPs in the national annexe to EN 1997-1. That is a huge step forward to a geotechnical design without borders in the Nordic countries.

ACKNOWLEDGEMENT

The authors would like to thank their colleagues in NMGEC7 for their willingness to share and contribute to the common objectives. The discussion

with an open mind and initially different views are the key to our successful work!

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NUMERICAL ANALYSIS OF SOIL NAIL WALLS COMBINED WITH PRESTRESSED ANCHORS

Samuel Dietrich¹, André Arnold²

KEYWORDS

Soil nails, prestressed anchors, mass displacement, ultimate limit state, failure mechanisms, numerical modelling, combining practice and theory

ABSTRACT

Soil nail walls typically consist of a shotcrete wall and soil nails and are often used for excavations and measures to ensure safety of slopes. The wall system requires a degree of deformation in the subsoil in order to activate nail forces. Soil nail walls are often combined with prestressed anchors to limit soil deformation and settlements to adjacent structures. Such combined systems might not fulfil the design requirements when the required forces in the soil nails cannot be mobilised due to limited soil-deformation in the presence of prestressed anchors. Furthermore, the prestressed anchors might influence the shape and size of failure mechanisms compared to failure mechanisms of soil nail walls without the combination with prestressed anchors. This contribution outlines first results of a numerical simulation of combined wall-systems consisting of soil nails and prestressed anchors using the 2D FE-code “Optum G2”. Full-scale tests on soil nail walls are used to calibrate the numerical model. Subsequently, the results of the full-scale tests and the numerical model are compared and discussed. Furthermore, a row of soil nails is replaced by prestressed anchors in the numerical model in order to investigate differences in load-bearing behaviour between combined and ordinary systems.

INTRODUCTION

Soil nail walls have been utilised for many years to enable excavations and as measures to enhance the safety of slopes. Deformations of the supported structure as well as the adjacent soil are needed to activate soil nail forces. Such deformations in the soil, however, might cause damage to surrounding structures. Therefore, soil nail walls are often supplemented with prestressed anchors. The use of prestressed anchors influences the load bearing behaviour of soil nail walls. The basic analytical model for the dimensioning of soil nail

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walls (Figure 1 right) assumes that the deformations of the whole system are sufficient to fully mobilise the required nail forces in the soil. Hence, the approach does not take any strain dependent development of nail forces due to the presence of prestressed anchors into account. The interaction of soil nails and prestressed anchors has been investigated at ultimate limit state in the framework of a bachelor's thesis at the Lucerne University of Applied Sciences and Arts or *Hochschule Luzern* (HSLU) [3]. The investigation was based on finite element modelling and was calibrated using a practical example provided by Gässler [1]. Gässler investigated the load bearing behaviour of soil nail walls in large-scale tests (Figure 1 left). The results and conclusions of the investigation still represents the state of the art in the dimensioning of soil nail walls.

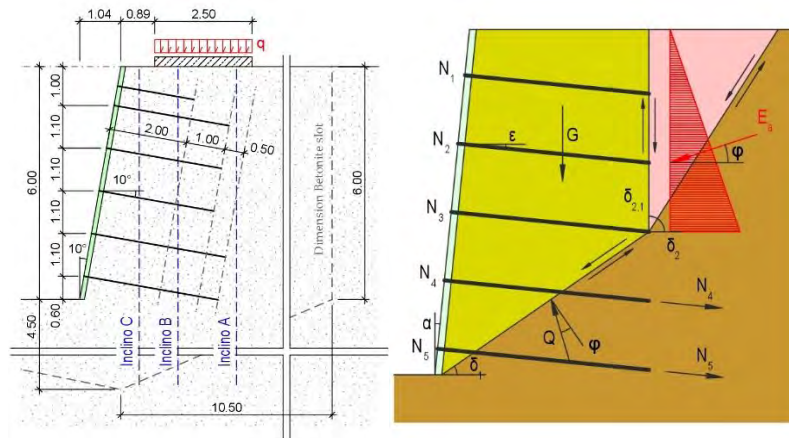


Figure 1 left: Large-scale test by Gässler (1987) [1]; right: Analytical model for the dimensioning of soil nail walls (newly drawn after Rüegger [5])

NUMERICAL MODELLING IN OPTUM G2

The finite element program OPTUM G2 (version 2021 2.2.20, build date 2022.08.19) with the advanced constitutive soil model HMC (Hardening Mohr-Coulomb; Krabbenhoft, 2019 [4]) was used. Gässler [1] analysed the soil present in the large-scale tests, however, the soil stiffness was not considered. Inverse calculations with different soil stiffness parameters were conducted for densely packed sand-soil. The boundary conditions and the parameter set of the numerical model are provided in Figure 2 and Table 1. The parameter-set of “hostun sand” [2] provided the most plausible results in terms of the inverse calculations and comparison to the test results of Gässler [1] (Table 1). The goal of the numerical analysis was to reproduce the large-scale tests given in [1], where the surcharge load q was stepwise increased until failure. In accordance with this procedure, an elastoplastic analysis was carried out in OPTUM G2 with step-by-step excavation and installation of the

soil nail layers. Subsequently, the load q at ultimate limit state was determined in an elastoplastic multiplier analysis.

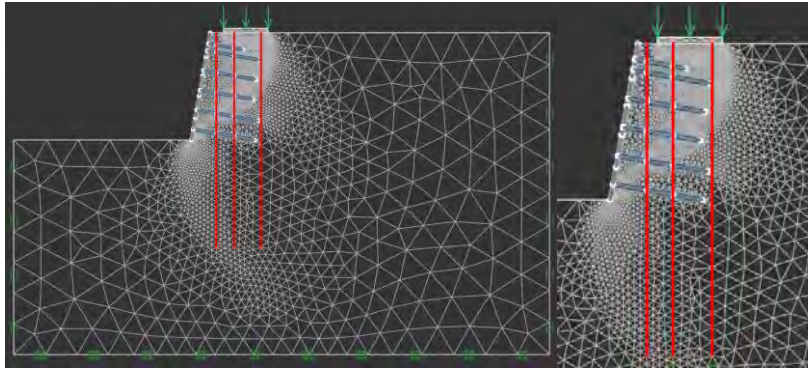


Figure 2 left: Overview of the model with boundary conditions, geometry and mesh of the numerical model for the inverse calculation of the large-scale tests by Gässler [1] right: Model details of the soil nail wall

RESULTS OF THE NUMERICAL ANALYSIS

Comparison with the large-scale test

The measured failure mechanisms in the large-scale test B of Gässler [1] compares favourably to the numerical analysis in terms of the failure mechanism shape as well as in terms of bearing capacity. However, the comparison of the numerical results to other large-scale tests documented by Gässler [1] does not show a good correlation. The details are shown in Figure 3. The bearing capacity of the large-scale test B reported by Gässler [1] of $q = 150 \text{ kN/m}^2$ also compares favourably to the elastoplastic analysis with $q = 134 \text{ kN/m}^2$. However, the nail forces obtained differ significantly.

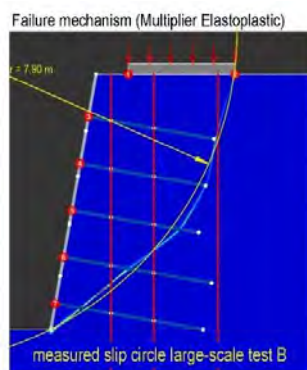


Figure 3 Comparison of failure mechanisms given by [1] to the numerical analysis

Table 1 Model explanations & input parameters

model explanations		«Nail row» parameters	
Width	30 m	D	6 cm
Height	18 m	E-Modulus	2E+05 MPa
Mesh	Adaptive net (6-n-Gauss-elem.)	Spacing	1.2 m
Boundaries	lower edge: non-displaceable	Axial strength	21.5 kN/m
	lateral edges: displaceable (vertical dir.) // non-displaceable (horizontal dir.)	Lateral strength	0 kN/m
Soil nails	Material model “nail row”	«Plate» parameters	
Shotcrete	Material model “plate”	EA	3.6E+06 kN/m
Prestressed anchor	Modelled with “Connector” and “Geogrid”	EI	4'320 kNm ² /m
«Geogrid» parameters		n _p	2'400 kN/m
EA	5.2E+04 kN/m	m _p	72 kNm/m
Yield force	496 kN/m	w	300 kg/m/m
«Connector» parameters			
E-Modulus	19.5E+05 MPa	Sectional area	4.0 cm ²
Yield strength	1'860 MPa	Spacing	1.50 m
Material parameters «hostun sand» soil-model: HMC (Krabbenhoft, 2019) [4]			
p _{ref}	100 kPa	φ'	42 °

m	0.55	ψ	13 °
$E_{50,ref}$	30 Mpa	c'	3 kPa
$E_{ur,ref}$	90 Mpa	γ_{dry}	15.6 kN/m ³
v_{ur}	0.2	K_0	0.331

Comparison of nail forces when using prestressed anchors

The experimental setup B utilised by Gässler [1] is adopted in the numerical model illustrated in Figure 3. In the model, however, the second row of soil nails was replaced by prestressed anchors. The prestressed anchors were defined with a free anchor length of 7 m (according to Swisscode SIA 267 [6]), an anchorage length of 7 m and a fixation load of 335 kN. The free anchor length was set as «Connector» and the anchorage length as «Geogrid» ([4]; Table 1). The fixation load of the prestressed anchor was defined on the basis of a similar executed construction project with both soil nails and prestressed anchors. The relatively high fixation load was necessary to prevent excessive deformation of the supporting structure.

The results of the comparison are illustrated in Figure 4. The results indicate that the predicted nail forces in the numerical model are significantly lower with prestressed anchors compared to the numerical analysis without prestressed anchors for small surcharge loads. However, the results also show that the soil nails tend to have a similar load-bearing behaviour at ultimate limit state compared to the simulation without prestressed anchors.

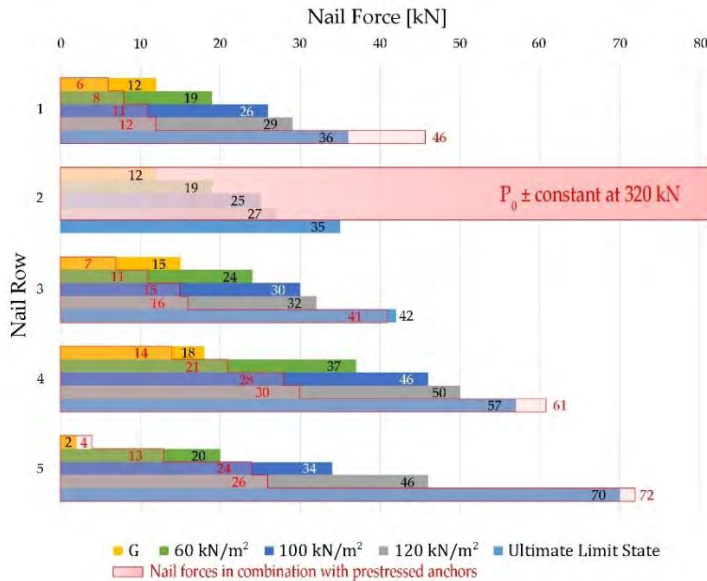


Figure 4 Mobilisation of nail forces at different levels of surcharge load q (Figure 1); G: situation without surcharge load q

The prestressed anchor row enables a surcharge load increase $q = 134 \text{ kN/m}^2$ to 725 kN/m^2 at ultimate limit state. However, the soil nail forces remain approximately constant or with only small deviations for different values of the fixation load.

Further investigations on an idealised soil nail wall

An idealised model was created to investigate the influence of different construction stages on the mobilisation of soil nail forces (Figure 5). This model was initially defined as a soil nail system without prestressed anchors. The lengths of the soil nails were defined such as to fulfil the ultimate limit state without taking into account the dimensioning according to the design standards. An elastoplastic numerical analysis, using a ϕ' -c-reduction, resulted in a safety factor of 1.03. The focus was placed on the mobilisation of nail forces in the different construction stages. Details are provided in Figure 5. Subsequently, the second soil nail row was substituted with prestressed anchors to compare the mobilisation of forces. The fixation load was again set with a relatively high value as the variation of the fixation load did not significantly influence the mobilisation of soil nail forces, as portrayed in Figure 4.

The results in Figure 6 again indicate that the soil nail forces tend to decrease with the presence of prestressed anchors. Comparison of the results given in Figure 6 to the system described by Gässler [1] (Figure 4) shows that the

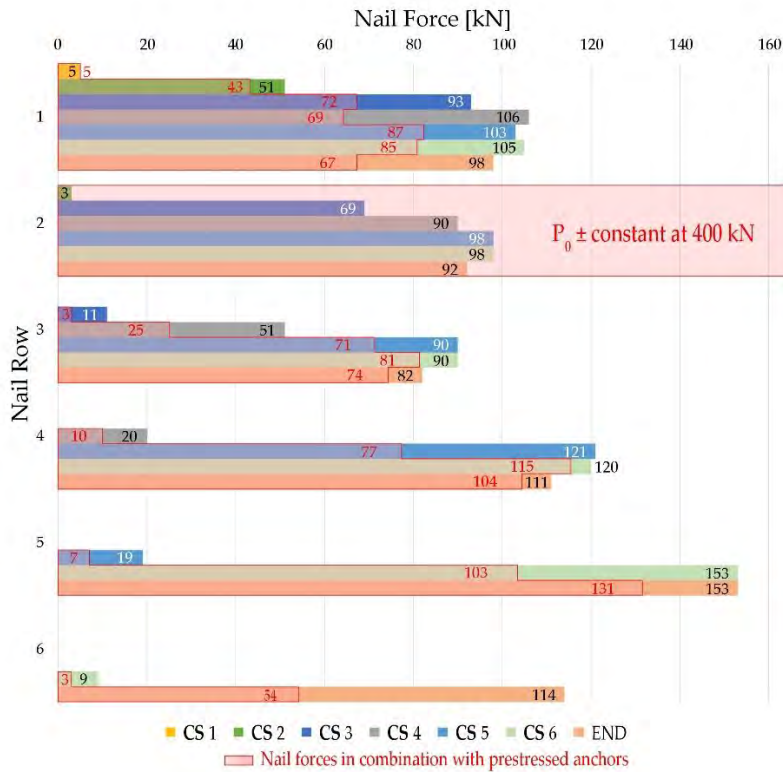


Figure 6 Soil nail forces of the idealised system for different construction stages

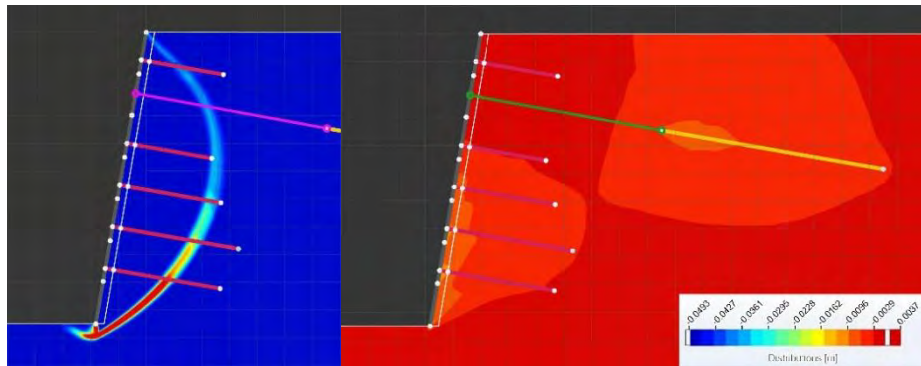


Figure 7 left: Possible failure mechanism (visualisation of shear dissipation) right: Horizontal deformations

CONCLUSIONS

The numerical investigations clearly show that the presence of prestressed anchors in a soil nail wall has a direct influence on the forces mobilised in the

soil nails. All numerical simulations show decreasing soil nail forces due to the presence of prestressed anchors. The overall stability of the supporting structure at ultimate limit state was not decreased in comparison to ordinary soil nail walls. However, the fixation loads of the prestressed anchors were relatively high in order to prevent deformations of the supporting structure.

The investigation of various construction stages predicts a critical failure mechanism in the form of a rigid body rotation below the prestressed anchor row. Therefore, it is important to also investigate failure mechanisms which are not formed over the overall height of the soil nail wall but develop below prestressed anchor rows – especially for prestressed anchor rows located near the top of the wall. Consequently, the position of prestressed anchor rows in soil nail walls can lead to incorrect dimensioning of the wall if failure mechanisms tend to form below the prestressed anchor rows. The well-known analytical model given by Gässler [1] should hence be supplemented with failure mechanisms which are formed below the prestressed anchors with continuing construction states.

The numerical results are to be verified in terms of physical modelling to compare the development of failure mechanisms and to gain more information on the real behaviour of soil nail walls in combination with prestressed anchors.

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NUMERICAL MODELLING OF BIOCEMENTATION UPSCALING EXPERIMENT: INSIGHTS FROM VARIED INJECTION STRATEGIES

S. ten Bosch¹, D. Terzis¹, and L. Laloui¹

KEYWORDS

Numerical modelling, biocementation, field-scale, multiphysics

ABSTRACT

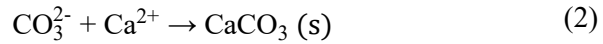
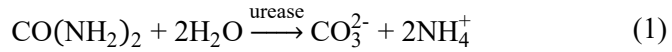
Biocementation through microbially induced calcite precipitation (MICP) is an innovative technology using the precipitation of calcium carbonate, leading to improved soil strength and stiffness. A recently performed upscaling experiment aimed to evaluate the efficacy of diverse injection methods for enhancing the calcite content distribution within an initially unsaturated soil domain. This study evaluates the experiment using numerical modelling, to better understand the complex processes involved and interpret the influence of injection variations. The multiphysical framework represents the bio-hydro-chemical processes of MICP and their couplings. An extensive monitoring campaign allows for a multilevel validation of the modelling approach. Modelling results show that the hydraulic aspect of the injections in the initially unsaturated soil domain could be replicated. The distribution of calcite content and DPT results corroborate the zone of improvement observed in the numerical and experimental data. Additionally, numerical modelling helps to unravel the multiphysical processes in this large-scale field experiment allowing for interpretation of variations in reaction rates and bacteria distribution arising from diverse injection strategies. In this aspect, this study contributes to the development of a numerical framework to serve field-scale, geotechnical applications and contribute to the critical aspects of treatment design and optimization as well as quality assessment and control.

INTRODUCTION

Biocementation for ground improvement is a rapidly advancing area of research, offering an environmentally friendly solution surpassing conventional ground improvement methods' limitations. Utilising ureolysis-based MICP targets the formation of calcium carbonate in the soil through two key reactions. Urea hydrolysis (eq.1), catalysed by the enzyme urease,

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increases pH and forms carbonate and ammonium ions, followed by calcium carbonate precipitation (eq.2) in a calcium-supersaturated environment.



Calcite, the most stable polymorph of calcium carbonate under typical conditions [1], forms within the soil matrix, enhancing soil density and forming bonds between particles. This technology finds applications in the fields of soil strengthening, slope stabilization, liquefaction control and erosion control amongst others.

Designing MICP treatments poses challenges due to complex biochemical processes, necessitating sophisticated tools. Many researchers have developed modelling frameworks encompassing chemical, biological, hydrological, and mechanical processes and their interactions (e.g. [2 - 4]). However, studies often rely on 2D simulations validated with lab-scale samples, which may not accurately capture the complexities in large-scale MICP applications.

Only recently some works began to explore design choices in injection timings and bacteria concentrations using modelling frameworks [5,6], but a comprehensive investigation of the influence of all design parameters on the treatment outcomes remains to be understood. To address this aspect, this work presents a modelling campaign replicating an upscaling experiment described in Büyüklü et al. (2023) [7] where variations in injection rate, volume of injected solutions and injection methods were used.

METHODOLOGY

Theoretical framework

A multiphysical framework including hydraulic, chemical and biological components is used, an overview is presented in Figure 27. Modelling is done with COMSOL Multiphysics vs. 6.2 [8].

The upscaling experiment, initially under unsaturated conditions, necessitates the use of Richards' equation. The Van Genuchten model is used to describe soil water retention and its influence on hydraulic conductivity, akin to Faeli et al. (2023) [9]. Initial Van Genuchten model parameters derive from the grain size distribution of the porous material, with final adjustments made based on calibration with pore water pressure measurements while ensuring realistic values. Mass conservation equations address the transport of diluted species, accounting for diffusion, dispersion, and convection within the

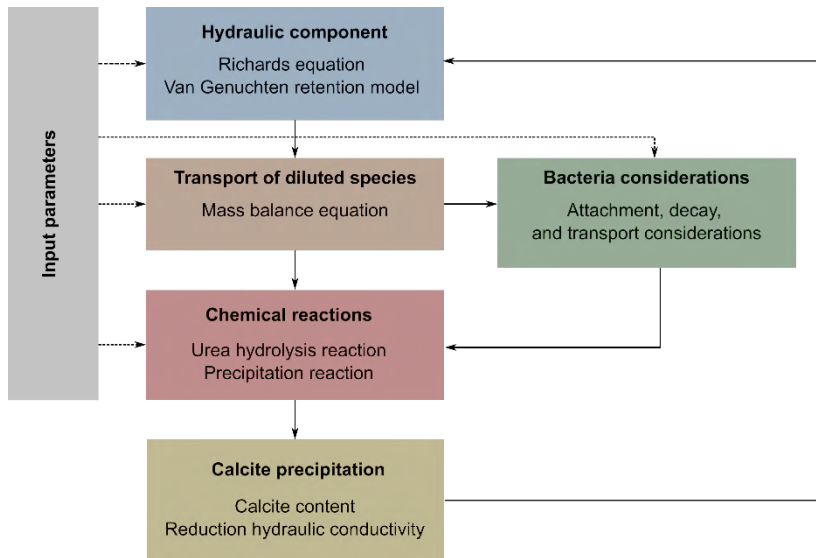


Figure 27, Overview of multiphysical modelling framework.

variably saturated domain. Initially suspended in the injected solution, bacteria distribution is then governed by attachment, decay, and transport. Variations in concentrations over time are modelled using attachment and decay rates, assumed constant as per existing literature (e.g., [5,10]). The urea hydrolysis rate follows Michaelis-Menten kinetics, dependent on bacteria and urea concentrations, and the precipitation rate is defined using carbonate and calcium concentrations. A reduction factor, calibrated from various studies [11,12], is used in the precipitation rate to account for chemical and biological simplifications. A coupling includes the reduction in porosity and saturated hydraulic conductivity as result of calcite precipitation, the latter described using a Kozeny-Carman relationship. Model parameters are based on literature values and not calibrated apart from the ones indicated in this section.

Model of the upscaling experiment

Büyüklü et al. (2023) [7] explored the impact of different MICP solution injection settings and techniques on calcite precipitation distributions. Several injection wells, each varying in flow rates, design, the number of injections or the volume of the injected solutions, were employed. A comprehensive monitoring campaign included the use of piezometers, dynamic penetrometer resistance (DPT) measurements and soil sampling for measurements of calcite content around one of the wells.

The injection wells are modelled as individual experiments using a 2D axisymmetric approach, facilitated by the absence of interactions between wells or walls of the setup. Figure 28 illustrates the general setup of the models, and depicts varied injection configurations and piezometer

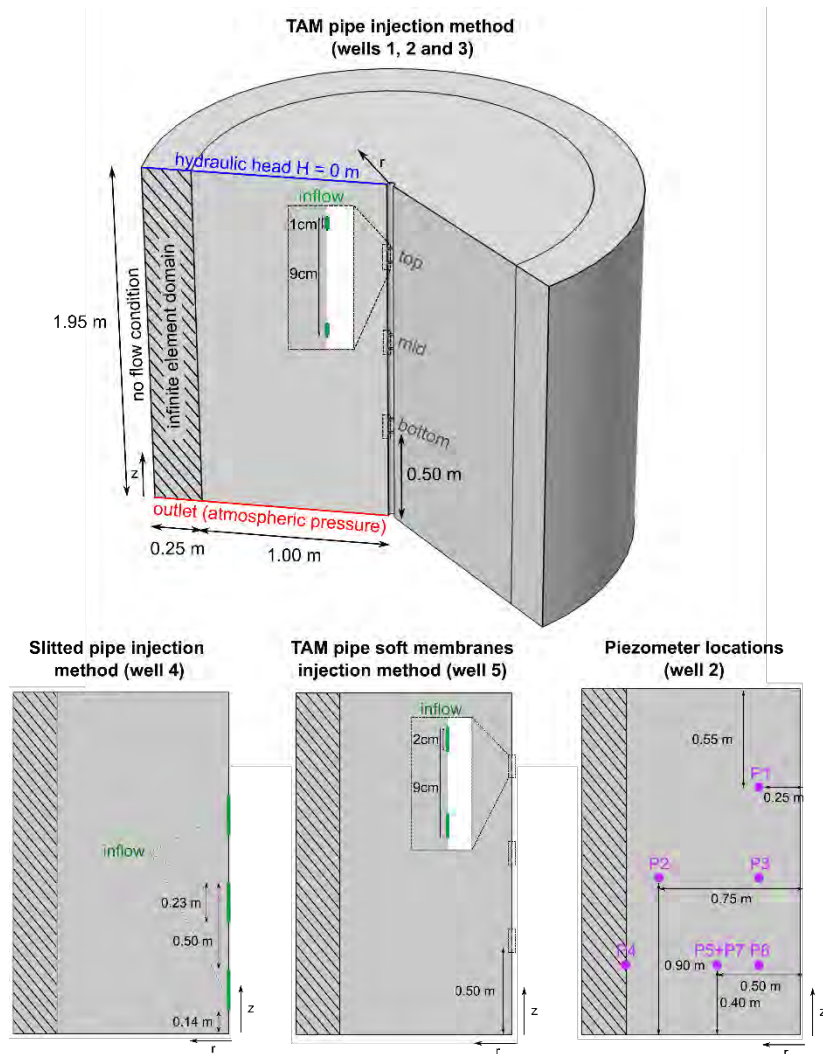


Figure 28, 2D axisymmetric general model geometry and variations to include different injection techniques and piezometer locations.

placements to replicate specific experimental settings for each well. Time-dependent boundary conditions at inflow points mimic the injection patterns applied, specifying relevant flow rates and concentrations of the solutions.

Each model employs a different mesh to accommodate slight geometry variations, but similar mesh quality is ensured. Simulations span from the first injection to the moment of quantification of soil improvement, i.e., the time when DPT testing was performed.

RESULTS

The extensive monitoring campaign allows a detailed comparison between experimental and numerical results.

Pore water pressure profiles obtained with the piezometers are compared with numerically obtained profiles at the representative locations. Results for the piezometers located in proximity to well 2 are presented in Figure 29. For all wells, the profile of improvement obtained through the model can be compared with experimental improvement ranges obtained from the DPT measurements and observations. Numerically obtained calcite profiles for the wells are presented in Figure 30 and an image capturing the experimentally obtained soil columns is presented in Figure 31. Moving to a quantitative analysis, a comparison in terms of calcite content profile surrounding one of the wells was possible after examining the calcite contents of 21 soil samples at varying heights and radii from well 3 using gravimetric acid digestion. A

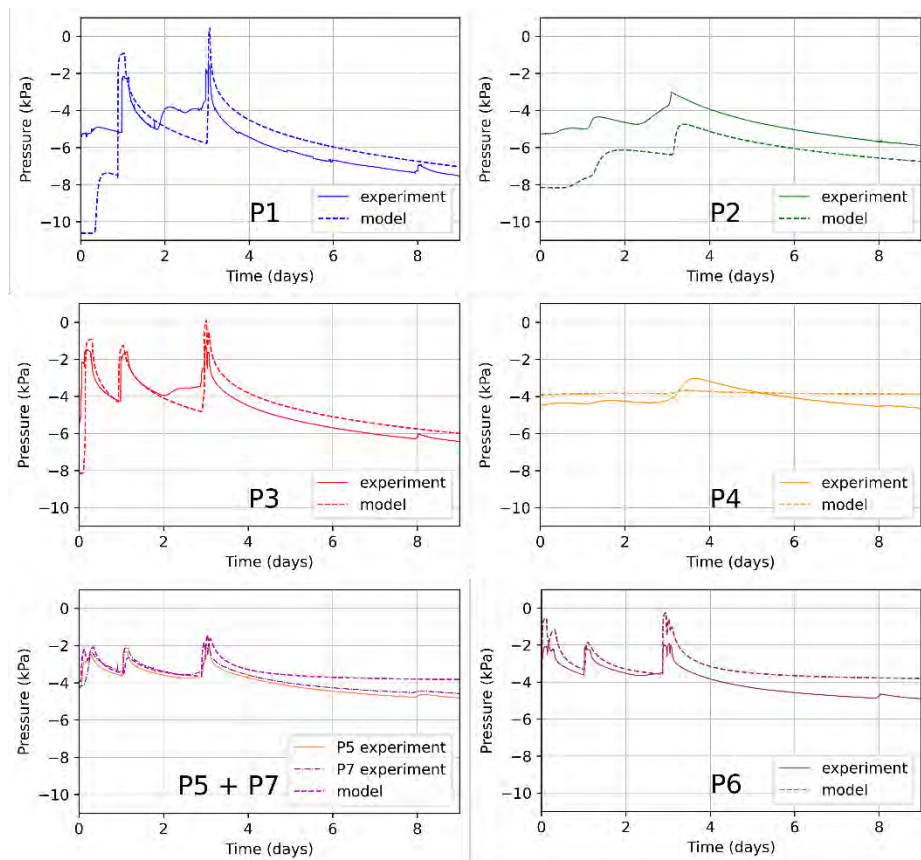


Figure 29, Comparison of experimental and numerically obtained pore water pressures at varying locations in proximity to well 2, at locations as shown in Figure 2.

comparison between these experimental and numerical results is presented in Figure 32.

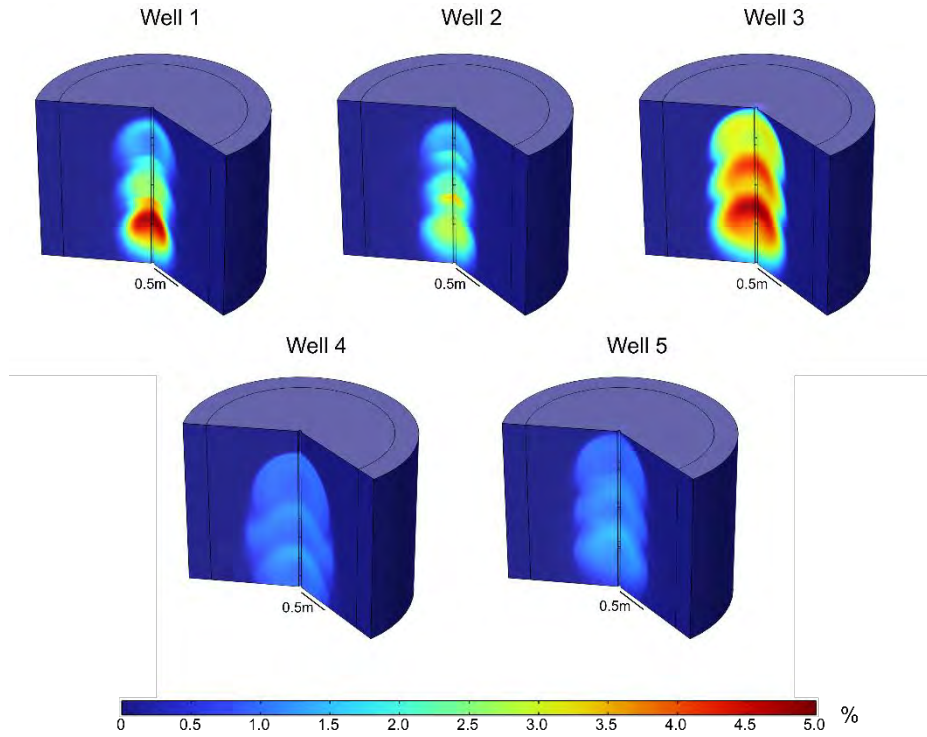


Figure 30, Model predictions of calcite content profiles as result of the treatments applied.



Figure 31, Sand columns obtained as a result of the MICP treatment applied in the upscaling experiment of Büyüklü et al. (2023) [7].

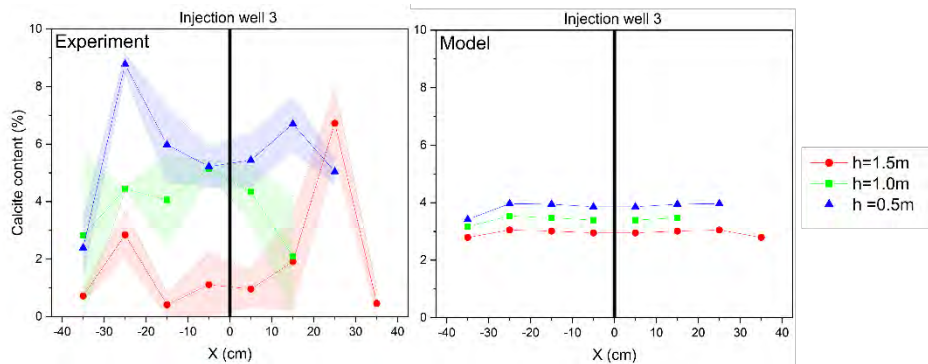


Figure 32, Calcite content for sampling points obtained with (left) experimental campaign and (right) the modelling result.

DISCUSSION

The comparison of pore pressure profiles in Figure 29 shows negative pore pressures relative to atmospheric pressure due to the unsaturated conditions. For locations higher in the soil profile the model initially predicts larger negative pressures, compared to the experimental observations. This is likely due to the starting point of the model not capturing potential influences of rainfall and an initial degree of saturation of the soil before the start of the injections in the experiment. Despite this initial difference, it can be seen that the framework can sufficiently capture the changes in pore water pressures across the soil domain as a result of the MICP treatment by means of injections of solutions.

Figure 32 shows a trend in the experimental measurements of increasing calcite content for deeper positions within the soil profile. This trend is also captured by the model; however, there is a lower variability in the predicted calcite contents compared to the experimental values. Potential sources of error stem primarily from experimental procedures, as gravimetric acid digestion was applied to a soil with a significant natural carbonate content in the untreated state and a relatively high content of fines.

DPT measurements and visual inspection of the soil columns (Figure 31) revealed smaller soil improvement radii for well 1 and 2 compared to wells 3, 4 and 5, along with higher tip resistances around well 3 [7]. Additionally, injecting larger solution volumes, as applied in well 2, did not notably impact soil improvement in terms of radius and resistance. The treatment applied to well 4, using the novel injection technique with a slitted pipe, resulted in a more homogeneous precipitation pattern over the height.

The bio-chemo-hydro framework replicates these qualitative observations (Figure 30), with higher calcite contents and larger improvement radii around well 3. This is primarily attributed to the utilization of a higher injection rate

and the use of multiple cycles of cementation solution injection. The model provides further insights, explaining why for instance increased solution volumes in well 2 had minimal effect: low injection rates during bacterial solution injection resulted in a smaller radius of bacterial attachment, while higher flow rates were used during cementation solution injection. Modelling highlights the complex interplay between the injection rate used and bacteria attachment, which then governs the spatial variation of the urea hydrolysis reaction rate. Similar to the experiment, lower calcite contents are obtained for wells 4 and 5, but the distribution is noted at a larger radial distance from the well and is more homogeneous over depth.

Nonetheless, some disparities persist between experimental and numerical findings, notably in some cases the distance of improvement from the well is overestimated by the model compared to the expected distance based on the DPT campaign. It is expected that these discrepancies partly stem from observed behaviour that cannot be replicated, such as surface settlement upon fluid injections, erosion of soil near injection points, and non-functioning injection outlets, which were all observed in the upscaling experiment.

CONCLUSIONS

This study aims to demonstrate the capabilities of a modelling framework to capture key trends observed in a large-scale experiment and to enhance understanding of variables to ultimately master the design of MICP treatment. Main trends in pore water pressures, and calcite content distributions are captured, while highlighting the key role of the used injection rate. Thereby, this study aids in advancing modelling tailored for field-scale MICP applications.

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NUMERICAL STUDY OF THE THERMAL PERFORMANCE OF ENERGY MICROPILES IN NORDIC REGION

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KEYWORDS

Energy micropiles, thermal performance, Numerical simulation, Nordics

ABSTRACT

The idea of using underground structures for heating and cooling was first concretized in the 1980s with energy slabs, followed by piles, retaining walls and tunnels [1]. These structures, which play an energy role in addition to their conventional structural role, are referred to as “energy geostructures”. Pile foundations are currently among the most used energy geostructures. They are particularly suitable for utilizing the ground thermal energy, as they extend to depths where soil temperature is quite constant and independent from daily or seasonal variations [2,3]. However, for structural retrofitting, underpinning of existing foundations, and in areas with space, vibration, and noise constraints, micropiles are the most suitable solution. Moreover, micropiles are suited for difficult ground and drilling conditions as is the case in many Nordic countries.

Due to its very cold climate, the Nordic region needs more heating than cooling. This study investigates the thermal performance of an energy micropile in the Nordic region. The results show that the energy micropile provided a thermal power of about 31 W/m after a 30 day-simulation period, which demonstrates the potential of energy micropiles for heating buildings in the Nordic region during winter. However, a drop in heat output was observed over time, which later stabilized. Therefore, for long-term use, a hybrid solution of energy micropiles, solar panels and district heating systems is recommended to compensate for the unbalanced thermal load and increasing the performance of the system. In addition, the group effect of energy micropiles and the thermal effects of the presence of grout in the system should be further investigated.

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INTRODUCTION

In the early 1950s, in response to the demand for innovative techniques to underpin historic buildings and monuments, the “palo radice” or “root pile” was developed by an Italian construction company, Fondedile [4]. The technology was later introduced in several other countries, and “root pile” was replaced by “micropiles”. Micropiles are smaller piles with a diameter of less than 30 cm and a length of usually less than 30 m [4]. Despite their small size, large loads of up to 5000 kN can be sustained thanks to the high-capacity steel elements utilized. They are particularly suitable when soil and drilling conditions are difficult. The equipment used to install micropiles is not as large and heavy as that required for conventional piles. Therefore, they can easily be used in swampy areas and in areas with wet or soft surficial soils [4], as is the case of the Nordic region, with minimal impact on the environment. The most populated areas of the Nordic region are covered with very soft soils, such as marine, glacial, and post-glacial clays, and silts [5]. There exist several study cases on the use of micropiles in soft soils [6,7]. Today, micropiles are mostly used as foundation support elements, underpinning of existing foundations, structural retrofitting and in situ reinforcement. Depending on the ground properties and the required bearing capacity among many factors, micropiles made of steel, grouted, driven, or drilled may be used. Figure 1 shows a case study of the installation of a grouted micropile in an existing building for seismic strengthening.

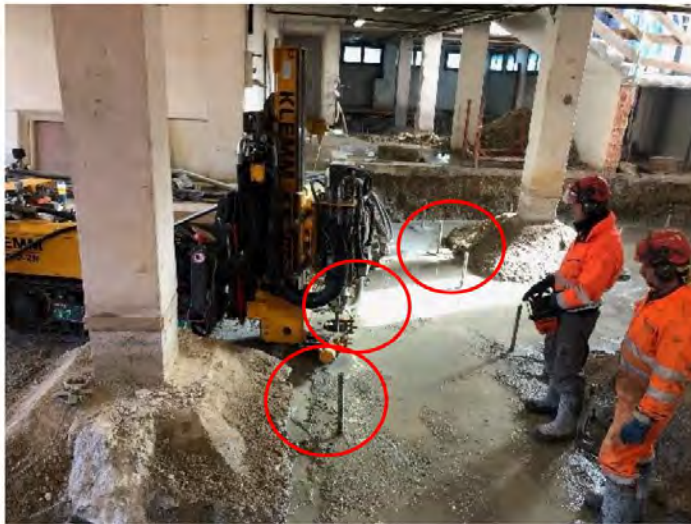


Figure 33 Installation of TITAN micropiles for seismic retrofitting (the red circles enclose the top of some micropiles).

The use of micropiles could be extended to the heating and cooling of buildings, by using plastic pipes in which a heat exchanger fluid circulates, exchanging heat between the ground and the building (Figure 2).

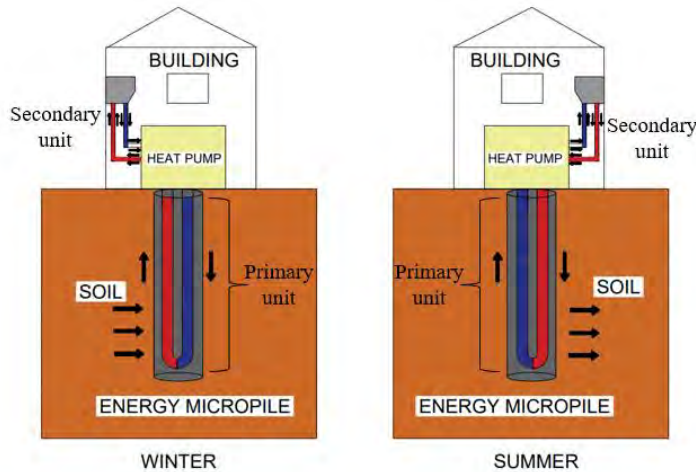


Figure 34 Scheme of energy micropile systems in a building.

In this way, micropiles can provide both structural and energy rehabilitation [8] and are then referred to as energy micropiles. In fact, energy micropiles have proven their efficiency in cooling of buildings in summer, with a thermal performance of about 30W/m [9], comparable to that of energy piles. In the Nordic countries, which are known for their very cold climate, more heating is required than cooling. This numerical study investigates the thermal performance of an energy micropile in the Nordic region. The software COMSOL Multiphysics has been used for the analysis. The model was created and validated with TRT test data available in literature and further used to simulate an energy micropile over a period of 30 days under winter conditions.

NUMERICAL SIMULATION

The software COMSOL Multiphysics version 6.1 (COMSOL, 2018) was used in this study to conduct the thermal analysis. The 3D model comprises three main elements: pipes, the structural body (composed of grout and/or steel), and the ground. The micropile was 12 m long and 20 cm wide with a 103/72 steel pipe reinforcement in which a 32 mm pipe was introduced. The model was validated using the thermal response test data in [10].

Mathematical formulation

The equation governing incompressible flow is derived from the fundamental principles of conservation of mass and momentum. Specifically, the

conservation of mass principle for a stationary and incompressible flow is mathematically expressed by the continuity equation as:

$$\nabla \cdot \rho \mathbf{u} = 0$$

The Navier-Stokes equation for incompressible flow, accounting for transient effects, is expressed as [11]:

$$\rho \left(\frac{\partial \mathbf{u}}{\partial t} + (\mathbf{u} \cdot \nabla) \mathbf{u} \right) = -\nabla p + \rho \mathbf{g} + \mu \nabla^2 \mathbf{u}$$

where ρ represents the fluid density [kg/m³], $\mathbf{u}$ denotes the velocity field of the fluid [m/s], p signifies the pressure field [Pa] and μ denotes dynamic viscosity. The heat transfer in solids and liquids interface of Comsol Multiphysics solves for the following equations [12]:

$$C_p \left(\frac{\partial T}{\partial t} + \mathbf{u} \cdot \nabla T \right) + \nabla \cdot (\mathbf{q} + \mathbf{q}_r) = Q_{ted} + Q$$

$$C_p \left(\frac{\partial T}{\partial t} + \mathbf{u} \cdot \nabla T \right) + \nabla \cdot (\mathbf{q} + \mathbf{q}_r) = Q + Q_p + Q_{vd}$$

Where $\mathbf{q}$ is the heat flux by conduction [W/m²], $\mathbf{q}_r$ is the heat flux by radiation [W/m²], Q_{ted} is the thermoelastic damping and accounts for thermoelastic effects in solids and Q_p represents the work done by pressure changes and Q_{vd} represents viscous dissipation in the fluid.

Initial and boundary conditions

The soil and micropile domains were assumed to be isotropic and homogenous and at an initial temperature of 10°C. A thermal insulation boundary condition was assumed at the top surface of the soil domain, while a temperature of 10°C was fixed at both the bottom and perimeter boundaries. A coaxial pipe configuration was adopted in the grouted micropiles wherein the steel tube served as the inlet pipe and a centre HDPE pipe as the outlet pipe. A fine mesh distribution was adopted in the model as illustrated in Figure 3.

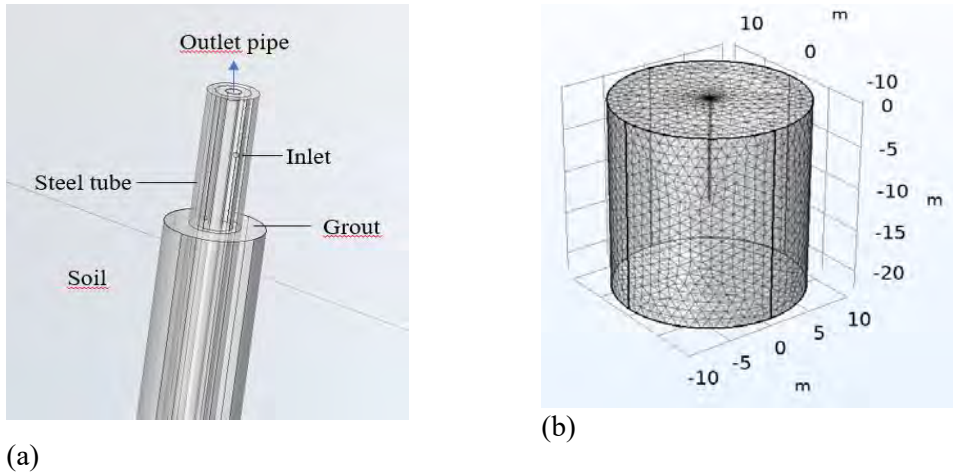


Figure 35 3D FEM model:(a) geometry and (b) mesh distribution.

The flow rate and inlet temperature of the fluid were $10^{-4} \text{ m}^3/\text{s}$ and 4°C respectively. A nonlocal integration coupling over the boundary surface was defined to evaluate the outflow temperature at the outlet point of the pipes. The thermal power was calculated using the following thermal energy equation as:

$$H = \frac{Q_{in} \rho_{hcf} C_p \Delta T}{L}$$

The properties of the materials used in the simulation are given in table 1.

Table 1. Main properties of materials used in the simulation

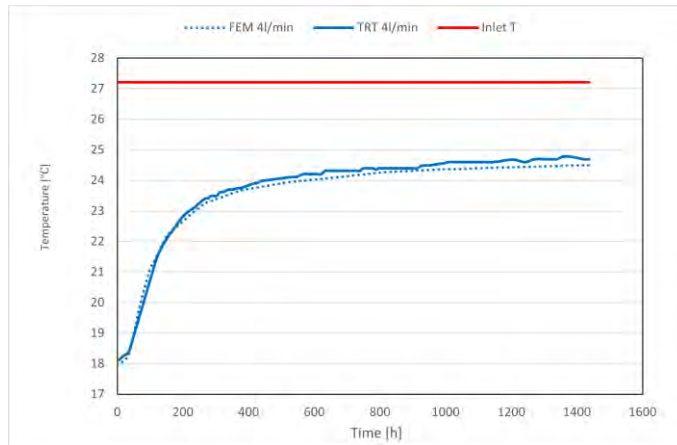
Properties	Fluid (water)	Pipes (HDPE)	Steel (S355J2H)	Grout	Soil (Clay)
Thermal conductivity [W/(m K)]	0.6	0.6	45	1.6	1.2
Specific heat capacity [J/(kg K)]	4186	2000	420	880	1800
Density [kg/m ³]	1000	950	7850	2300	1700

RESULTS AND DISCUSSION

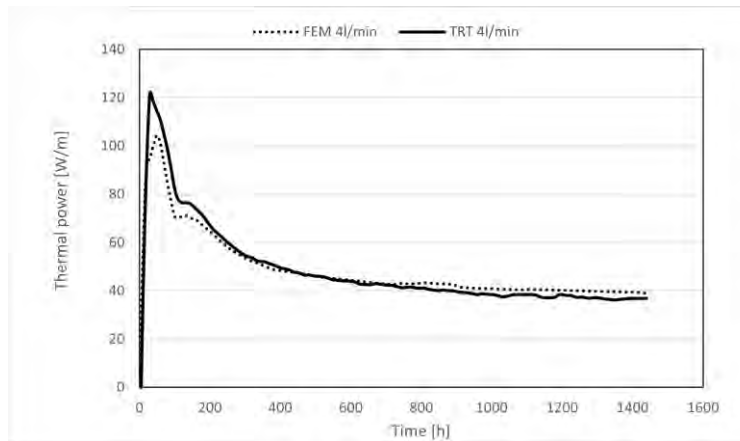
Model validation

As previously indicated, the thermal response test data of [10] was used to validate the model. A perfect agreement of the outlet temperature and the

thermal power can be observed between the tests results and the simulation results (Figure 4).



(a)



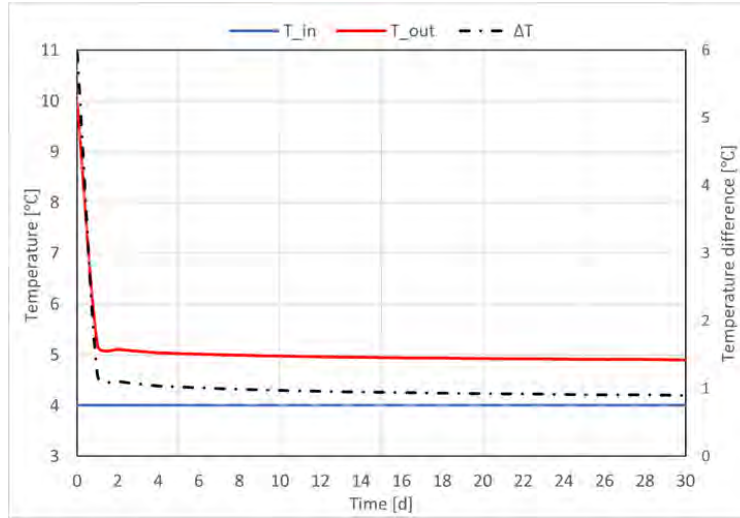
(b)

Figure 4 Comparison between thermal response test data and FEM results: temperature (a) and thermal power (b).

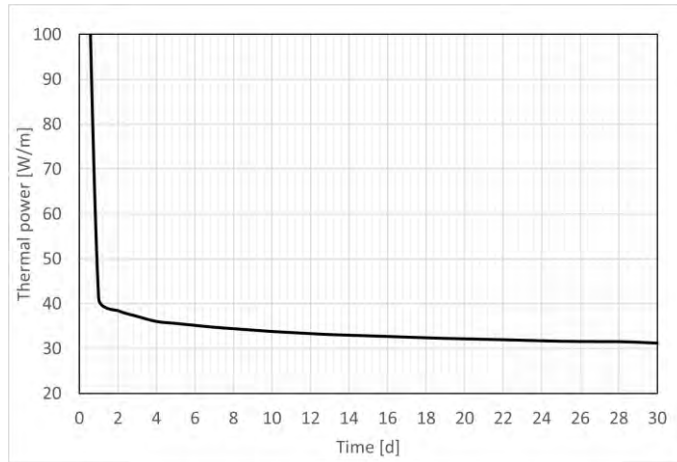
The largest difference is observed at the peak of the thermal power plot, representing an approximate difference of 15%. This validation confirms the reliability of the numerical model, affirming its suitability for further analysis.

Simulation results

The validated model was then used for the simulation. Figure 5 shows the fluid outlet temperature with time.



(a)



(b)

Figure 5 Simulation results in terms of: temperature (a) and thermal power (b).

An outlet fluid temperature approximately equal to its initial temperature is observed at the beginning of the simulation when $t < 1$ day. However, after 1 day, the temperature considerably drops to 5°C thereby decreasing ΔT . As time progresses, ΔT decreases more slowly and the outlet temperature eventually stabilizes at a value of 4.9°C . The same pattern is observed with thermal power, where the thermal power at $t = 1$ day is 41.13 W/m which then reduces to about 31.17 W/m after 30 days.

The performance of the system is highest at the beginning when the temperature difference between the soil domain and the pile domain is largest. As the exchange fluid circulates in the system, it extracts heat from the ground and therefore leaves at a higher temperature. With more heat extraction, the difference in temperature between the two domains decreases thereby reducing the performance of the system. Furthermore, as micropiles are usually installed in a group, although a large number increases the total heat extraction from the ground, this may lead to a faster thermal equilibrium in the system. Moreover, due to thermal interference, the average thermal performance of a single micropile may decrease over time. In addition, although the reinforcement tube is usually flushed before the exchange fluid is circulated, some grout may still be present in the annular space, which may lead to grout intrusion in the heat pump and thus disrupt the thermal performance of the system.

CONCLUSIONS

In this paper the thermal performance of a grouted coaxial energy micropile installed in clay during winter was analysed for the Nordic region. The 30-day study shows that the energy micropile can provide about 31.17 W/m which illustrates its potential for heating buildings in the Nordic region. However, since the thermal performance of the system was observed to decrease over time, it is recommended to use a hybrid system of energy micropiles, solar panels and district heating systems to balance the unbalanced thermal load and improve performance. In addition, the effect of a larger number of micropiles in a group and the presence of grout in the annular space may reduce the thermal performance of the system and should therefore be further investigated.

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ON THE BENEFITS OF VISUAL PROGRAMMING IN GEOTECHNICAL PLANNING PRACTICE

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KEYWORDS

Visual Programming, Automation, Geotechnics, VPL, BIM

ABSTRACT

Currently, the adoption of the method of Building Information Modelling (BIM) in geotechnical planning practice is rising. Most of the BIM-authoring tools offer the possibility to use visual programming environments, which also allow for text-based scripting within them. These can exemplarily be used to extend software capabilities and to automate workflows. Whilst practitioners occasionally utilize these tools, there is no comprehensive analysis on the use cases of them in geotechnical planning processes. In this contribution these use cases are analyzed, categorized and evaluated considering all common major steps in the planning phase. Identified applications are (I) parametric and computational geometry, (II) visualization and documentation, (III) integration and automatization, (IV) model management and coordination and (V) design analysis and simulation.

INTRODUCTION

The adoption rate of Building Information Modelling (BIM) is increasing across all disciplines involved in the construction process. A relevant part of its promised efficiency gains results from automation routines that run on top of the information model. BIM authoring tools provide various built-in routines for common tasks. However, in construction projects specific problems may arise that cannot be adequately supported by these built-in routines. These scenarios more frequently occur with increasing project complexity and as the use differs from the authoring tool's intended scope. To overcome this hurdle in workflow automation, many authoring tools enable users to write custom routines. Visual programming environments (VPE) are a common option for doing so.

Research question

Geotechnical projects utilizing the BIM methodology are often complex and use authoring tools, that are not specifically designed for geotechnical engineering

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problems. Therefore, various custom routines can be expected throughout the planning process to enable efficient workflows. The aim of this contribution is to explore the application and benefits of visual programming in geotechnical planning processes.

Methodology and structure

The following section of this contribution presents the main terms and concepts in visual programming. It then conducts a literature study on their use in the AEC sector with focus on geotechnical engineering. The identified applications are clustered and the underlying benefits are discussed. An illustrative example of a pile foundation is created to demonstrate the applicability to a common geotechnical problem. The present study concludes by highlighting the potential of visual programming in geotechnical engineering.

LITERATURE STUDY

Visual Programming Languages (VPLs)

A programming language is considered a VPL, if it includes visual expressions in its syntax [1]. VPLs typically follow the dataflow paradigm, hence they are also known as *flow-based* [1], [2]. This means that they illustrate the flow of information from input to output within an information processing system. Figure 1 displays a script written in Autodesk Dynamo, that generates a cylinder. A VPL script is created by placing nodes from a node collection on a canvas and connecting the node outputs to node inputs with wires. The dataflow can be read from left to right by following the wires from node to node. If an input parameter, such as the number slider defining the circle's and therefore the pile's radius is changed, all subsequent nodes are updated when the script is recomputed. Geometric-centric VPLs, commonly used in AEC, allow for rapid iteration over potential design solutions based on user interactions.

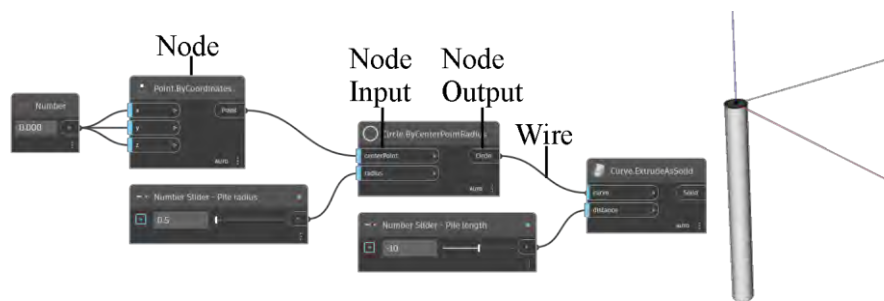


Figure 36 Dynamo-Script for creating a cylinder / pile and its resulting geometry

The advantages and disadvantages of VPL compared to traditional textual programming are well-discussed. VPLs are easier to learn, as investigated in regard to query languages in [3] and [4] and exemplified by Scratch, a VPL designed for teaching programming fundamentals. The visualization of the data flow may benefit visually-orientated people and VPLs are well-suited for rapid prototyping. On the downside, the execution time tends to be slower, and functionality is limited. Additionally, larger scripts or projects can be harder to organize and maintain.

VPLs in architecture, engineering and construction (AEC)

Many BIM authoring tools and CAD packages come with their own VP plugins or provide an interface to interact with such tools, although there are also standalone VP environments. Examples include GenerativeComponents in various Bentley programs, Grasshopper in Robert McNeel and Associates' Rhinoceros3D, Dynamo in various Autodesk programs, Geometry Nodes / Sverchok in Blender and Marionette in Vectorworks. This means that no additional installations or setups are necessary, making them more accessible.

In VPLs, scripting nodes are often available, allowing for textual programming within the VPL. This provides greater flexibility and enables the creation of more complex and advanced programs. Therefore, the use of custom scripts is actually addressed when discussing the benefits of VPLs.

Ritter et al. [5] discuss the use of VPLs in the AEC sector. They identify six use cases, which can be grouped into two categories: (1) generation of geometric and semantic information and (2) querying and analysis of existing models [2]. The first group includes geometric modelling, knowledge-based design, and system modelling, while the latter group includes query languages, design decision support, and code checking.

Collao et al. [6] reviewed the applications of VPLs in infrastructure projects. They found, that most papers using VP in this domain address the design phase and within that, parametric design being the dominating use case. The authors also noted a rising popularity of these tools, as evidenced by the increasing number of publications in recent years. They conclude, that future efforts are expected in improving the interoperability with web applications expanding the use of VPLs in monitoring.

VPLs in geotechnical engineering

In the following, examples of the VPL usage in geotechnical engineering are listed.

In Khan et al. [7] Grasshopper is utilized for earthworks modelling through a voxelization algorithm. Kahlström and Hansen [8] employ Grasshopper to create as-built-models of dry deep mixing ground improvements based on

machine data. VPLs are used for geometry creation, data analysis and visualization. In Alaei [9] Dynamo is utilized for geometry processing and generation, including structures like piles and tunnels as well as the subsoil model. Ninić et al. [10] present an integrated workflow for BIM and numerical simulations using Dynamo and Python for geometry creation, data processing and post-processing. Huang et al. [11] use Dynamo with Python scripting nodes in a similar workflow for geometry creation (subsoil and underground metro station), preparation of executable scripts for numerical simulations and visualization of calculation results. In [12–14] meta models are additionally run in Dynamo using Python components enabling a real-time design assessment. Khan et al. [15] use Grasshopper for excavation safety modeling. In detail, geometry and semantic information is queried, analyzed and new geometry is created based on knowledge. Quantity take-offs for the new geometry are then conducted. El Sibaii et al. [16] employ Dynamo to process data from geotechnical investigations and create an information model from them. Zhu et al. [17] leverage Dynamo for manipulating BIM elements, data processing, constraint handling and design optimization in a segmented tunnel design. Wang et al. [18] use Dynamo geometric modelling, automating parameter assignment and create cross-passages within a tunneling project. Xun et al. [19] use the same VPL for visualization, data import and export in a tunneling project. They establish a connection to a MySQL database, reading sensor data periodically. Several other authors have addressed the parametric modelling of tunnels, e.g. [20–25].

Dynamo is used for modelling a subway station, extracting parameters and creating files for running a numerical simulation in Xie et al. [26]. In Jiao et al. [27] Dynamo is utilized to identify and resolve collisions of outside corner anchor rods in excavation pits. Additionally, data export and real-time visualization is achieved using the VPL. Filardo et al. [28] leverage Dynamo and its Python environment to create an automated and optimized construction schedule for a pile foundation. Peansupap et al. [29] use Dynamo for creating an automated system for clash detection and resolving in strut arrangements of deep excavations. Chuang et al. [30] leverage the same VPL for modeling a deep foundation pit. Barbieri et al. [31] highlight, that VPLs can be used for creating specific modeling and parametrization tools ad hoc and use automating rock bolts placement along lines with constant spacing as an example. Grasshopper has also been used to optimize slab and pile design layouts [32,33].

In previous research of the authors, VPLs have been applied to various problems related to the calculation of geotechnical structures such as pile foundations and sheet pile walls. Use cases include integration of heterogeneous data sources, models and software applications, analysis and visualization and reporting [34–36]. For this paper, an educational example has been developed. It demonstrates fundamentals of VPLs using a pile

foundation. Features include automatic adaption of pile length based on minimal depth within the load-bearing soil layer and automated report creation. The script and additional contents can be accessed in [37].

To conclude, the literature on VPLs in geotechnical engineering is compared with findings in [6], which focuses on infrastructure projects. In both domains, VPLs are mostly used in the design stage. Another mutuality is, that Dynamo is by far the most common VPL, followed by Grasshopper. The papers listed above mainly use Dynamo in conjunction with Autodesk Revit. Within this setup, a common approach is to create reusable parametric elements within Revit, named families, then use Dynamo to place them inside the project and control their parameters. The tendency in [6], that VPLs are becoming more popular in recent years reflects also in the found literature on VPLs in geotechnical engineering. Tunnels are the type of geotechnical construction, where until now most uses of VPLs can be found in literature.

It is evident, that many of the previous works utilize tabular data formats as starting point or for storing the results. Therefore, tools such as Microsoft Excel play an important role in current automation practices in the field of geotechnics.

Regarding the use of VPLs, five major groups can be identified: (I) parametric and computational geometry, (II) visualization and documentation, (III) integration and automation, (IV) model management and coordination and (V) design analysis and simulation. As mentioned, the first application is the most common. As for visualization, the most frequently seen technique is coloring objects within the 3D / BIM model. Reporting and drawing generation are rarely addressed in literature. Automation and integration of various data sources and applications are carried out on both small and large scales, with the aim of achieving efficiency, particularly when designs are altered. There are only a few publications that indicate the use of VPLs in model management and coordination. This may be because these tasks are likely not within the scope of geotechnical engineers in larger project. Yet, they are used for quality assurance of geotechnical models. Finally, three options for using VPLs in design analysis and simulation are identified: (I) performing the calculations within the VPL, (II) use the VPL to create input files for another application and (III) load external models for calculation within the VPL.

CONCLUSION

The conducted literature review indicates that the trends of VPLs in geotechnical engineering align with those in the infrastructure sector. The benefits of VPLs in geotechnical engineering include: (I) Automation of repetitive tasks leads to efficiency gains, (II) the expansion of software capabilities enables the use of generic BIM tools in the field of geotechnics,

(III) improved interoperability is achieved through importing, processing and exporting capabilities in VPLs, (IV) design quality is improved through by knowledge-based design, simple design iterations, and automated issue detection and resolving.

VPLs provide a relatively easy way to access and modify the data in BIM projects. As these projects are increasingly demanded and a trend towards a more data centric way of working can be observed, further increases in VPL adaption in geotechnical engineering can be expected.

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ON THE INFLUNCE OF SYSTEM STIFFNESS FOR NUMERICAL DESIGN APPROACHES

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KEYWORDS

Design approach, Eurocode 7, numerical simulation, FEM, excavation pit

ABSTRACT

The use of the Finite Element Method (FEM) to predict deformations at the serviceability limit state (SLS) has become established in geotechnical engineering over the past few decades. The revision of Eurocode 7 provides numerical methods to be used in the design procedures for the ultimate limit state (ULS). To this end, the Input Factoring Approach (IFA) and the Output Factoring Approach (OFA) are introduced. At present, it is not clear which boundary conditions are relevant for the different design approaches.

Research has focused on the influencing factors of excavation pit design for decades. As a result, the most important parameters for excavation pit design are the shear parameters (ϕ' and c'), the tensile stiffness EI of the wall and the soil stiffness E_s . The influence of different parameters on the design approach (IFA and OFA) for a single braced as well as a double braced excavation pit is analysed by means of a parameter variation. The design-relevant method is determined by applying known stiffness factors for the analysed systems. In conclusion, the results are discussed and an outlook on future research directions is outlined.

INTRODUCTION

In Germany the FEM is currently primarily used for predicting deformations in SLS. Therefore, numerical methods are applied at a relatively late stage in the construction planning process. The revision of Eurocode 7 introduces a normative regulation of the use of FEM for ULS design in geotechnical engineering for the first time [1]. Stability analysis will be conducted using two methods: Input Factoring Approach (IFA), which reduces shear parameters, and Output Factoring Approach (OFA), which factorizes characteristic internal forces resulting from numerical calculations. The analyses will cover both ground failure and structural failure of the geotechnical structure.

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However, both approaches require time-consuming calculations that can be complicated by occurrence of various failure mechanisms or numerical instabilities. At an early planning stage, this may not be feasible due to limited ground information, and therefore established analytical methods are used. Therefore, it would be advantageous to use only one method in early stages of the planning process. To understand the design-relevant approach (IFA or OFA), it is important to consider the various influencing factors. Various authors have examined the factors influencing excavation pit design [2-6]. The present paper analyses the factors influencing the design approach using two synthetic excavation pits and applies stiffness parameters for the system from literature to analyse the design.

THEORETICAL BACKGROUND

In the design of geotechnical structures, it is necessary to factorize both actions and resistances with partial safety factors. However, due to the non-linearity of the soil material, this procedure cannot be carried out by means of numerical methods. To address this issue, both the IFA and OFA method, which consider both actions and resistances with partial safety factors (refer to Table 1), will be used in future. Numerical simulation has the advantage of taking into account the stress history. Nevertheless, the appropriate point in the calculation to factorise the input parameters is uncertain. One option is to apply the partial safety factors at the beginning of the simulation (IFA, variant A), while the other is to apply them in form of a bifurcation calculation during the design phases (IFA, variant B).

Table 1. Partial factors for IFA and OFA.

	OFA	IFA
Permanent action (G_k)	$\gamma_G = 1.00$	$\gamma_G = 1.00$
Variable action (Q_k)	$\gamma_Q/\gamma_G = 1.50/1.35$	$\gamma_Q = 1.30$
Effects of actions (E)	$\gamma_E = 1.35$	$\gamma_E = 1.00$
Internal friction ($\tan \varphi'$)	$\gamma_{\tan \varphi} = 1.00$	$\gamma_{\tan \varphi} = 1.25$
Cohesion (c')	$\gamma_c = 1.00$	$\gamma_c = 1.25$

Previous research [2-6] has used numerous case studies to investigate the influence of behaviour on the deformation as well as the load-bearing capacity of excavation pits. However, due to the complexity of the soil material and the numerous boundary conditions, it is almost impossible to identify generalised dependencies. In soft soils, Clough et al. [3] define the base heave as the significant failure mechanism due to plastic flow in the area of the earth

support. To contain the associated horizontal wall deformation $u_{x,\max}$, Clough et al. [3] define a system stiffness factor according to Equation 1 that ensures a factor of safety against basal heave FOS_{base} . The system stiffness requires the wall stiffness EI , average vertical support spacing $\bar{h}$ and unit weight of water γ_w as input parameters.

$$\frac{EI}{\gamma_w \bar{h}^4} \quad \text{Eq. 1}$$

Bryson and Zapata-Medina [6] present a generalised approach to analysing excavation structures. They compile all parameters that influence the safety of the excavation against basal heave, based on three-dimensional numerical analyses of a triple braced excavation pit. The relative stiffness ratio R , defined by Equation 2, can be classified into three groups. Firstly, the ratio of the wall stiffness parameters E and the in-situ soil stiffness E_s . Secondly, the geometric parameters above the excavation base in form of the average vertical $\bar{h}$ and horizontal support spacing s_H , the excavation depth h and the moment of inertia of the wall I . And thirdly, the parameters that affect the earth support area: average weight of the soil γ_s , undrained shear strength s_u and wall length H .

$$R = \frac{E}{E_s} \frac{\bar{h} \cdot s_H \cdot h}{I} \frac{\gamma_s \cdot H}{s_u} \quad \text{Eq. 2}$$

NUMERICAL MODELS

Two synthetic systems are analysed: a single braced excavation according to Schweiger [7] and a double braced excavation according to Dahmen [8]. The numerical models' sections, including the discretisation, are shown in Figure 1. The mesh uses 15-node triangular elements with a target element size of $l_e = 4.002$ m for system 1 (4,827 elements) and $l_e = 3.002$ m for system 2 (2,497 elements), with local mesh refinement in the excavation pit area. The distance to the boundary of the model is selected to be large enough ($2 - 3h$) to avoid any significant boundary effects.

The Hardening Soil constitutive model by Schanz et al. [9] is used to model the sandy soil, with parameter values presented in Table 2. To conduct the parameter study, the stiffnesses and strengths of the soil are varied using the parameters listed in Table 2, with the initial parameters highlighted in bold. The model does not include any hydraulic boundary conditions. The numerical model implements either an AZ 18-700 (System 1) or a Larssen 43 (System 2) sheet pile wall with elastoplastic material behaviour. Additional sheet pile walls, including AZ 36-700, AZ 48-700, and an 80 cm wide diaphragm wall for System 1, and Larssen 25, Larssen 430, and an 80 cm

wide diaphragm wall for System 2, are also analysed in the parameter study. The struts exhibit elastoplastic material behaviour with a tensile stiffness of $EA = 493,500 \text{ kN/m}$ for system 1 and $EA = 150,000 \text{ kN/m}$ for system 2.

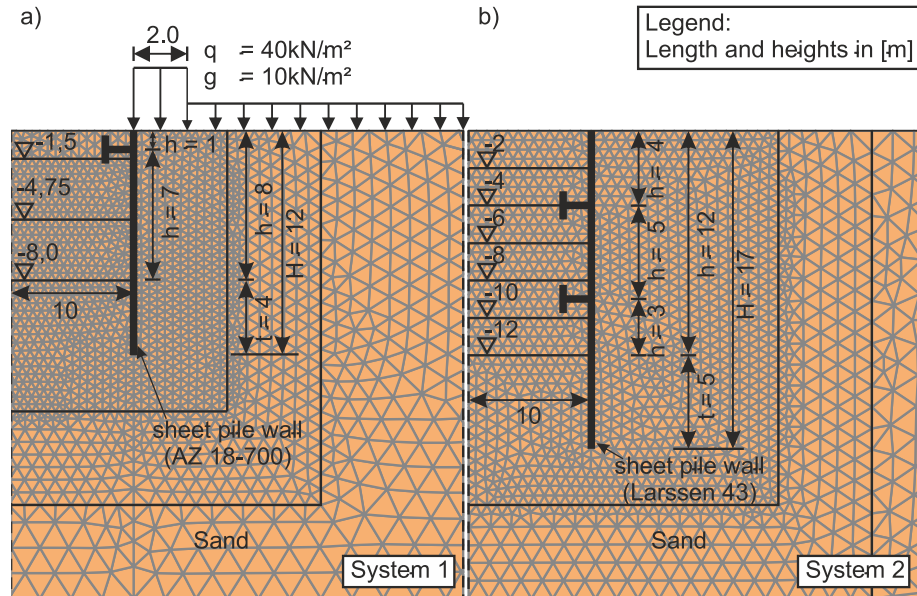


Figure 1 Numerical model for the single braced excavation (a) from Schweiger [7] and the double braced excavation (b) from Dahmen [8].

Table 2. Material parameters for the Sand (Hardening Soil model). Initial parameters are shown in bold.

γ_{unsat}	γ_{sat}	E_{50}^{ref}	$E_{\text{oed}}^{\text{ref}}$	$E_{\text{ur}}^{\text{ref}}$	m	ν_{ur}
[kN/m ³]	[kN/m ³]	[MN/m ²]	[MN/m ²]	[MN/m ²]	[-]	[-]
18	19	20 , 40, 60, 80, 100	20 , 40, 60, 80, 100	60 , 120, 180, 240, 300	0.5	0.2
φ'	c'	ψ	R_{inter}	p_{ref}	K_0^{nc}	R_f
[°]	[kN/m ²]	[°]	[-]	[kN/m ²]	[-]	[-]
25, 30 , 35, 40	0.1 , 5, 10, 15	0	0.63	100	0.500	0.9

RESULTS OF THE PARAMETER STUDY

Figures 2 and 3 display the maximum and minimum bending moments (M) for various retaining walls, stiffness as well as strength parameters, in the final state of the excavation resulting from the design calculation according to IFA, variants A and B as well as OFA. In addition, the results for the final state with characteristic parameters are presented.

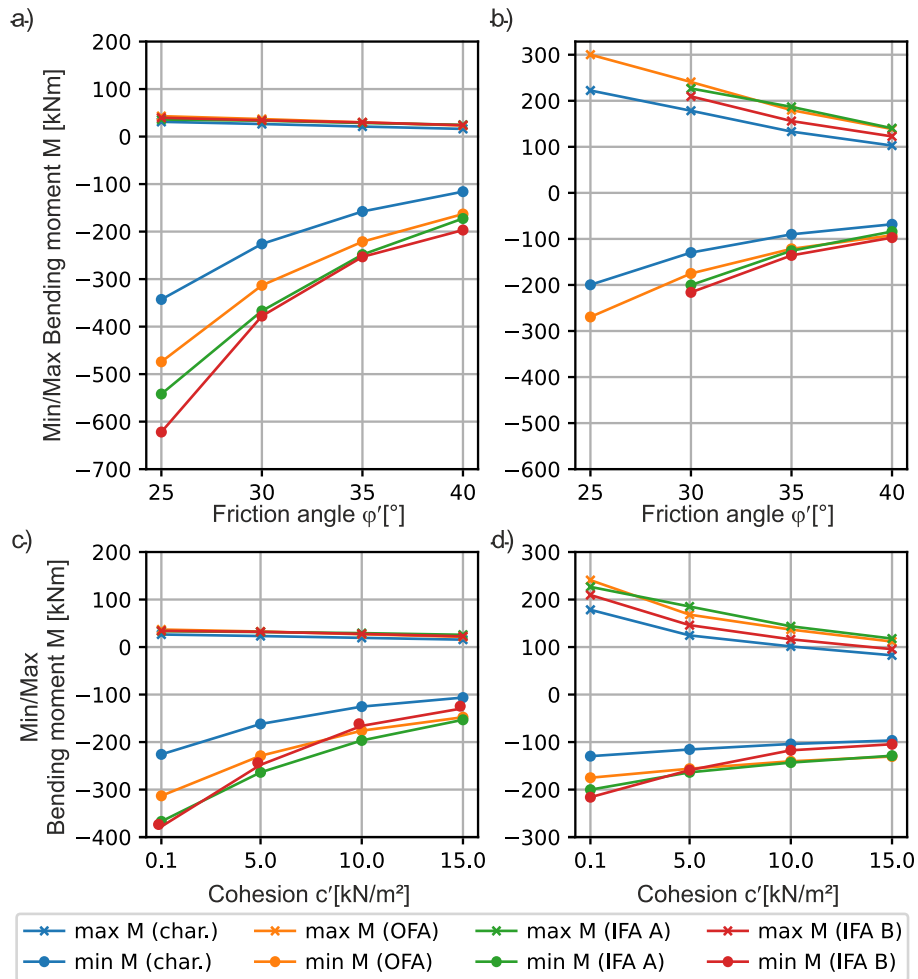


Figure 2 Maximum and minimum bending moments M for the varied values of the friction angle ϕ' (a and b) and cohesion c' (c and d). System 1 left column and system 2 right column.

IFA, variant B is the dominant design approach for the systems 1 and 2 when varying the soil strength parameters (ϕ' and c') in Figure 2, as the minimum

moment $M_{\min}$ is relevant for the design of the sheet pile wall. It is evident that the delta Δ in minimum bending moment $M_{\min}$ between IFA (variant A or B) and OFA decreases if soil strength increases.

Only for system 2 with $\varphi' = 30^\circ$, the OFA calculation results in a slightly higher maximum bending moment $M_{\max}$ at the strut compared to the IFA calculation (see Figure 2b). This indicates that OFA tends to become design-relevant when a stiff component dominates the system, regardless of whether it is the soil or the strut.

The investigation of the different soil stiffnesses in Figure 3a and b indicates no significant influence on the design-relevant approach, with the oedometric stiffness E_{oed} serving as the representative value on the x-axis. Figures 3c and 3d demonstrate that different wall types (represented by the moment of inertia I_y) result in design-relevant bending moments in OFA design when wall stiffness is high.

The retaining wall's variation confirms the previously mentioned tendency of OFA as design-relevant method if stiff components are dominating the entire system. Additionally, the soil strength is significant in evaluating the design-relevant approach (IFA or OFA). Reducing the soil strength parameters in IFA simulations can be design-relevant if the soil is close to the limit state. In such cases, the soil can carry comparatively less load and the bending moments of the retaining wall increase with IFA as design-relevant approach. If the soil has sufficient load-bearing capacity, the bending moments in IFA design will remain small and OFA becomes design-relevant due to factoring the characteristic effects of action.

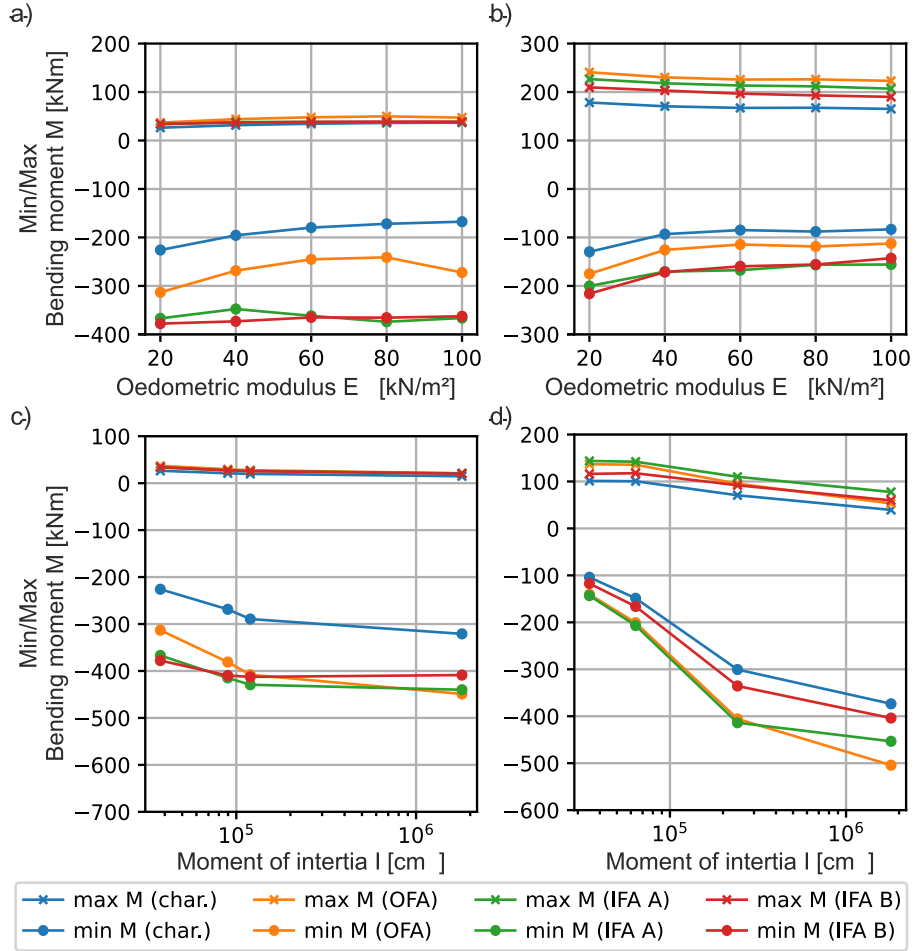


Figure 3 Maximum and minimum bending moments M for the varied values of the oedometric modulus E_{oed} (a and b) and moment of inertia I_y of the wall (c and d). System 1 left column and system 2 right column.

INFLUENCE OF THE SYSTEM STIFFNESS

The ratio χ , which compares the design bending moment M_d according to IFA (either variant A or B) with OFA, is defined to analyse the influencing factors according to Equation 3. A ratio $\chi > 1$ means that the relevant design results considering the bending moment of the retaining wall are based on the IFA calculation whereas for $\chi < 1$ they are based on the OFA simulation.

$$\chi = \frac{\max(|M_{d,IFA A}|; |M_{d,IFA B}|)}{|M_{d,OFA}|} \quad \text{Eq. 3}$$

In both excavations analysed, the system stiffness is calculated using the unit weight of the soil γ_s instead of the unit weight of water γ_w , as hydraulic conditions are not applied. In addition, the numerical models are analysed considering drained conditions, resulting in the conversion of undrained shear strength s_u using the approximation in Equation 4.

$$s_u \approx \tau = \frac{\sigma'_1 + \sigma'_3}{2} \cdot \sin \varphi' + c' \cdot \cos \varphi' \quad \text{Eq. 4}$$

Figures 4 and 5 show the ratio χ for the parameter variations carried out in section 4 for systems 1 (orange) and 2 (green) as a function of the respective system stiffnesses according to Clough et al [3] and Bryson and Zapata-Medina [6].

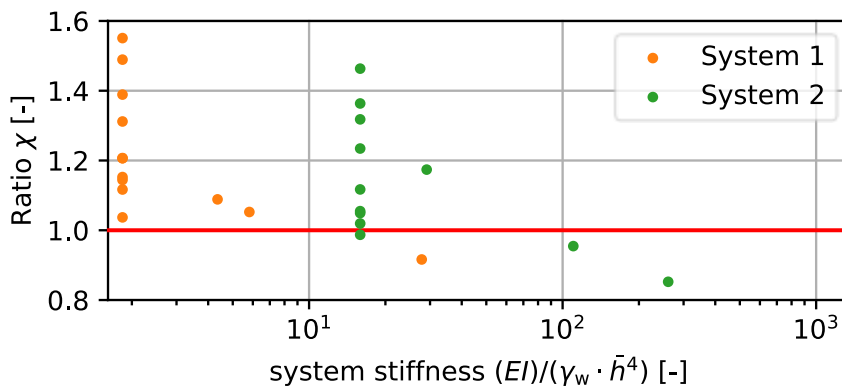


Figure 4 System stiffness using the varied parameters for system 1 and 2 in relation to the ratio χ .

The investigation of the system stiffness according to Clough in Figure 4 shows that a higher bending stiffness EI of the retaining wall leads to a stiffer system behaviour, with ratio $\chi < 1$ and OFA simulations becoming design-relevant. However, the variation of soil strength parameters (φ' and c') has not been considered, and the system stiffness is only partially identical, even though the values for the ratio χ vary. Although the influence of strength parameters on the design-relevant method (IFA or OFA) is relatively minor (see Figure 2), it is important not to ignore this aspect when considering the system stiffness.

Figure 5 illustrates the use of the more general approach by Bryson and Zapata-Medina [6] to consider the strength parameters. The parameter variations that determine the decisive design moment as a result of IFA are in a similar range ($R > 17.5$). Only system 2 shows an outlier, where OFA is identified as design-relevant method. In this case, the parameter variation $\varphi' = 30^\circ$ and $c' = 15 \text{ kN/m}^2$ is applied, indicating that the soil has a

comparatively high shear strength. The factor of safety $FoS = 1.878$ for the final state due to strength reduction is within an above-average safe range. Therefore, a construction other than the double braced excavation pit would be more economical and the system selected here is not optimal.

In principle, however, it can be stated that the relative stiffness ratio R according to Bryson and Zapata-Medina [6] allows a tendency to predict the design-relevant method (IFA or OFA) for the systems analysed here.

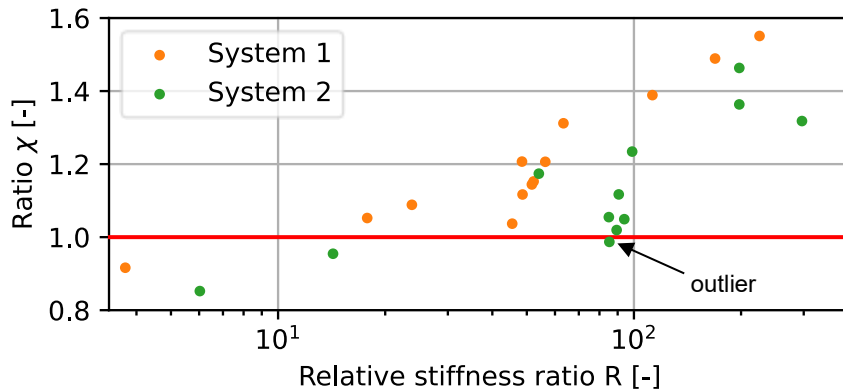


Figure 5 Relative stiffness ratio R using the varied parameters for system 1 and 2 in relation to the ratio χ .

CONCLUSION AND OUTLOOK

This paper analyses the numerical design methods IFA and OFA for single and double braced excavations. The study investigates the influence of various parameters on the bending moment of the wall and highlights the dependency on the design approach. The results indicate that OFA is more relevant for design when a stiff component dominates the entire system, as these components can absorb greater stress/load. If the soil is close to the ultimate limit state and the wall must absorb the necessary loads, a reduction in strength parameters by means of IFA design becomes relevant.

To predict the relevant design approach, the system stiffness according to Clough et al. [3] and the relative stiffness ratio R according to Bryson and Zapata-Medina [6] are applied to the two systems as a function of the varied parameters. It is shown that the relative stiffness ratio R can be used to determine whether IFA or OFA method is appropriate for design purposes. This allows for better use of numerical methods in early stages of excavation planning to analyze stability in ULS.

Future investigations will include synthetic systems that consider hydraulic conditions and layered soil. Furthermore, the investigation will encompass real excavation pits and systems from literature.

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ON THE LIQUID LIMIT OF SENSITIVE CLAY

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KEYWORDS

Quick clay, Kaolinite, Clay properties, Liquid limit, NaCl

ABSTRACT

Quick clays are ubiquitous materials in Sweden, Norway, and Canada. When disturbed, these sensitive clays transform into a liquid, losing all its strength, and cause natural disasters. Both mineralogy and salinity have a major impact on quick clay behaviour. Liquid limit is one of the determining properties for quick clays since the water content in this type of clay is usually higher than the liquid limit. In this study, two different types of commercial kaolinites, and natural Kärä clay were utilised to determine how mineralogy affects the liquid limit. In addition, a 1M NaCl solution was added to the clays to understand the impact of salinity on the liquid limit. The results showed that although the liquid limit for Kärä clay increases with an increase in salinity, adding NaCl solution to kaolinite causes a drop in the liquid limit. Therefore, kaolinite cannot be the determining mineral for the emerging liquid limit of sensitive clay.

INTRODUCTION

In Sweden, Norway, and Canada nearly all shallow landslides have taken place in regions with quick clay [1], such as Stenungsund landslide in September 2023. Quick clays are sensitive fine-grained sediments that can permanently transform into a liquid upon anthropogenic and environmental loading [2]. In Sweden, clay that has a sensitivity of 50 or above and a fully remoulded shear strength of less than 0.4 kPa is referred to as quick clay [3, 4]. These clays are found in regions that were originally covered by glaciers such as Canada, Alaska, Scandinavia, and northern Russia [3, 5].

The mechanisms in quick clay that govern the sudden liquefaction are mainly unknown. However, a number of factors have been reported to affect the sensitivity of clays, such as mineralogical composition, particle size distribution, concentration of salt in the pore fluid (marine sediments), pH of the pore fluid, and organic content [6].

The concentration of electrolytes in the water, as well as the type and amount of fine minerals, have a significant impact on clay properties such as liquid limit (LL). In quick clays, the amount of water content is usually much higher

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than the liquid limit [6]. Extensive research has been conducted to determine the effect of pore water salinity on the liquid limit of different clays. It has been shown that adding NaCl to illite-rich marine clay causes an increase in the amount of the LL [6]. Song et al. [7] studied the impact of NaCl on two different types of illite-rich coastal clay and concluded that the LL increases with an increase in the salinity of pore water. Palomino & Santamarina [8] demonstrated that the LL of Wilklay RP-2 kaolin decreases as NaCl concentration increases. Sridharan and Prakash [9] concluded that adding a 0.5M NaCl solution to Kundra clay (kaolinitic clay) causes a decrease in the LL. However, Di Maio et al. [10] stated that adding NaCl up to 6M has no significant impact on the LL of kaolin clay.

The studies referenced show that pore fluid chemistry has a profoundly different effect on the LL of clay depending on clay mineralogy. Quick clay, however, has a diverse mineralogical composition that have been reported to affect its sensitivity [11]. According to Cabrera and Smalley [12], high sensitivity develops when non-expanding inactive clay-sized particles of quartz and feldspar are present. On the contrary, the presence of high-swelling minerals such as smectite decreases the sensitivity of clay [13].

The influence of salinity on a mixture of different minerals has not been studied extensively. In this study, two different monogranular monomineral industrial kaolin clays, as well as natural quick clay, composed of multiple minerals, were analysed to better understand how salinity affects the liquid limit of sensitive clays. The macroscale tests included a series of fall cone test at different ionic strength. Following the idea of Cabrera and Smalley [12], the intent is to understand if the inactive kaolinite fraction could be the determining mineral for this sensitive clay behaviour.

MATERIALS AND METHODS

Soil samples

Natural clay samples were collected from the Chalmers soft soil test site in Kärna municipality (Gothenburg, Sweden) from a depth of 4.5 to 5.5 m. The natural water content for this clay was 80%. X-ray diffraction analysis was also conducted in ALS Scandinavia AB laboratory to determine the minerals in this natural clay. The result of XRD analysis is demonstrated in table 1. Moreover, Speswhite kaolin clay (produced by IMERYS France), and Acros Organics kaolin clay were tested. The average particle size for Speswhite is 1.3 μm and the particle size for Acros Organics is 1.8 μm . Therefore, the main difference between the two kaolinite clays is the particle charge, which is controlled by the particle size. The specific gravity (G_s) of both kaolin is 2.6 [14, 15].

Table 6. XRD results for natural Kärä clay.

Mineral	Result (%)
Quartz	51.3 ± 1.5
Albite	10.8 ± 1.0
Microcline	8.3 ± 1.0
Cordierite	0.2
Calcite	17.3 ± 1.0
Halit	1.8 ± 0.5
illite	6.4 ± 1.0
Kaolinite	3.9 ± 0.5

Salts

To determine the effect of salinity on the liquid limit, sodium chloride (NaCl) was used in this study. The salt was purchased from Sigma-Aldrich USA with a purity of $\geq 99.0\%$.

Method

Natural Kärä clay samples were dried, ground, and sieved. Soil passing through a 500 μm sieve was then used to determine the liquid limit using the fall cone method (BS 1377-2, [16]). Demineralised water was added gradually to the sieved soil so the average depth of cone penetration in the sample was in the range of 15 to 25 mm. After each penetration in this range, a small amount of the sample was dried for 24 hours at 110 °C, and the water content was calculated. At least four samples with different water content were considered for the determination of the LL.

In the next step, Kärä clay was washed until the electrical conductivity reached below 100 μS to ascertain that there was negligible amount of ions left in the pore fluid [8]. Both kaolin clays did not need to be washed since the original electrical conductivity was found to be negligible ($<40 \mu\text{S}$). The whole procedure was repeated with the washed Kärä clay, and the liquid limit was measured. Then, the impact of salt on the LL of each clay was investigated. To do so, the 1M salt solution, instead of demineralised water, was added gradually to the washed Kärä clay, Speswhite, and Acros Organics.

RESULTS AND DISCUSSION

The results of the fall cone test for each type of clay, as well as the liquid limit, are illustrated in Figure 1. A linear regression was used to determine the liquid limit.

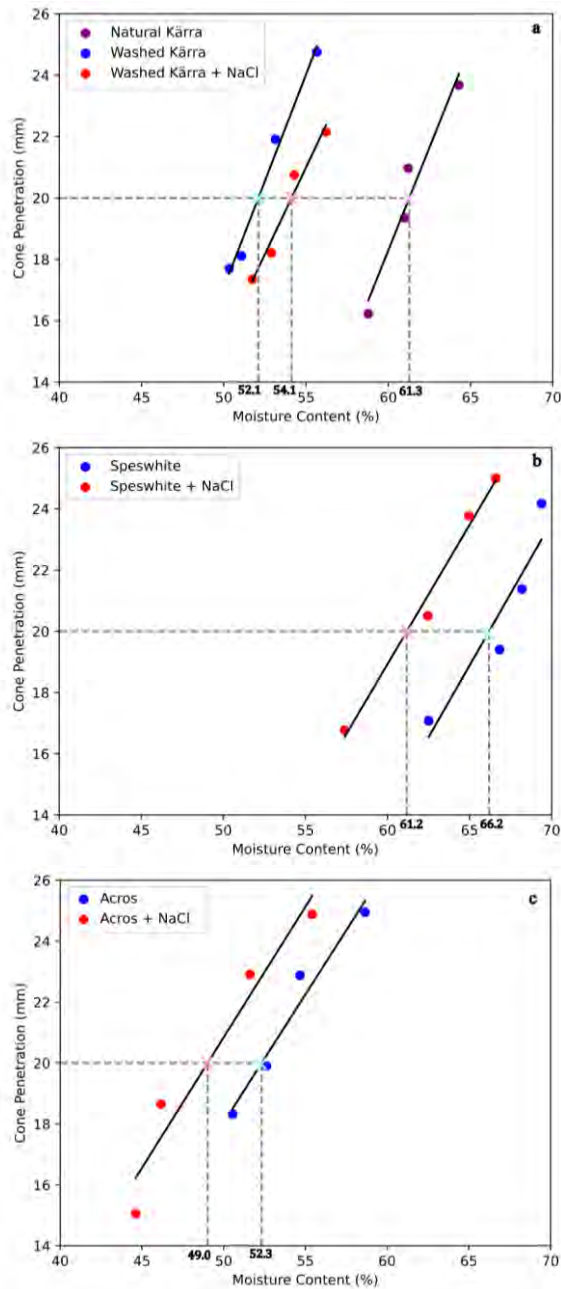


Figure 37. Cone penetration test results, and the liquid limit based on British standards for a) Kärä clay, b) Speswhite kaolinite, and c) Acros kaolinite.

Following the definition of British Standards, the liquid limit is equal to the water content when the cone penetration is 20 mm. The slope of the linear regression for cone penetration vs. moisture content can indicate the

sensitivity of clay to pore fluid. A steeper slope shows higher sensitivity to the salinity of the pore fluid [8]. As can be seen, Kärä clay, which is a quick clay, was more sensitive to salinity than both kaolinites. Speswhite kaolinite, also, had the highest LL at 66.2% while Acros kaolinite had the lowest at 52.3%.

Figure 1a demonstrates the liquid limit for three different conditions of Kärä clay. The liquid limit was higher (61.2%) in the natural condition and was reduced substantially when there was a negligible amount of ions in the pore fluid. After adding 1M NaCl solution to the washed clay, the liquid limit increased by 2% reaching 54.1%, which was still less than the liquid limit in the natural condition. The reduction in the liquid limit with salt leaching was reported in a few studies [6, 17]. This phenomenon, however, contradicts the diffuse double layer theory. Based on this theory, higher ion concentrations in the pore fluid reduces the thickness of the double layer, hence, the interparticle distance. Therefore, the capacity for holding water is reduced with higher salinity, resulting in a decrease in the liquid limit. The change in LL of sensitive clay at higher ion concentrations, however, can be attributed to changes in the soil fabric.

The liquid limits of Speswhite and Acros kaolinite are shown in Figure 1b and 1c respectively. After the addition of 1M NaCl solution, the liquid limit for Speswhite decreased by 5% while for Acros this reduction was approximately 3%. This is aligned with the result of Palomino & Santamarina [8] for Wilklay kaolinite. In contrast to the Kärä clay, the behaviour of both commercial kaolinites can be explained by the diffuse double layer theory. In other words, the reduction in the diffuse double layer thickness causes a decrease in interparticle repulsive forces, resulting in a smaller interparticle distance. Therefore, the liquid limit decreases at higher ion concentrations in the pore fluid.

Overall, the behaviour in terms of LL for Kärä clay contrasts with that of both kaolinites. While with an increase in salinity, the liquid limit increases for Kärä clay, it decreases for both kaolinites. Therefore, kaolinite does not seem to impact the LL of Kärä clay.

CONCLUSION

The liquid limits of a natural sensitive clay from Kärä, Speswhite kaolin clay, and Acros kaolin clay were determined in natural and saline environments to investigate the effect of salinity on clay behaviour. The main results are summarised below:

- The sensitivity to the ionic strength of the pore fluid for Kärä clay was higher than for Speswhite and Acros.

- An increase in salinity caused an increase in the liquid limit of Kärä clay while Speswhite and Acros kaolinite experienced a reduction in the liquid limit after a 1M NaCl solution was added.
- The change in LL of Kärä clay cannot be explained by the diffuse double layer theory but can be attributed to the change in soil fabric.
- Although the liquid limit of Speswhite is higher than Acros, the behaviour of both kaolinites agrees with the diffuse double layer theory.
- Kaolinite does not seem to impact the liquid limit of Kärä clay.

ACKNOWLEDGEMENT

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OPTIMIZATION OF GROUND INVESTIGATIONS WITH THE VALUE OF INFORMATION ANALYSIS

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KEYWORDS

Value of Information, Ground, Investigations, Optimization

ABSTRACT

This study investigates the problem of optimizing the number and locations of the ground investigations. Determining the number and locations of geotechnical investigations is central in reducing the ever-present uncertainties in soil properties. Good knowledge and understanding of the soil properties is central in ensuring that the design meets necessary safety and cost constraints. The selection of the numbers of ground investigations and their locations often varies from site to site due to lack of objective measures for determining the optimal ground investigation program. This results in a great variation of the numbers and locations of ground investigations in practice. This study aims to approach the problem from a probabilistic perspective and propose a more explicit and objective framework for the determination of the optimal numbers and locations of ground investigations. The proposed framework is based on a probabilistic geotechnical modelling approach and the Value of Information (VoI) analysis. The value of ground investigations results from the improved knowledge and the reduction of the potential negative consequences in case of an inadequate design. The potential benefits of conducting additional soil investigations are compared to the costs of ground investigations to determine the optimal number of samples and locations. The performance of the proposed algorithm is investigated on a slope stability problem.

INTRODUCTION

Geotechnical engineers are often faced with the task of making decisions under uncertainties resulting from lack of knowledge about ground properties, loads, soil behavior etc. In making these decisions, they are commonly required to balance the competing goals of safety and costs without having an explicit approach in dealing with the decision-making problem. This often results in a great variation in the conducted number of ground investigations and their locations across sites and projects. This study aims to examine the decision-making problem in a more explicit way by casting the problem in a

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probabilistic setting and applying the Value of Information (VoI) analysis to determine the optimal number and locations of ground investigations.

CONDITIONAL RANDOM FIELD

Two-dimensional conditional Gaussian random field is implemented to model uncertain soil properties and update them with measurements [e.g., 1].

Measurements of soil properties include measurement and interpretation errors. A discretized random field is denoted as $R(x, z)$ and specified with a vector of mean values $\mu = [\mu(x_1, z_1), \dots, \mu(x_m, z_m)]^T$ and a covariance matrix Σ with the following elements $\Sigma_{i,j} =$

$\sigma(x_i, z_i)\sigma(x_j, z_j)\rho((x_i, z_i), (x_j, z_j))$; $i, j = 1, \dots, m$, where σ is the standard deviation and ρ is the correlation coefficient modelled with an ellipsoidal autocorrelation model parameterized with horizontal and vertical correlation lengths, θ_x and θ_z , respectively [e.g., 1]. The joint probability density function (pdf) of R corresponds to a multivariate normal pdf, defined as follows:

$$R \sim f(\mathbf{r}) = N(\mathbf{r}; \boldsymbol{\mu}, \boldsymbol{\Sigma})$$

Consider that observations or measurements are available on s locations of the discretized domain, $\mathbf{o} = \{o_1, \dots, o_n\}$ with the covariance matrix Σ_o . The likelihood of the observations is:

$$\mathbf{o} \sim f(\mathbf{o}) = N(\mathbf{o}; \mathbf{H}\boldsymbol{\mu}, \Sigma_o)$$

where H is a linear function, specified as a matrix composed of s rows and m columns. Each row of the H matrix is assigned to an observation with the value of one on the column index corresponding to the index of the observation. The rest of the elements have zero values. The joint pdf of the random field conditioned on the observation is defined as [e.g., 1]:

$$[R|\mathbf{o}] \sim f(r|\mathbf{o}) = N([r|\mathbf{o}]; \mu_{r|\mathbf{o}}, \Sigma_{r|\mathbf{o}})$$

where

$$\mu_{r|\mathbf{o}} = \mu + \boldsymbol{\Sigma}H^T[\mathbf{H}\boldsymbol{\Sigma}H^T + \Sigma_o]^{-1}(\mathbf{o} - \mu\mathbf{H})$$

$$\Sigma_{r|\mathbf{o}} = \boldsymbol{\Sigma} - \boldsymbol{\Sigma}H^T[\mathbf{H}\boldsymbol{\Sigma}H^T + \Sigma_o]^{-1}\mathbf{H}\boldsymbol{\Sigma}$$

with $\mu_{r|\mathbf{o}}$ being the mean, and $\Sigma_{r|\mathbf{o}}$ the covariance matrix of the conditional random field.

VALUE OF INFORMATION ANALYSIS

The decision model presented in [2] is adopted in this report. A binary variable Y is introduced to indicate the performance of a system with $Y = 1$ denoting failure and $Y = 0$ denoting non-failure [e.g., 2]. In case that the

system is not able to satisfy the design requirements, a set of measures can be applied to improve the system response and satisfy the design requirements. Let d denote a decision variable with $d = 0$ denoting no measures and $d = 1, \dots, n_d$ the d^{th} decision for a measure improving the system performance, where n_d is the number of alternative engineering measures [e.g., 2]. Losses due to failure of a system can include economic, environmental, and health consequences and are denoted with c_f , while the costs of engineering measures are denoted as $c_e(d)$. Furthermore, the following restrictions are made on the costs, $c_e(0) = 0$ and $c_e(d) > 0$ of $d > 0$ [e.g., 2].

The decision-making process is based on a utility function, $u(Y, d)$, that encapsulates the major considerations including safety and costs of engineering measures. The following utility function is selected in this study [e.g., 2]:

$$u(Y, d) = \begin{cases} -c_e(d), & \text{if } Y = 0 \\ -(c_e(d) + c_f), & \text{if } Y = 1 \end{cases}$$

The likelihood of a geotechnical system experiencing safe performance or failure depends on the uncertain and spatially variable soil properties, R , which are modelled with the conditional random field approach introduced earlier. Let $f(\mathbf{r})$ denote the prior knowledge about uncertain soil properties available before conducting soil investigations. Realizations from the prior distribution are used as an input to geotechnical model to evaluate the system response in terms of a limit state described by the performance function $g(\mathbf{r}, d)$, for a given decision variable d . The prior failure probability is evaluated as follows [e.g., 2]:

$$p_{f,0} = P(Y = 1|d = 0) = \int_{g(\mathbf{r},d) \leq 0} f(\mathbf{r}) d\mathbf{r}$$

The expected benefit of taking a decision d can be calculated as the expectation of the utility function [e.g., 2]:

$$E_Y[u(Y, d)] = u(0, d)(1 - p_{f,0}) + u(1, d)p_{f,0} = -[c_e(d) + c_f p_{f,0}]$$

where $E_Y[\cdot]$ denotes expectation with respect to the distribution of Y [e.g., 2]. The expected benefit will be used as a basis for deciding among the alternatives, with the optimal decision being the one that maximized the expected outcome.

Initially, the expected utility is calculated based on prior information on the uncertain parameters $\mathbf{r}$. After performing field or laboratory investigations, the knowledge on the uncertain parameters can be updated with observations, $\mathbf{O}$, based on the Bayes' theorem. In this study both the prior and the likelihood are multivariate normal distributions. This allows for analytical solution for the Bayesian updating problem. The posterior failure probability is denoted as

$p_{f,d} = P(Y = 1|\mathbf{O}, d)$. The posterior failure probability can be used to update the expected utility as follows [e.g., 2]:

$$\begin{aligned} E_{Y|\mathbf{O}}[u(Y, d)] &= u(0, d)(1 - p_{f,d}(\mathbf{O})) + u(1, d)p_{f,d}(\mathbf{O}) \\ &= -[c_e(d) + c_f p_{f,d}(\mathbf{O})] \end{aligned}$$

where $E_{Y|\mathbf{O}}[\cdot]$ is the expectation with respect to the updated distribution of Y given $\mathbf{O}$. One may be able to update their decision based on the updated utility function. The difference between the maximum expected utility under the optimal decisions before and after collecting observations $\mathbf{O}$ is called Conditional Value of Information (CVOI) [e.g., 2]:

$$CVOI = \max_d E_{Y|\mathbf{O}}[u(Y, d)] - \max_d E_Y[u(Y, d)]$$

CVOI measures the improvement of the expected utility after collecting additional information from field or laboratory investigations. Prior to performing field or laboratory investigations, it is important to assess the effectiveness of the planned investigation program. Before the investigation program is conducted it is necessary to evaluate the uncertainty in observations $\mathbf{O}$ based on the prior knowledge. Let $f_{IP}(\mathbf{O})$ denote the pdf of $\mathbf{O}$ from a given Investigation Program (IP) that is inferred from the prior knowledge. Mathematically, it can be inferred from the total probability theorem as follows [e.g., 2]:

$$f_{IP}(\mathbf{O}) = \int f_{IP}(\mathbf{O}|\mathbf{r})f(\mathbf{r})d\mathbf{r}$$

The potential benefit of performing the IP is evaluated by conducting Value of Information (VoI) analysis. VoI is defined as the average of CVOI considering all possible values of $\mathbf{O}$ [e.g., 2]:

$$VoI_{IP} = \int CVOI_{IP}(\mathbf{O})f_{IP}(\mathbf{O})d\mathbf{O}$$

Integrating the earlier expression into the value of information analysis for the considered IP, VoI_{IP} , the following expression can be obtained [e.g., 2]:

$$\begin{aligned} VoI_{IP} &= \int \max_d \{ -[c_e(d) + c_f p_{f,d}(\mathbf{O})] \} f_{IP}(\mathbf{O}) d\mathbf{O} - \max_d \{ -[c_e(d) \\ &\quad + c_f p_{f,0}] \} \end{aligned}$$

RESULTS AND DISCUSSION

Geometry of the slope stability problem is shown in Figure 1. The slope is 20 m high with the slope inclination of 1:2. Slope stability is analyzed with the Janbu's direct method, $F_S = \bar{s}_u \frac{N_0}{P_d} + \varepsilon_M$, where $\bar{s}_u$ is the average shear strength along the failure surface, N_0 is the stability coefficient, and $P_d = \gamma \cdot H$, where $\gamma = 19 \text{ kN/m}^3$ and $H = 20 \text{ m}$, and $\varepsilon_M \sim N(0, 0.05)$ is the assumed

model error. Reliability analysis is conducted with the Monte Carlo method to evaluate the probability that the factor of safety is lower than one, $P_F = P(F_S \leq 1) = P\left(\bar{s}_u \frac{N_0}{p_d} + \varepsilon_M \leq 1\right)$. F_S is calculated by considering 12 potential failure surfaces ending in the proximity of the slope toe. The surface with the lowest factor of safety is selected as the critical failure surface.

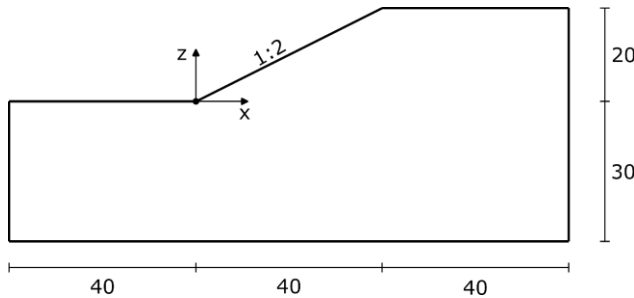


Figure 1 Slope stability problem geometry.

Prior knowledge on undrained shear strength is specified with a normal random field defined with a linearly increasing trend, $\mu_{s_u} = 40 + 1.8z$, constant standard deviation, $\sigma_{s_u} = 8 \text{ kPa}$, and correlation lengths of $\theta_x = \theta_z = 50 \text{ m}$. The actual and, initially unknown, soil properties are generated from a realization of a normal random field defined with a linearly increasing trend, $\mu_{s_u} = 40 + 1.8z$, constant standard deviation, $\sigma_{s_u} = 1.2 \text{ kPa}$, and correlation lengths of $\theta_x = \theta_z = 50 \text{ m}$. VoI analysis is conducted based on a utility function introduced earlier with the failure cost being $c_f = 1$ and the cost of soil investigation being $c_e = 0.01$. VoI analysis was conducted to simulate the procedure for performing 10 soil investigations.

Initially, prior analysis was conducted to assess the safety of the slope based on the prior knowledge. Initial failure probability was estimated to $P_{F0} = 0.119$. VoI analysis was the conducted to simulate 10 soil investigations. Prior to performing soil investigation, a trial analysis was conducted. In the trial analysis, performing soil investigations was simulated at all the points in Figure 2, while accounting for the inherent variability and measurement errors. VoI values after the trial analysis for the first investigation are shown in Figure 3. Positive values show that that there is a benefit in performing investigation at that location. The optimal location is identified as the location that maximizes the expected VoI. Soil investigation is then performed at the optimal location, and the measured value with the addition of the measurement error, $\epsilon \sim N(0,0.05)$, is used to update the mean and standard deviation of the conditional random field.

Figure 4 shows the optimal locations for the first five investigations and the effects of investigations on the uncertainties in soil properties. Performing

ground investigations reduces the original standard deviation of 8 kPa to the inherent variability and measurement error. The reduction of uncertainty does not only affect the location of the measurement, but also the neighboring areas due to spatial similarity, simulated with the autocorrelation function. Figure 3 shows also that the VoI algorithm was able to independently identify points along the potential failure surfaces as the optimal points for performing investigations.

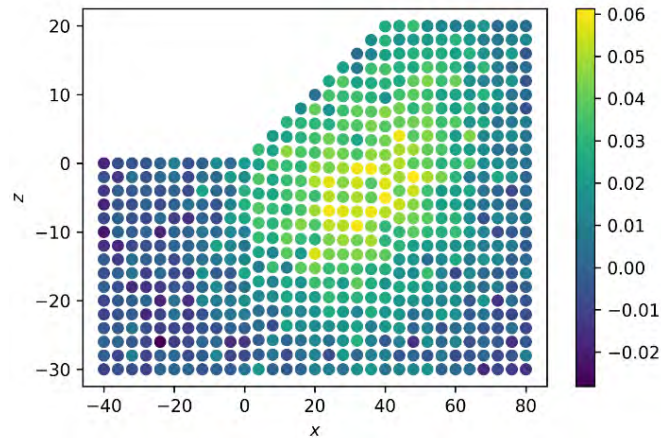


Figure 2 VoI values after trial analysis before for the first investigation.

Figure 4 compares the VoI of performing the 10 investigations and the costs of investigations. Although VoI is higher than the cost of investigations for all the 10 soil investigations, one can see that after the first 4 investigations, the increase in the VoI is not significant anymore. The gains in VoI from performing additional soil investigations are diminishing, which can be also seen in Figure 5 that shows the VoI subtracted for the cost of investigations. Thus results in 4 ground investigations as the optimal number of ground investigations.

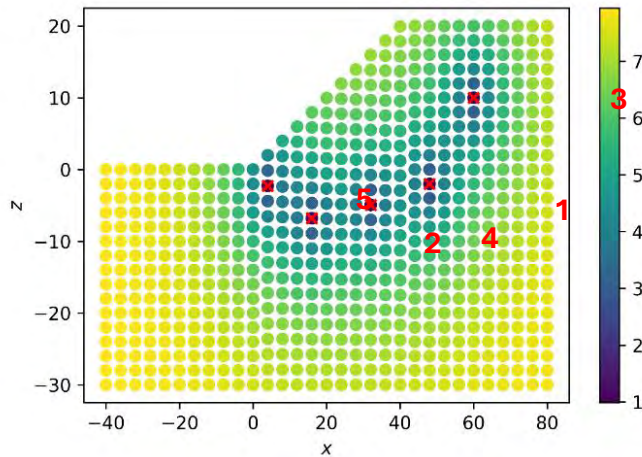


Figure 3 Effects of measurement on soil uncertainties and the optimal locations of the first five ground investigations.

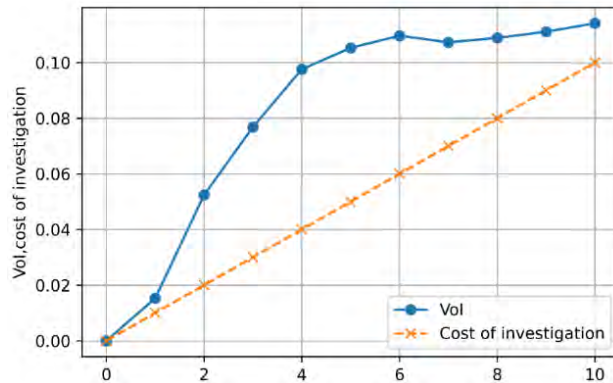


Figure 4 Comparison of VoI and costs of ground investigations.

Additional results were conducted for a range of correlation lengths, with the results presented in Table 1. Higher values of correlation lengths have a positive effect on VoI as a measurement leads to reduction in uncertainties in a larger area surrounding the measurement location. This resulted in a larger number of soil investigations being more optimal.

Conversely, low values of correlation lengths resulted in low VoI values because low correlation length resulted in the impact of the measurement being limited to a small zone around the soil measurement. Additionally, due to relatively coarse discretization of the domain in the slope stability problem, the estimates of factor of safety did not benefit substantially from those measurements. Implementation of more advanced slope stability models with a finer discretization and non-circular failure surfaces is needed to better assess the effects of small correlation lengths on the optimal number of soil

investigations. Determining realistic values of correlation lengths is important for assessing the optimal number of samples based on the implemented VoI methodology.

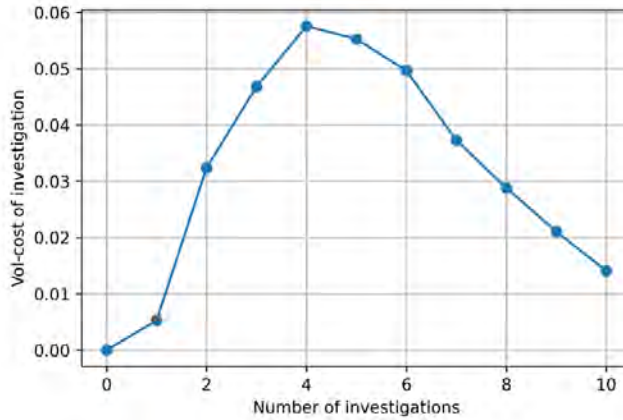


Figure 5 VoI subtracted for the costs of ground investigations.

Table 1: Optimal numbers of ground investigations for varying correlation lengths.

θ_x	θ_z	N_{Opt}
100	2	2
100	10	4
10	10	0
50	50	4
100	100	3
1000	10	3
50	20	4
100	2	2

CONCLUSIONS

The report demonstrated the use of Value of Information (VoI) analysis in planning soil investigations and determining the optimal number of soil investigations. VoI analysis was combined with a conditional random field model that allowed integration of soil investigations, with the addition of measurement and interpretation errors, in the modelling of uncertain soil properties.

The results show that VoI analysis can manage the trade-off between cost of investigation and potentially adverse consequences of geotechnical system failure to determine the optimal number of soil investigations and their

locations. The results showed that the optimal number of investigations is significantly affected by the correlation length. Higher correlation length allows a measurement to have a greater impact on the reduction of uncertainties in the surrounding soil zone, which results in a greater VoI and bigger increase in safety in comparison to situations with lower correlation lengths.

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PEAT - STRENGTH AND STABILITY OF EXISTING RAILWAY EMBANKMENTS

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KEYWORDS

Peat, strength, field, laboratory, testing, anisotropy, undrained, stability calculations, embankments, railway

ABSTRACT

In the paper some selected results are presented from an on-going research project about strength of fibrous peat below embankments and stability of existing embankments on peat. Field and laboratory tests, and stability calculations of the railway embankment at site Missenträsk, are presented. The field case high light some important aspects when dealing with the task of calculating and determining the stability of an existing embankment on peat. The peat below the embankment shows significant undrained strength anisotropy and strength increase with time, and considering this clearly affects the calculated factor of safety of the embankment. A “new” way is compared with the “traditional” way of considering strength for stability calculations of embankments. The developed field sounding method CYPT shows the variation of strength with depth both in the peat below the embankment and in the peat at the bog beside the embankment.

INTRODUCTION

Experiences in Sweden and e.g. the Netherlands often show low calculated safety factors of existing embankments on fibrous peat (low and medium decomposed peat). However, failure of embankments on fibrous peat rarely occurs. Fibrous peat most likely has a higher strength than what we measure and evaluate with present experimental methods [1], and thus there is a need for new knowledge and methods for strength determination.

In the paper is presented some results from an on-going research project conducted at SGI, in cooperation with the Swedish Transport Administration (Trafikverket), regarding undrained shear strength of fibrous peat and stability

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of embankments [2]. The study focuses on strength increase with time (long term) in vertical/active direction below existing embankments. Field and laboratory tests are conducted to study strength properties and a new peat probe and a new peat sampler have been developed and used in field in the project. Three sites with old and not any more used railway embankments overlaying fibrous peat, have been investigated in situ with the new developed peat probe called CYPT and for comparison also with the tests T-bar, CPT and field vane. The compressed peat below the embankment as well as the peat of the bog at the side of the embankment have been investigated. In the laboratory, triaxial tests, direct simple shear tests and different compression tests have been performed. The main objective of the project is to develop and bring forth a new working procedure to determine undrained shear strength of fibrous peat for stability calculations of embankments.

In this paper some selected results from field and laboratory tests, and stability calculations of the railway embankment at site Missenträsk, are presented.

TEST SITE

The test site at Missenträsk is a part of the no longer used railway between the villages of Jörn and Arvidsjaur in the north of Sweden, figure 1. The railway was built 95-100 years ago and traffic stopped around the year 1990.

The embankment at the site is 1,5 m thick and lays directly on the peat. The compressed peat layer below the embankment is about 2,0 m and overlays a till. At the bog at the side of the embankment the (natural, not compressed) peat layer is about 2,5-2,8 m thick.

The peat at the bog is generally classified as H2 according to the von Post scale, with the water content varying between 600-900 %. The peat below the embankment is classified as H2 and has a water content varying between 450-650 %.



Figure 1 Investigations of peat at the (no longer used) railway at the test site Missenträsk.

FIELD TESTS

Development of new probe CYPT and interpretation of results

In the research project a new testing device for sounding in peat in situ has been developed, called CYPT (CYlindrical Penetration Test), [2]. Actually, five different variants of CYPT have been developed: one large with diameter 140 mm (research probe) and a small with diameter 70 mm (production probe), and with possibilities to either have the end surface (the point surface) attached with pins or being smooth (to evaluate effects of interface friction), and possibilities to have a single plate or double plates end (140 mm diameter). In figure 2 is shown two variants, CYPT 140 mm diameter with pins and CYPT 140 mm diameter double plates (inner and outer plate). The CYPT probe is connected to a standard CPT system and corresponding point pressure, sleeve friction, and pore pressure are registered, and the same penetration rate as in standard CPT was used. Using the double plate CYPT, the pressure on the outer plate is measured through the sleeve friction sensor and the inner plate through point pressure sensor (both plates have the same area). The idea of introducing two plates, and thus separating the inner and outer pressure, is to study the possible effects on the sounding resistance of tension forces in the fibrous peat.

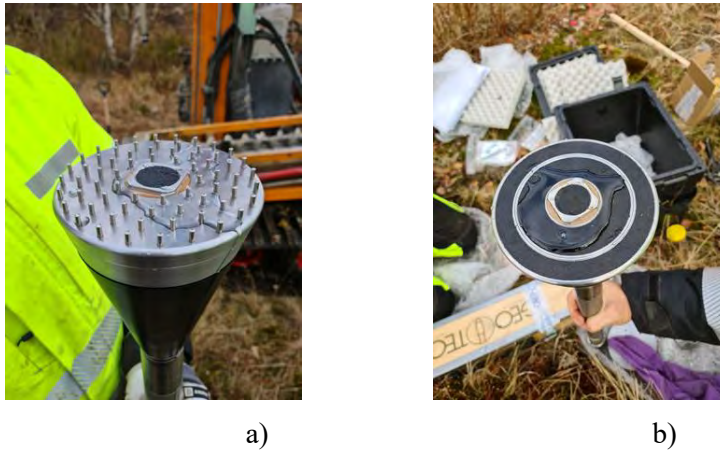


Figure 2 Two variants of the, in the research project, developed field sounding device CYPT, a) 140 mm diameter point surface attached with pins and a filter in the middle to measure the pore pressure, b) double plate (outer, inner) 140 mm diameter point surface with filter in the middle, (CYPT held upside down in the pictures).

In figure 3 some selected results from testing with CYPT in peat below embankment and peat beside embankment at the bog at site Missenträsk are presented. The variation of point resistance with depth is similar below the embankment and at the bog, there is a maximum point resistance after about 0,2-0,3 m and after that the resistance decrease with depth. The point resistance is, however, much higher below the embankment compared to at the bog, indicating a much higher strength of the peat below the embankment. Furthermore, as expected the point resistance decreases with depth and thus the strength of the peat below the embankment decreases with depth (significant reduction of point resistance at 1,1-1,2 m depth of the peat layer below the embankment).

In the research project the results of the CYPT have been compared with results from the field methods T-bar and CPT, and all three sounding methods show in general the same variation with depth of point resistance, both for (natural, not loaded) peat at the bog and for peat below the embankment. This suggests that the data received from the CYPT correspond to the variation of strength with depth. In the on-going research project the possibilities of using results from CYPT to also evaluate undrained shear strength of peat is investigated. Comparisons with and possible correlations to triaxial test data are searched for.

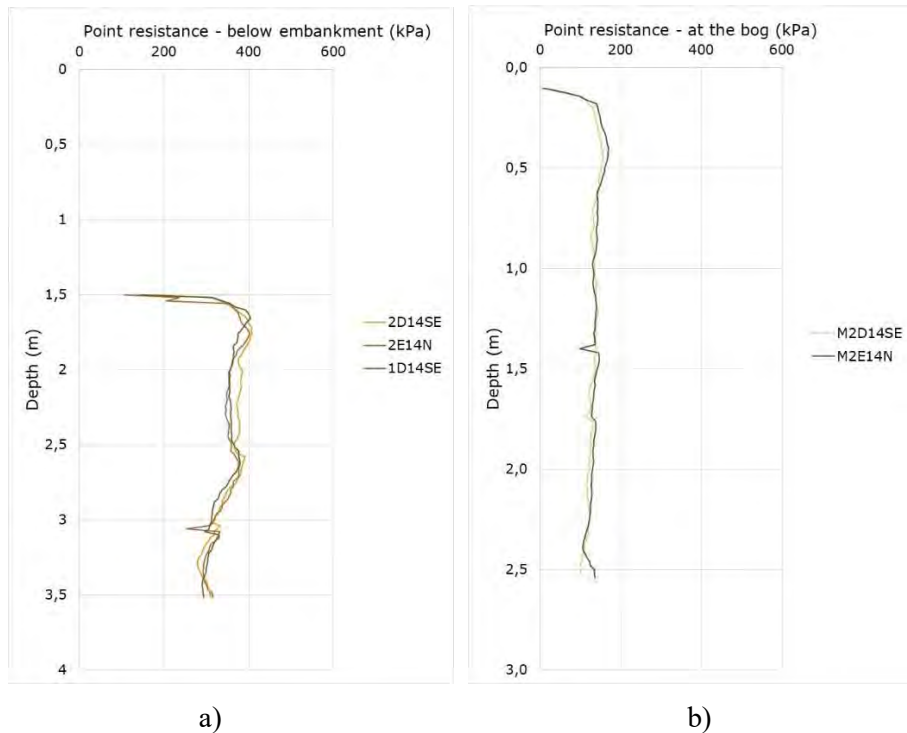


Figure 3 Field testing with the new sounding method, CYPT, point resistance versus depth (note different depth scales and depth references in figure a and b), a) in peat below the embankment (starting 1,5 m below the upper surface of the embankment), b) in peat at the bog beside the embankment.

LABORATORY TESTS

Mechanical behaviour of peat and strength evaluation

Triaxial tests have been conducted to study the undrained behaviour and to evaluate strength of the peat. In figure 4 the stress-strain, pore pressure change and effective stress path responses from some of the triaxial compression (active) undrained tests conducted on peat at site Missenträsk are presented. Samples of peat taken from below the embankment (i.e. peat in situ loaded and compressed by the embankment during many years) and from the bog beside the embankment (i.e. peat natural in situ not loaded) have been investigated.

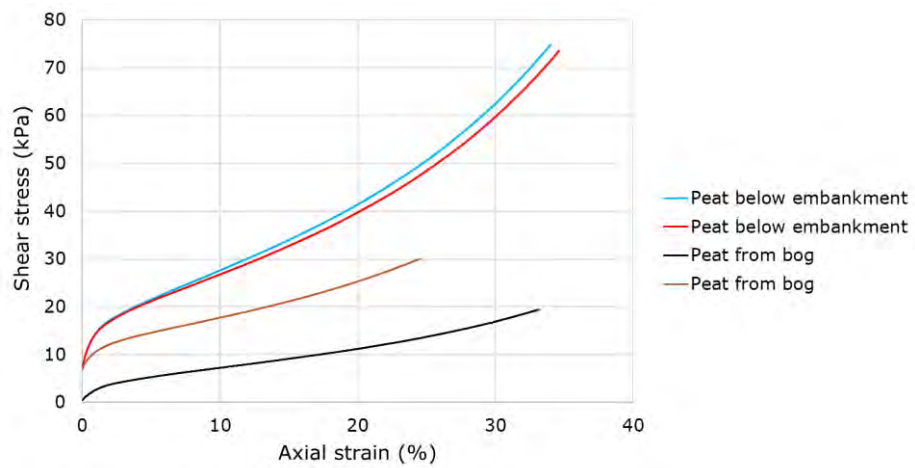
For the samples taken below the embankment, at small axial strains, about 0.5 to 1.5 %, there is a bend in the shear stress – axial strain curve, figure 4 a. The high initial stiffness significantly decreases after the bend and between about 2-3 % to about 15 % axial strain the stiffness is close to constant, i.e. there is a almost a linear stress-strain relation. For axial strains larger than about 15 %

the stiffness increases during the rest of the shear, i.e. the curve bends (weakly) upwards. The stress-strain response of samples from the bog is qualitatively the same as for the samples from below the embankment. The differences being lower stiffness and lower shear stress for the samples from the bog. The peat samples from below the embankment, and one of the peat samples from the bog, were consolidated (stresses applied before the shear phase shown in figure 4) in the triaxial test for a pressure corresponding to the (long term) load of the embankment (i.e. about 28 kPa effective vertical stress) and one of the samples from the bog consolidated for low stresses (here 7 kPa effective vertical stress).

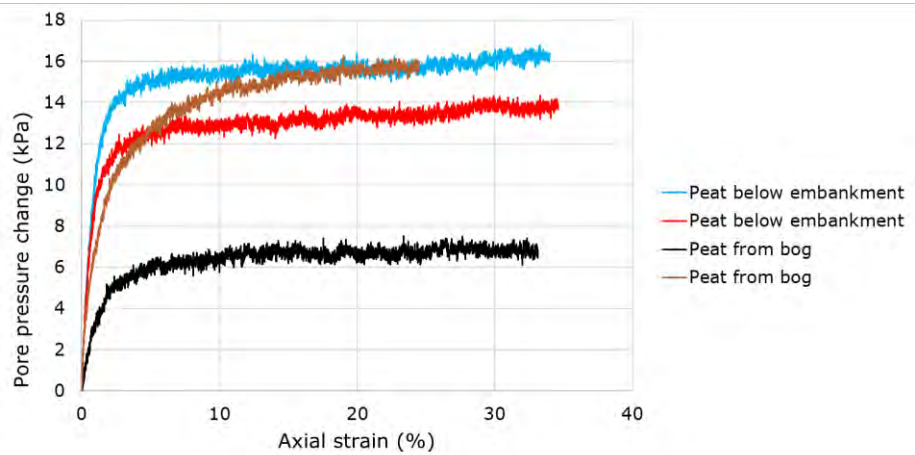
The effective stress paths first go upwards to the left, i.e. the mean effective stress decrease during an increase in shear stress, after which the stress paths turn upwards to the right, i.e. the mean effective stress increase and the shear increase during the rest of the shear, figure 4 c. During the first part of the shear there is a high pore pressure increase, which significantly contributes to the stress path going to the left, after which the pore pressure increase is small, figure 4 b. The response is principally the same for samples from below the embankment as from the bog.

In the triaxial compression tests on peat from both below the embankment and from the bog the shear stress increase with increasing strain, i.e. no peak value of shear stress is achieved. This strain-hardening behaviour is typical of most fibrous peat independent if drained or undrained triaxial tests are conducted [3]. Strengths used as input for stability calculations of the embankment in Missenträsk (Chapter 5) has been chosen to be evaluated as the shear stress at 15 % axial strain in the triaxial tests (and at the shear stress at 0,225 radians in the direct shear tests).

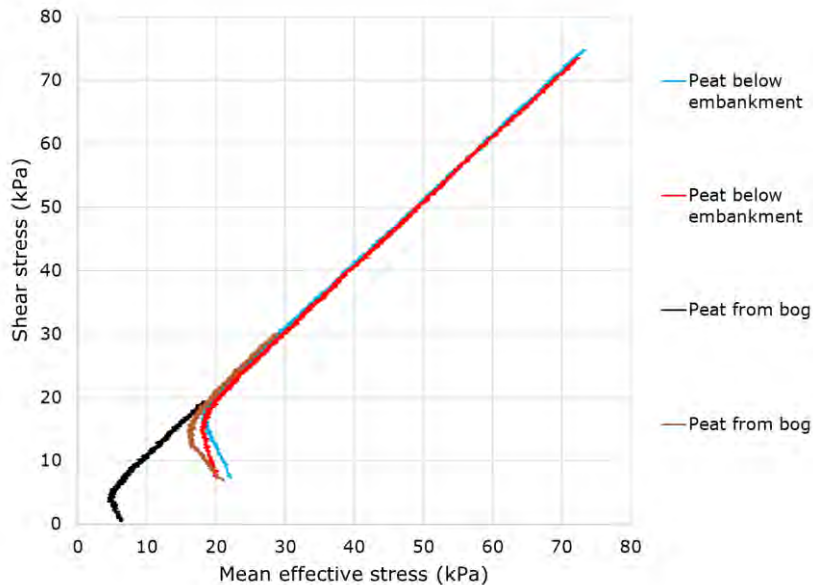
Triaxial extension (passive) undrained test and undrained direct simple shear tests have also been conducted on peat samples from the site Missenträsk (not showed in this paper).



a)



b)



c)

Figure 4 Triaxial undrained compression (active) tests conducted on peat from below embankment and from the bog, test site Missenträsk, a) shear stress (axial – radial stress) versus axial strain, b) pore pressure change versus axial strain, c) effective stress paths with shear stress (axial stress – radial stress) versus mean effective stress ((effective axial stress + effective radial stress)/2).

STABILITY CALCULATIONS

Calculations of embankment stability according to “new” and “traditional” methodology

For the studied section of the railway embankment at Missenträsk stability calculations have been performed. Stability has been studied for two different ways of applying the undrained shear strength. In the first case, “new” methodology, suggested in the research project, strength anisotropy and strength increase with time of peat below embankment is considered. In the second case, “traditional” methodology, strength anisotropy is not considered, and strength is based on investigations of peat at the side of the embankment. The stability calculations are used mainly to show the differences in calculated safety factor comparing the “new” and the “traditional” way. The calculations do not claim to e.g. capture a true failure mechanism.

In the calculations undrained conditions in the peat are assumed and thus the strength of the peat can be described by undrained shear strength. Here is

presented results from using the traffic load 32 kN/m^2 , refers to soil conditions of the embankment after about 95 years of construction. The calculations are conducted using Morgenstern-Price's method of slices and according to and using the stability calculation software Slope/w (GeoStudio version 2021.4) [4].

For the “new” methodology considering strength anisotropy, the undrained shear strength values of the peat below and beside the embankment are based on undrained triaxial tests (active, passive) and undrained direct simple shear tests conducted on samples taken from below the embankment and from the bog beside the embankment. The variation of strength with depth, for both “new” and “traditional” methodology, is based on in situ tests using the new probe CYPT (and T-bar and CPT).

In figure 5 is presented the soil profile, chosen parameter values and the failure surface corresponding to the calculated lowest factor of safety for the “new” calculation case. The peat is divided into an upper and lower layer below the embankment, below the slope and beside the embankment. Since the undrained shear strength decrease with depth below the embankment the peat is divided into an upper and lower layer with different strengths. The side boundaries of the layers are assumed to have a slope of about 3:1, and this is based on the deformation pattern obtained in previous laboratory model tests on fibrous peat conducted at SGI [3].

Based on the results of the undrained triaxial tests (active, passive) and undrained direct simple shear tests on peat from below the embankment, the active strength was set to 1.6, and the passive 0.7 times the direct shear strength. The same strength anisotropy relations were assumed for the peat beside the embankment.

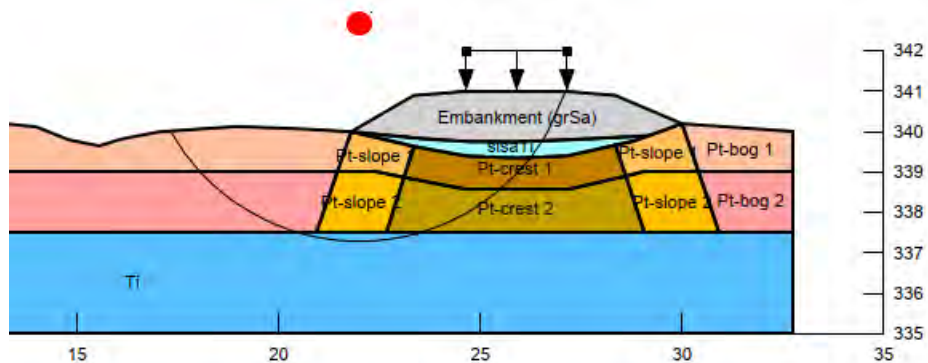
The evaluated undrained shear strength from the direct simple shear tests on peat from below embankment is 21 kPa, which is seen in figure 5 as input for the upper layer. Based on tests with CYPT (and other sounding methods) it is seen that the resistance (and undrained shear strength) is about 0.7 of that in the upper layer, giving direct shear strength 15 kPa in the lower layer, see figure 5. Thus, the strength of the upper layer of peat is in this case based on laboratory test results and the lower layer of peat based on the field sounding method results, by correcting the strength results from the upper layer. The values of peat below the embankment slope are assumed (interpolated) based on the values from below embankment and from beside of the embankment.

In figure 6 is presented the soil profile, chosen parameter values and the failure surface corresponding to the calculated lowest factor of safety for the “traditional” calculation case, where isotropic (same in all directions) strength conditions are assumed. The soil layers are similar but simplified as compared to “new” case (figure 5). The undrained strength (direct shear) is 5 kPa for the peat beside the embankment. In the upper layer below the embankment the

strength is 13 kPa, evaluated from an undrained direct simple shear test conducted on peat from beside embankment and consolidated in the direct shear test for the load of the embankment, i.e. the load that is acting on the peat below the embankment. The strength of the lower layer is set to 0,7 times the upper layer, yielding 9 kPa.

Color	Name	Slope Stability Material Model	Unit Weight (kN/m ³)	Total Cohesion (kPa)	Anisotropic Strength F _n	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Embankment (grSa)	Mohr-Coulomb	19			0	37
	Pt-bog 1	Undrained (Phi=0)	10,5	6	Anisotropy		
	Pt-bog 2	Undrained (Phi=0)	10,5	4	Anisotropy		
	Pt-crest 1	Undrained (Phi=0)	11,5	21	Anisotropy		
	Pt-crest 2	Undrained (Phi=0)	11,5	15	Anisotropy		
	Pt-slope 1	Undrained (Phi=0)	11	13	Anisotropy		
	Pt-slope 2	Undrained (Phi=0)	11	9	Anisotropy		
	sisaTi	Mohr-Coulomb	20			0	34
	Ti	Mohr-Coulomb	20			0	40

a)



b)

Figure 5 Calculations of stability of railway embankment at Missenträsk according to “new” methodology, a) Parameter values chosen for stability calculations b) Soil model and the slip surface with lowest calculated factor of safety for a traffic load of 32 kN/m².

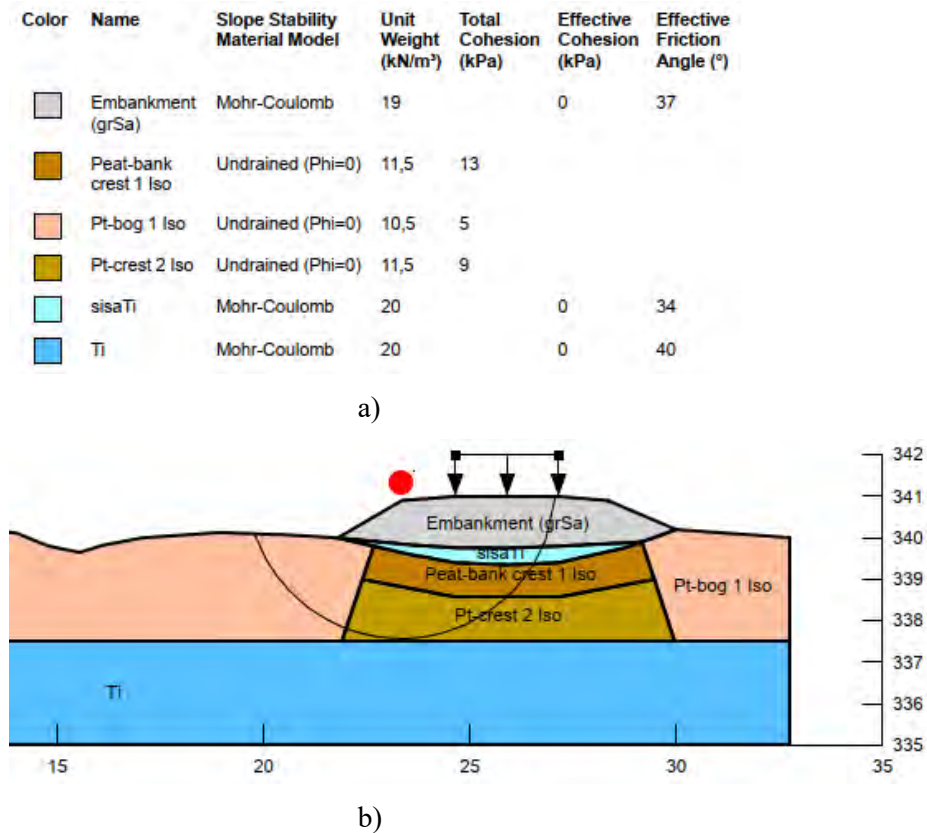


Figure 6 Calculations of stability of railway embankment at Missenträsk according to “traditional” methodology, a) Parameter values chosen for stability calculations b) Soil model and the slip surface with lowest calculated factor of safety for a traffic load of 32 kN/m².

The lowest calculated factor of safety for undrained failure is 1,48 when considering strength anisotropy (“new” methodology) and 0,91 when assuming isotropic conditions (“traditional” methodology).

The failure surface (for lowest factor of safety) goes down slightly in the underlying till layer for the case considering anisotropy, because below the slope toe the effective stresses are very low and thus the shear strength low in the uppermost part of the till. However, in the isotropic case, the failure surface only touches the till layer because the undrained shear strength is lower in the peat than the drained shear strength in the uppermost part of the till.

CONCLUSIONS

The field case showed in this paper high light some important aspects when dealing with the task of calculating and determining the stability of an existing embankment on peat.

When considering strength anisotropy and strength increase with time and depth, of peat below an existing embankment according to the suggested “new” methodology, the calculated factor of safety becomes much higher than when conducting “traditional” analyses and calculations.

To not consider strength anisotropy and underestimating strength increase below an embankment, is probably a main explanation behind that existing embankments sometimes show calculated factors of safety that are low or below one, without that any failure having occurred.

Fibrous peat is in general a strain-hardening material, meaning that the peat may take more and more load without going to failure. This is something that should be considered when calculating stability and bearing capacity of embankments on peat, e.g. when choosing failure criteria for evaluation of experimental data or a constitutive model to simulate the behaviour of peat.

The field sounding method, CYPT, developed in the research project seems to be able to capture the variation of strength of peat with depth, both below an embankment and at a bog.

ACKNOWLEDGEMENT

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PHYSICS-INFORMED NEURAL NETWORKS WITH CURRICULUM TRAINING FOR POROELASTIC FLOW AND DEFORMATION PROCESSES

Yared W. Bekele¹

KEYWORDS

PINNs, curriculum training, flow, deformation, poroelasticity

ABSTRACT

Physics-Informed Neural Networks (PINNs) have emerged as a highly active research topic across multiple disciplines in science and engineering, including computational geomechanics. PINNs offer a promising approach in different applications where faster, near real-time or real-time numerical prediction is required. Examples of such areas in geomechanics include geotechnical design optimization, digital twins of geostructures and stability prediction of monitored slopes. But there remain challenges in training of PINNs, especially for problems with high spatial and temporal complexity. In this paper, we study how the training of PINNs can be improved by using an idealized poroelasticity problem as a demonstration example. A curriculum training strategy is employed where the PINN model is trained gradually by dividing the training data into intervals along the temporal dimension. We find that the PINN model with curriculum training takes nearly half the time required for training compared to conventional training over the whole solution domain. For the particular example here, the quality of the predicted solution was found to be good in both training approaches, but it is anticipated that the curriculum training approach has the potential to offer a better prediction capability for more complex problems, a subject for further research.

INTRODUCTION

Physics-Informed Neural Networks (PINNs) [1] have emerged as a highly active research topic across multiple disciplines in science and engineering. PINNs embed governing equations, often in the form of partial differential equations, directly into the loss function of a neural network. This integration enables PINNs to learn solutions to complex scientific and engineering problems, even where data availability might be sparse. PINNs have been investigated in various disciplines including computational fluid dynamics [2], [3], [4] to model turbulent flows and fluid-structure interaction, material

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science [5], [6] to predict material properties and optimize design, biomechanics and biomedicine [7], [8] to simulate physiological processes and even in finance and economics [9], [10] for modelling economic systems. PINNs have also been studied in computational geomechanics [11], [12], [13], [14] to model complex physical processes in geological media like soils and rocks.

Despite the promise offered by PINNs for various applications, challenges remain in their training process and generalization capability. Some of these challenges include striking a balance between data fidelity and physical compliance to avoid overfitting, gradient vanishing/exploding during automatic differentiation in case of complex equations, optimization challenges because of the added constraints from physical laws, high computational cost and poor prediction capability for complex systems governed by highly nonlinear/coupled partial differential equations. Various researchers have attempted to address the challenges in the training of PINNs in different ways [15], [16], [17], [18]. Curriculum regularization and curriculum training are promising approaches suggested by the cited researchers to improve the training and prediction capability of PINNs. Curriculum regularization and curriculum training, though sounding similar, are conceptually distinct strategies used to enhance the training of PINNs. Curriculum regularization in PINNs refers to a method where the training process gradually introduces complexity in terms of the physics constraints applied to the model. Unlike standard training, which enforces all the physical laws from the start, curriculum regularization starts with simpler, perhaps less stringent, versions of these laws or conditions, and progressively moves towards the full, detailed physical models. This approach helps in managing the training complexity and avoids overwhelming the model with highly complex physical constraints at the initial stages, potentially leading to better convergence and stability in training. Curriculum training, on the other hand, involves gradually increasing the difficulty of the training dataset itself, rather than the complexity of the regularization or constraints. The idea is inspired by human learning, where we often start with simple concepts and progressively tackle more complex problems. In the context of neural networks, this might mean starting with easier, perhaps clearer or less noisy data, and progressively introducing more challenging or complex data as the training progresses. The latter i.e. curriculum training is the focus of this paper.

This paper focuses specifically on the application of curriculum-based training to coupled flow and deformation problems in poroelastic media. By doing so, it aims to contribute to the body of knowledge by suggesting a pathway for the development of more accurate and reliable computational models in the field of geomechanical simulations. The paper is structured as follows. We first present the governing equations of poroelasticity as they are

commonly introduced in computational geomechanics literature. The PINN model architecture and loss metrics are then defined and presented. An idealized numerical example is then presented wherein the analytical solution and the curriculum training strategy are presented in detail. Results are then presented that compare PINN models trained with and without the curriculum training strategy. Concluding remarks are then presented highlighting the advantages of curriculum training with a mention of points for further research.

GOVERNING EQUATIONS OF POROELASTICITY

The governing equations of poroelasticity are a combination of the overall mass balance equation, the equilibrium or linear momentum balance equation and the linear elastic constitutive equations for stress-strain relationships.

Equilibrium equation

The equilibrium equation, also called the linear momentum balance equation, for a two-phase (solid matrix and fluid) saturated porous medium under isothermal conditions can in a general form be written as

$$\nabla \cdot \bar{\sigma} + \rho \bar{b} = \mathbf{0} \quad (19)$$

where $\bar{\sigma}$ represents the total stress tensor in the porous medium, ρ is the overall density of the porous medium and $\bar{b}$ represents the body forces. For a porous medium, it is beneficial to introduce the concept of effective stress, the component of the stress carried by the solid matrix. The effective stress tensor $\bar{\sigma}'$ as a function of the total stress $\bar{\sigma}$ and the pore fluid pressure $\bar{p}$ is given by

$$\bar{\sigma}' = \bar{\sigma} - \bar{p} \mathbf{I} \quad (20)$$

where $\mathbf{I}$ is an identity matrix. Using equation (20) in (19) and neglecting body forces results in the simplified equilibrium equation as

$$\nabla \cdot \bar{\sigma}' + \nabla \bar{p} = \mathbf{0} \quad (21)$$

For a full description of the equilibrium equation, we need to introduce a constitutive model relating stresses and strains. Assuming isotropic linear elasticity, the effective stress tensor $\bar{\sigma}'$ can be expressed in terms of the infinitesimal strain tensor $\bar{\epsilon}$

$$\bar{\sigma}' = 2\mu \bar{\epsilon} + \lambda \text{tr}(\bar{\epsilon}) \mathbf{I} \quad (22)$$

where μ and λ are the Lamé parameters. The infinitesimal strain tensor $\bar{\epsilon}$ can be expressed in terms of the deformation vector $\bar{u}$ as

$$\bar{\epsilon} = \frac{1}{2}(\nabla \bar{\mathbf{u}} + (\nabla \bar{\mathbf{u}})^T) \quad (23)$$

For a two-dimensional problem, the deformation vector is $\bar{\mathbf{u}} = \{\bar{u}, \bar{v}\}^T$ where $\bar{u}$ and $\bar{v}$ are the deformations along the x and z directions, respectively. Using this and combining equations (21), (22) and (23) results in the following two equilibrium equations for a two-dimensional case:

$$(\lambda + 2\mu) \frac{\partial^2 \bar{u}}{\partial \bar{x}^2} + \mu \frac{\partial^2 \bar{u}}{\partial \bar{z}^2} + (\lambda + \mu) \frac{\partial^2 \bar{v}}{\partial \bar{x} \partial \bar{z}} + \frac{\partial \bar{p}}{\partial \bar{x}} = 0 \quad (24)$$

$$\mu \frac{\partial^2 \bar{v}}{\partial \bar{x}^2} + (\lambda + 2\mu) \frac{\partial^2 \bar{v}}{\partial \bar{z}^2} + (\lambda + \mu) \frac{\partial^2 \bar{u}}{\partial \bar{x} \partial \bar{z}} + \frac{\partial \bar{p}}{\partial \bar{z}} = 0 \quad (25)$$

Mass balance equation

The mass balance equation for a two-phase (solid matrix and fluid) saturated porous medium under isothermal conditions, obtained from superposition of the mass balance equations for the individual phases, is given by

$$\alpha \nabla \cdot \dot{\bar{\mathbf{u}}} + \left(\frac{\alpha - n}{K_s} + \frac{n}{K_f} \right) \frac{\partial \bar{p}}{\partial t} + \nabla \cdot \bar{\mathbf{w}} = Q \quad (26)$$

where $\alpha = 1 - K/K_s$ is Biot's coefficient, $\bar{\mathbf{u}}$ is the solid deformation vector, n is the porosity, K_s is the bulk modulus of the solid, K_f is the bulk modulus of the fluid, K is the total bulk modulus of the porous medium, $\bar{p}$ is the pore fluid pressure, $\bar{\mathbf{w}}$ is the fluid velocity vector and Q is a fluid source or sink term. For a porous medium with incompressible constituents, we have $1/K_s = 1/K_f = 0$ and ignoring any source/sink terms, the mass balance equation reduces to

$$\nabla \cdot \dot{\bar{\mathbf{u}}} + \nabla \cdot \bar{\mathbf{w}} = 0 \quad (27)$$

If the fluid flow in the porous medium is assumed to be driven only by pressure gradients (neglecting the effects of gravity), the fluid velocity can be described by Darcy's law and can be expressed as

$$\bar{\mathbf{w}} = -\frac{\mathbf{k}}{\gamma_f} \nabla \bar{p} \quad (28)$$

where $\mathbf{k}$ is the hydraulic conductivity matrix and γ_f is the unit weight of the fluid. For an isotropic porous medium, the hydraulic conductivity is the same in all directions of flow and the magnitude of the hydraulic conductivity, k ,

will apply to the diagonal components of $\mathbf{k}$. Based on equations (27) and (28) and the introduction of the deformation vector $\bar{\mathbf{u}}$, the mass balance equation reduces to

$$\frac{\partial}{\partial \bar{t}} \left(\frac{\partial \bar{u}}{\partial \bar{x}} + \frac{\partial \bar{v}}{\partial \bar{z}} \right) - \frac{k}{\gamma_f} \left(\frac{\partial^2 \bar{p}}{\partial \bar{x}^2} + \frac{\partial^2 \bar{p}}{\partial \bar{z}^2} \right) = 0 \quad (29)$$

Nondimensionalized poroelastic equations

The governing equations of poroelasticity in two dimensions are those given in equations (24), (25) and (29). The field variables are the deformations $\bar{u}$ and $\bar{v}$ and the pore pressure $\bar{p}$. For implementation into a PINN, it is more convenient to nondimensionalize the governing equations. Introducing a reference length l for the porous medium, we define the nondimensionalized spatial and deformation variables as

$$x = \frac{\bar{x}}{l}, \quad y = \frac{\bar{y}}{l}, \quad u = \frac{\bar{u}}{l} \quad \text{and} \quad v = \frac{\bar{v}}{l} \quad (30)$$

The corresponding nondimensionalized time and pore pressure variables can be derived and shown to be given by

$$t = \frac{(\lambda + 2\mu)k}{\gamma_f l^2} \cdot \bar{t} \quad \text{and} \quad p = \frac{\bar{p}}{\lambda + 2\mu} \quad (31)$$

Using equations (30) and (31) in the governing equations (24), (25) and (29), we get the nondimensionalized poroelastic equations as

$$f(x, z, t) = (\eta + 1) \frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial z^2} + \eta \frac{\partial^2 v}{\partial x \partial z} + (\eta + 1) \frac{\partial p}{\partial x} = 0 \quad (32)$$

$$g(x, z, t) = \frac{\partial^2 v}{\partial x^2} + (\eta + 1) \frac{\partial^2 v}{\partial z^2} + \eta \frac{\partial^2 u}{\partial x \partial z} + (\eta + 1) \frac{\partial p}{\partial z} = 0 \quad (33)$$

$$h(x, z, t) = \frac{\partial^2 u}{\partial t \partial x} + \frac{\partial^2 v}{\partial t \partial z} - \frac{\partial^2 p}{\partial x^2} - \frac{\partial^2 p}{\partial z^2} = 0 \quad (34)$$

where the nondimensional parameter η is given by

$$\eta = 1 + \frac{\lambda}{\mu} \quad (35)$$

PHYSICS-INFORMED NEURAL NETWORK MODEL

Model architecture

The model architecture used for the PINN model is a fully connected network where the number of layers and hidden units per layer are optimized during training. For the two-dimensional governing equations derived earlier, the PINN model accepts the spatial and temporal parameters $\{x, z, t\}$ as inputs and predicts the field variables $\{\hat{u}, \hat{v}, \hat{p}\}$ as outputs. Figure 38 shows an illustration of the model architecture. An important component of the PINN model is evaluation of the PDEs based on the predicted values of the field variables. This is performed using *automatic differentiation* which keeps track of the input variables and associated weights throughout the model. TensorFlow is used for implementation of the model and automatic differentiation is performed by defining custom training steps and gradient tapes.

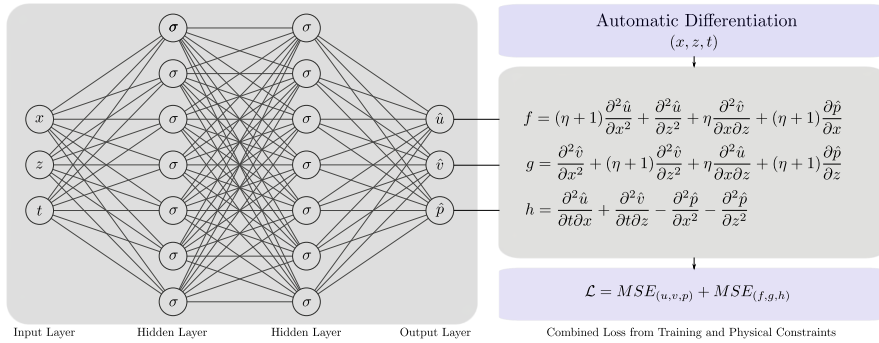


Figure 38 PINN model architecture. Note that the number of layers and units per layer is only for illustration purposes.

Loss metrics

The performance of the PINN model is measured by evaluating the losses both from the data-driven part of the training and the physics loss resulting from applying the PDEs as constraints to the model. The training data involves initial and boundary conditions for the specific problem under consideration. Mean squared error (MSE) is chosen as the metrics for calculating the data loss for each predicted field variable. Based on the actual field variable training data $\{u, v, p\}$ and their corresponding predicted values $\{\hat{u}, \hat{v}, \hat{p}\}$, the losses for each variable are calculated from

$$\begin{aligned}
MSE_u &= \frac{1}{N} \sum_{i=1}^N (u_i - \hat{u}_i)^2, \quad MSE_v = \frac{1}{N} \sum_{i=1}^N (v_i - \hat{v}_i)^2 \quad \text{and} \quad MSE_p \\
&= \frac{1}{N} \sum_{i=1}^N (p_i - \hat{p}_i)^2
\end{aligned} \tag{36}$$

where N represents the number of training data points. The total data loss is calculated as the sum of the individual data losses for each variable i.e.

$$MSE_{data} = MSE_u + MSE_v + MSE_p \tag{37}$$

The performance of the PINN model with respect to obeying the governing PDEs is measured by the mean squared error loss for each PDE. The physics losses are given by

$$\begin{aligned}
MSE_f &= \frac{1}{N_c} \sum_{i=1}^{N_c} f_i^2, \\
MSE_g &= \frac{1}{N_c} \sum_{i=1}^{N_c} g_i^2 \quad \text{and} \quad MSE_h = \frac{1}{N_c} \sum_{i=1}^{N_c} h_i^2
\end{aligned} \tag{38}$$

where N_c is the number of collocation points (x_c, z_c, t_c) where the PDEs are evaluated at. Note that residual terms, if there are any, need to be subtracted from each function during the evaluation of these losses. The total physics loss is the sum of the individual losses from each governing equation i.e.

$$MSE_{physics} = MSE_f + MSE_g + MSE_h \tag{39}$$

The total loss for the PINN model is the sum of the data and physics losses:

$$MSE_{total} = MSE_{data} + MSE_{physics} \tag{40}$$

The PINN model is trained to minimize this loss by adjusting the hyperparameters of the model, such as number of layers, number of units per layer, learning rate and batch size.

NUMERICAL EXAMPLE

Analytical solution

To demonstrate the curriculum training strategy, we first generate manufactured analytical solutions for the field variables u , v and p that

resemble a realistic two-dimensional geotechnical problem. The generated analytical solutions are:

$$u(x, z, t) = x \cdot (1 - e^{-\alpha \cdot z}) \cdot t \cdot e^{-\delta \cdot t} \quad (41)$$

$$v(x, z, t) = (1 - e^{-\beta \cdot z}) \cdot t^2 \cdot e^{-\epsilon \cdot t} \quad (42)$$

$$p(x, z, t) = 3z \cdot (1 - z) \cdot e^{-\zeta \cdot t} \quad (43)$$

where α , β , δ , ϵ and ζ are parameters that can be adjusted to generate various solutions. These manufactured analytical solutions are used to generate solutions for the field variables on a square domain with nondimensional x and z values ranging from 0 to 1 and nondimensional time t also ranging from 0 to 1. The parameters used to generate the solutions are $\alpha = 0.5$, $\beta = 2.0$, $\delta = 1.0$, $\epsilon = 1.0$, $\zeta = 1.5$ and $\eta = 2.5$. Figure 39 shows the analytical solution at $t = 0.5$ while Figure 40 shows a profile plot at $x = 0.5$ at selected time steps for the field variables u , v and p .

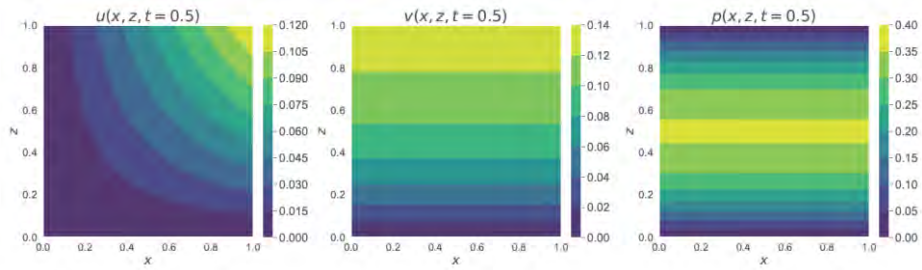


Figure 39 Analytical solution for u , v and p on the spatial domain at $t=0.5$.

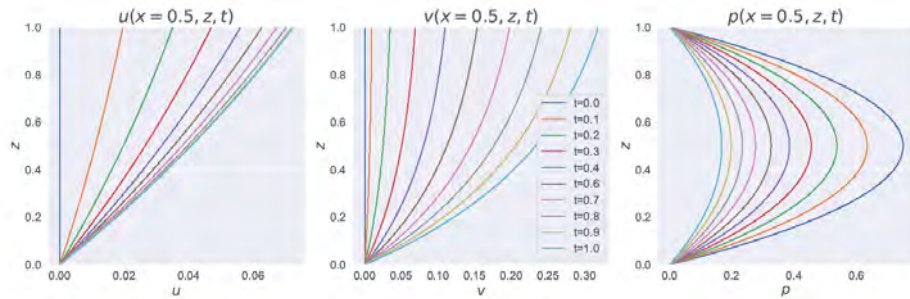


Figure 40 Profile plots of the analytical solution at $x=0.5$ for u , v and p .

These analytical solutions will not directly satisfy the governing PDEs and thus will result in residuals that need to be accounted for. The residuals can be derived by substituting the analytical solutions in Equations (41)-(43) into the governing PDEs in Equations (32)-(34). The resulting residuals corresponding to each PDE are presented in the Appendix.

Curriculum training strategy

Curriculum training is a learning strategy inspired by the idea of learning progressively, analogous to a human student learning a subject by gradually tackling more complex concepts. This approach, also known as curriculum learning, starts with simpler tasks or a subset of the data and progressively increases the difficulty level, either by including more complex examples or by expanding the data range. This methodology has shown effectiveness in various domains, such as natural language processing and computer vision, often leading to faster convergence and better generalization.

In the context of PINNs, curriculum training can be particularly advantageous when dealing with problems with complex physical processes. PINNs leverage the underlying physics of the problem, encoded in the neural network through the differential equations, to predict the field variables. For the specific case of time-dependent problems, like the poroelasticity problem we are dealing with here, curriculum training can be implemented by dividing the time range into intervals and training the model sequentially on these intervals. This strategy aligns with the temporal evolution of physical processes, allowing the model to first learn the initial behavior before advancing to more complex states as time progresses.

Figure 41 highlights the distinction between a standard training approach and one utilizing curriculum training. The plots show how initial and boundary condition data are extracted for training the PINN model using the standard approach and using curriculum training. The left plot shows the conventional approach where the model is trained on the initial and boundary condition data for the entire time domain. This can be challenging, as the model must learn to predict the complex dynamics across the entire time range at once. The right plot, however, demonstrates the curriculum training approach. Here, the time domain is discretized into 10 intervals, where training data for the intervals is extracted separately. Training begins with the first interval, closest to the initial conditions. Gradually, subsequent intervals are introduced, scaffolding the learning process as the complexity of the solution increases with time.

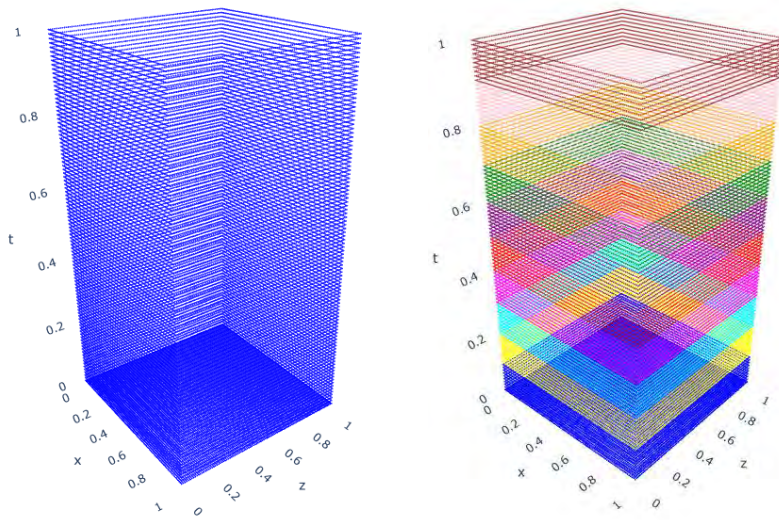


Figure 41 Extraction of initial and boundary condition training data for the poroelasticity problem. The plot on the left shows extraction of IC and BC data without curriculum training and the plot on the right shows extraction with curriculum training where the time range is divided into 10 intervals.

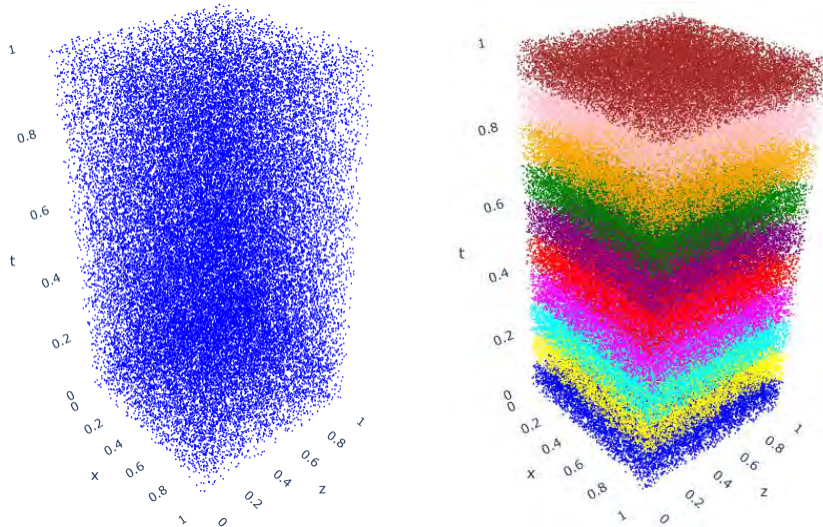


Figure 42 Generation of collocation points to evaluate the governing PDEs. The plot on the left shows generation of collocation data for the entire solution domain according to the standard training approach while the plot on the right shows generation of collocation points for the different training intervals for curriculum training.

Figure 42 compares two different methods for generating collocation points, which are used for evaluating the governing PDEs during the training of PINNs using automatic differentiation. The left plot on the figure shows collocation points scattered throughout the entire solution domain, generated using Latin hypercube sampling, for the standard training approach. This requires the model to tackle the complexity of the PDEs on the entire domain from the start, which can be quite demanding. On the right, the collocation points are generated progressively throughout the solution domain, matching the segmented intervals used for data extraction in curriculum training. This approach allows the model to focus evaluating the PDEs and optimizing the model gradually at different solution intervals, usually starting from simple to more complex sections. This helps to streamline the learning process, potentially leading to a more robust model as we will demonstrate in the following sections.

Training data with and without curriculum

The analytical solution is generated over the domain for a spatial discretization of $N_x \times N_z = 50 \times 50$ and a temporal discretization of $N_t = 50$. This implies a total dataset size of 125 000 for each of the input and output variables, (x, z, t) and (u, v, p) , respectively. But only the initial and boundary condition data are used for data-driven training while the PDEs are evaluated at randomly generated collocation points. After extraction of the initial conditions (2500 data points) and the boundary conditions (9800 data points), the total size of the data for training becomes 12 300 for each of the input and output variables.

Training without curriculum: For model training without the curriculum training strategy, the initial and boundary condition data throughout the domain are used at once. The batch size is adjusted for optimal training and the data is shuffled during training. The Latin Hypercube Sampling (LHS) method is used to generate collocation points in the solution domain. The number of collocation points generated in this case is 1000.

Training with curriculum: For model training with the curriculum training strategy, the temporal domain is divided into the desired number of intervals. For our example here, it is divided into 10 equally spaced intervals. For example, the first interval involves initial and boundary condition data of size 250 and 980, respectively. For subsequent intervals, only boundary condition data of size 980 are relevant. The number of collocation points generated for each interval is 100, implying a total number of collocation points of 1000 as in the case of training without curriculum.

Model hyperparameters

After some fine tuning, the following model hyperparameters are selected for the PINN model. The number of hidden layers and hidden units per layer is

chosen as 5 and 20, respectively, while the *tanh* activation function is chosen. A batch size of 256 is selected to be optimal. Adam optimizer is used with a learning rate of 0.001. These hyperparameters apply to model training both with and without the curriculum training strategy.

Results

Model training is performed for 3000 epochs for both model training with and without the curriculum training strategy. The loss versus epoch plot for model training without curriculum is shown in Figure 43. The corresponding plots for model training with curriculum are shown in Figure 44. For the particular numerical example here, the loss decreases optimally with the number of epochs in both cases, but the model takes a longer time to train in the first case i.e. model training without the curriculum training strategy. For the selected model hyperparameters and number of epochs, model training with the curriculum training approach takes approximately 3 hours on a laptop with a Core i7 CPU while a similar model without curriculum training takes nearly 6 hours on the same hardware.

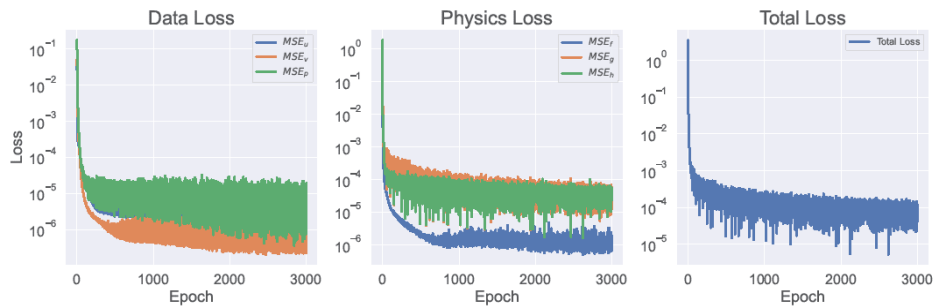


Figure 43 Data loss, physics loss and total loss versus epoch plots for model training without the curriculum training strategy.

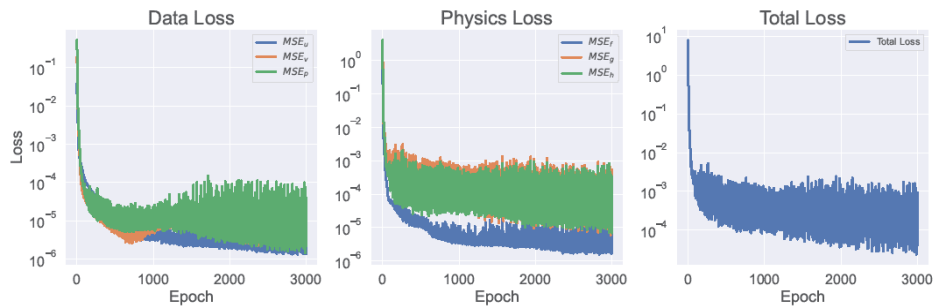


Figure 44 Data loss, physics loss and total loss versus epoch plots for model training with the curriculum training strategy.

Figure 45 shows a comparison between the analytical versus predicted solutions over the solution domain for model training without the curriculum training strategy. The results are shown for the solution of the field variables u , v and p at $t = 1.0$. The corresponding results for model training with the curriculum training approach are shown in Figure 46. The predicted solutions in both cases were found to be in good agreement with the analytical solutions.

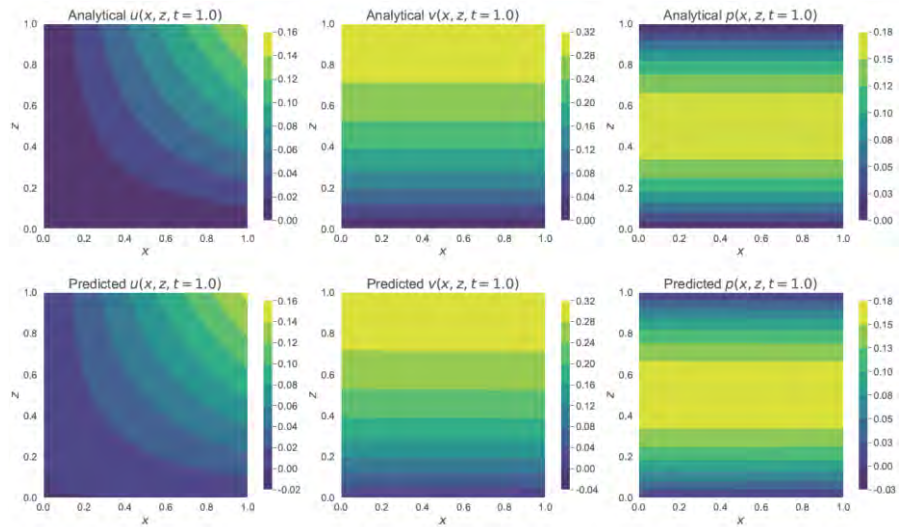


Figure 45 Analytical versus predicted solutions for the field variables u , v and p at $t=1.0$ for model training without curriculum.

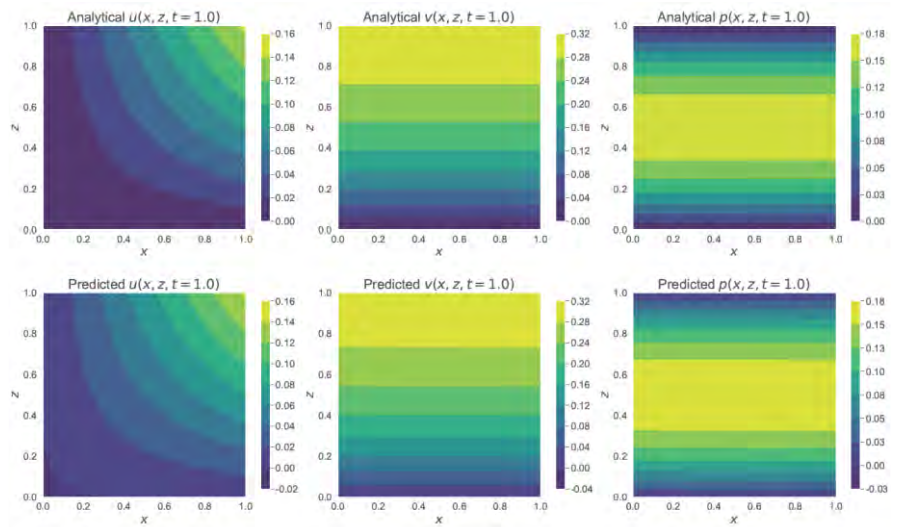


Figure 46 Analytical versus predicted solutions for the field variables u , v and p at $t=1.0$ for model training with curriculum.

To examine the quality of the predicted solutions, we look at the profiles of the field variables along the z direction for a selected x value and selected time steps. The resulting profiles are shown in Figure 47 and Figure 48 for model training without and with curriculum, respectively. In both figures, the profiles are plotted at $x=1.0$ and for a time range between 0 and 1 with an interval of 0.1.

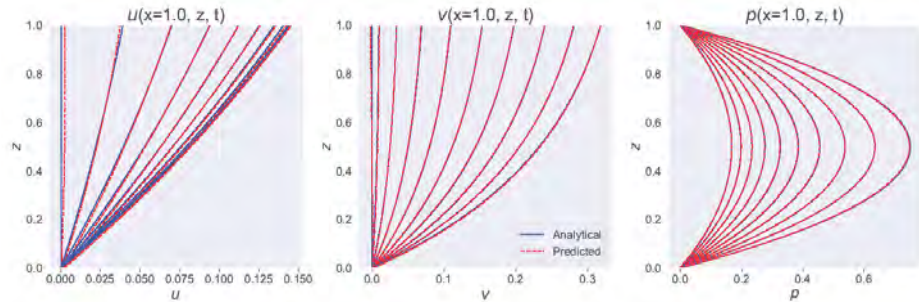


Figure 47 Analytical versus predicted solution profiles for the field variables u , v and p at $x=1.0$ for model training without curriculum. The profiles are shown at t intervals of 0.1 from 0 to 1.

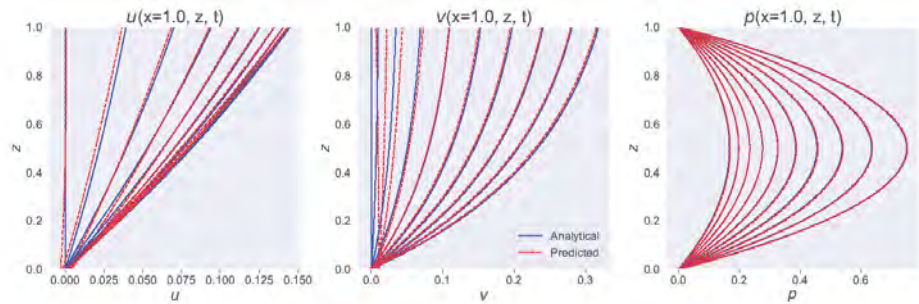


Figure 48 Analytical versus predicted solution profiles for the field variables u , v and p at $x=1.0$ for model training with curriculum. The profiles are shown at t intervals of 0.1 from 0 to 1.

In the numerical example discussed, both training approaches—using a curriculum training strategy and without it—yield reasonably accurate predictions. However, the primary distinction lies in the training duration. Training with the curriculum strategy is observed to be nearly twice as fast as the conventional method under the same model setup and using identical hardware. This efficiency presents a considerable advantage, especially when training PINNs for complex problems. In scenarios involving significant temporal variations within the solution domain, the curriculum-based approach is likely favored. It is anticipated that this method not only enhances training efficiency but also improves the accuracy of the predictions. Further

exploration and demonstration of this hypothesis on complex temporal problems will be pursued in future work.

CONCLUDING REMARKS

The research trend on PINNs in various science and engineering disciplines is briefly highlighted. While PINNs offer great promise for various practical applications, there remain several challenges in training PINN models and reaping their potential practical benefits for faster, near real-time or real-time numerical prediction. Of the various challenges that need to be addressed regarding the training of PINNs, we focused on reducing the high computational cost of training PINN models and improving the quality of the predicted solutions they offer. For this purpose, a curriculum training strategy is employed for developing a PINN model for coupled poroelastic flow and deformation processes. An idealized numerical example is used for to demonstrate the curriculum training strategy where the PINN model is trained gradually by introducing the training data at intervals along the temporal dimension of the solution domain. In line with the different temporal intervals, the governing PDEs are evaluated at collocation points corresponding only to that interval. The curriculum training approach is compared with a conventional training approach where the training data over the whole spatial and temporal domain of the solution is used at once for training and the PDEs are evaluated at collocation points over the whole solution domain. The results show that the curriculum training approach enables training of the PINN model nearly two times faster than the conventional approach for a similar model setup and on the same hardware. For the particular example here, the predicted solutions for both training approaches were found to be in good agreement with the reference solution. It is however anticipated that the curriculum training approach, in addition to enabling faster model training, could result in a superior prediction capability for problems with more temporal complexity in the solution. This will be a subject for further research.

APPENDIX

The residuals corresponding to the analytical solutions and the respective PDEs are:

$$\begin{aligned}
 r_u(x, z, t) &= -\alpha^2 \cdot t \cdot x \cdot e^{-\alpha \cdot z - \delta \cdot t} \\
 r_v(x, z, t) &= (\alpha \cdot \eta \cdot t \cdot e^{\beta \cdot z + \epsilon \cdot t + t \cdot \zeta} - \beta^2 \cdot t^2 \cdot (\eta + 1) \cdot e^{\alpha \cdot z + \delta \cdot t + t \cdot \zeta} \\
 &\quad - 3 \cdot (\eta + 1) \cdot (2z - 1) \cdot e^{\alpha \cdot z + \beta \cdot z + \delta \cdot t + \epsilon \cdot t}) \\
 &\quad \cdot e^{-\alpha \cdot z - \beta \cdot z - \delta \cdot t - \epsilon \cdot t - t \cdot \zeta} \\
 r_p(x, z, t) &= -\beta \cdot t \cdot (\epsilon \cdot t - 2) \cdot e^{-\beta \cdot z} e^{-\epsilon \cdot t} - (1 - e^{-\alpha \cdot z}) \\
 &\quad \cdot (\delta \cdot t - 1) \cdot e^{-\delta \cdot t} + 6 \cdot e^{-\zeta \cdot t}
 \end{aligned}$$

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PILE CAPACITY PREDICTION BASED ON MACHINE LEARNING USING SUPPORT VECTOR REGRESSION

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KEYWORDS

Machine Learning, Pile bearing capacity, Support Vector Regression

ABSTRACT

This study investigates the capability of a Machine Learning algorithm, Support Vector Regression (SVR), to predict the bearing capacity of driven concrete piles. Restrikes conducted to make Dynamic Load Tests (DLT) were used for training the algorithm as the DLT-derived capacity was used as the target value. The performance is compared to the Danish Driving Formula. The research explores the importance of domain knowledge and feature engineering in the process of selecting, manipulating, and creating relevant input variables (features) that improve model performance and generalization. The trained model demonstrates a very high correlation with the target and strong confidence in its bearing capacity predictions, outperforming the Danish Driving Formula. Sensitivity analyses show that a feature-engineered model has improved generalization and interpolation capabilities in parameter regions with few or no data in the training set.

INTRODUCTION

Accurate assessment of the bearing capacity of driven piles is a critical aspect of geotechnical engineering, influencing the design of foundations and the economy of the structure. Data from pile driving records can be used to predict pile bearing capacities, but the capacities are significantly affected by the build-up of excess pore water pressure during pile installation. Restrikes after a short time provide a valuable source of information that is significantly less affected by set-up. Restrikes can be made at low cost on piles after end-of-driving, where some combination of pile set and drop height is registered.

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This study aims to utilize the restrike data, which are readily available or can be obtained at low cost.

Often, *pile driving formulas*, e.g., the Danish Driving Formula [1] are used to determine pile capacities. These semi-empirical methods are based on energy considerations and use the driving data (end-of-driving or restrike). Other pile capacity prediction methods include Static Load Test (SLT) and Dynamic Load Testing (DLT) with signal matching, e.g., the CAPWAP method [2]. Where SLTs are often scarce due to their circumstantial setup and time consumption, DLTs are fast and therefore often preferred.

The Danish Driving Formula is simple to use and remains popular as it allows for immediate estimation of the present pile capacity during driving. However, it assumes that all energy related to the hammer impact is converted into pile penetration except for the strain energy related to elastic pile compression, and it has not been recalibrated for decades. Now, more data types are accessible, and piling equipment has become more advanced. Machine Learning (ML) offers a data-driven approach and enables the inclusion of other relevant variables, e.g., if a pile is coupled, without the need to establish physics-based equations.

Several ML techniques have been applied in geotechnical engineering with Artificial Neural Networks (ANNs) being a popular choice in research. In 1995, Chan et al. demonstrated the feasibility of ANN on the prediction of the ultimate pile bearing capacity from pile driving records, outperforming pile driving formulas [3].

Another popular ML technique is Support Vector Machines (SVM) [4] which has been found to work well for geotechnical applications in comparison with back-propagation neural networks [5]. In 2010, Pal and Deswal compared SVM to the Gaussian Processing (GP) algorithm, which suggests that the latter has a slightly improved prediction capability [6]. Both SVM and GP were shown to significantly outperform pile driving formulas.

This paper aims to verify previous findings and compare the performance of ML to the Danish Driving Formula. This study's focus is, by applying domain knowledge to the data science approach, to include other variables and to manipulate features to improve the performance and generalization of the trained algorithm. Hence, only a single algorithm, SVM, will be applied. It is chosen for its relatively simple and logical formulation and because it is supervised and popular for solving similar geotechnical problems, where it has shown promising results.

Section 2 provides more detailed insight into the background of the paper, and Section 3 outlines the methodology. The results are presented in Section 4 and discussed in Section 5, while Section 6 provides the overall conclusions.

BACKGROUND

Driving a pile induces excess pore water pressure within the soil penetrated. When the pore pressures dissipate, the effective stresses increase which in turn increases the pile capacity. The time-dependent increase in pile bearing capacity arising from pore water dissipation and other secondary mechanisms such as creep and thixotropy is referred to as set-up [7]. DLTs, for which restrikes are made to record stress waves, are most often conducted a period of time after end-of-driving to more accurately verify the pile capacity. CAPWAP analysis of the recorded stress waves provides a very reliable determination of the ultimate capacity of driven piles [2].

Therefore, the DLT with CAPWAP analysis is used as the target value in this study. By comparing data of the restrike for DLT with the DLT itself, the comparison is not affected by time, i.e. pile set-up. A model can be trained with this data set to predict DLT results, and if applied at restrikes at a different (earlier) time, the result reflects the capacity at that specific time.

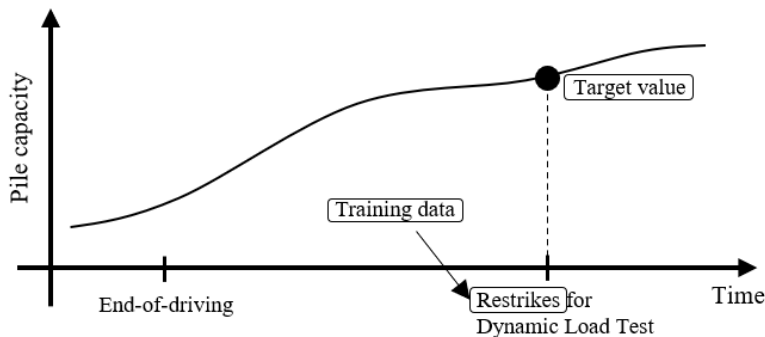


Figure 1 Schematic pile capacity development due to set-up.

Different ML techniques have previously shown improved capabilities of predicting the ultimate pile capacity in comparison with different empirical driving formulas not including the Danish Driving Formula. However, these research projects have focused on comparing prediction performance between different machine learning techniques and relating it to empirical methods [3][5][6]. Little attention was paid to the features used by the algorithms to learn if any additional information will benefit the performance, or how features may be manipulated to achieve generalization and increase performance. This study attempts to improve the performance and generalization capabilities of the trained ML algorithm using a domain knowledge-based feature engineering approach.

METHODOLOGY

For building the machine learning model, Support Vector Regression (SVR) is used. SVR finds a hyperplane in a high-dimensional space that best fits the data points while allowing some errors within a margin. A kernel function

transforms the data points into a higher-dimensional space from where the optimal hyperplane is found, and for this non-linear problem, the radial basis function (rbf) kernel is chosen. The data points closest to the hyperplane are the support vectors which have the greater influence on the hyperplane. In regression, the margin that minimizes the distance between the hyperplane and the data points is determined by the hyperparameters (C regularization, ε margin of tolerance, and γ kernel coefficient) which must be tuned in a grid search for optimal performance [8].

The SVR model is compared to the Danish Driving Formula [1], which takes the form:

$$R_{\text{dyn;m}} = \frac{\eta h G}{s + 0.5 s_0} \quad ; \quad s_0 = \sqrt{\frac{2 \eta h G_h L_p}{A E}}$$

and uses six variables: the hammer drop height h (m), the hammer weight G_h (kN), pile set per hammer blow s (m), pile length L_p (m), and cross-sectional area of the pile A (m²). The nondimensional efficiency factor of the hammer η is equal to 1.0 for modern hammers, and Young's modulus E is set at a constant uniform value of 20 GPa for concrete. The subscript “dyn;m” refers to a *dynamic* resistance that is *measured* and subsequently regulated by partial factors to provide a design value.

Data set

The data set that is used to build the SVR model is a sample of DLT reports of 367 impact-driven piles installed for 16 commercial projects scattered across the western part of Denmark. The target value is the derived pile bearing capacity of these reports based on the CAPWAP method which ranges from 600 to 4610 kN. This includes concrete piles with square cross-sections having side lengths ranging from 25 to 40 cm. The pile lengths range from 8 to 30 m where piles longer than 18 m are coupled with a pile joint. The piles are embedded in various common Danish soils of sand, clay, silt, gravel, clay till, and unhardened limestone, and the environment of deposition ranges over marine, sub-glacial, and meltwater, while the ages of deposition are either Danien or Quaternary. The hammer type used is either free-falling or accelerating. The restrikes conducted for the DLTs include the following variables: hammer weight (60 to 90 kN), drop height (0.2 to 1.2 m), and pile set (5 to 138 mm). Due to the increased significance of measurement error, pile sets lower than 5 mm have been disregarded.

Data processing and model building

Data exploration has been performed to better understand the data and its limitations. For example, the distribution of the pile lengths in Figure 2 shows a tendency for the pile capacity to increase with pile length though non-linearly. On the contrary, shorter piles do not necessarily have a lower

capacity as the distance to firm ground is independent of the pile length but not vice versa. Although stresses generally increase with depth, the pile base depth should be disregarded to avoid the deduction that capacity is always directly proportional to pile length.

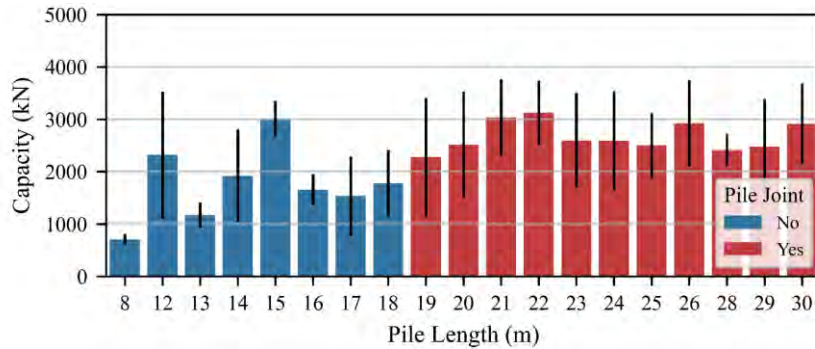


Figure 2 Mean capacity $\pm$ one standard deviation (black) vs. pile length grouped by pile joint type.

Features have been manipulated to achieve better generalization using domain knowledge of the physics behind the data and the relations between individual parameters. An example of this feature engineering is that the features *Hammer Weight* and *Drop Height* are multiplied into *Hammer Energy* as both parameters hold information about the input energy. Yet, for the hammer weight, the data set does not capture the parameter space well. In combining the features, a new parameter space is created which may close gaps in the data and provide better coverage of the population as illustrated in Figure 3.

An alternative approach of employing an unsupervised approach such as the Principal Component Analysis (PCA) submits control and domain knowledge from the model building process, and the model behavior becomes non-explanatory as opposed to a manual feature engineering approach.

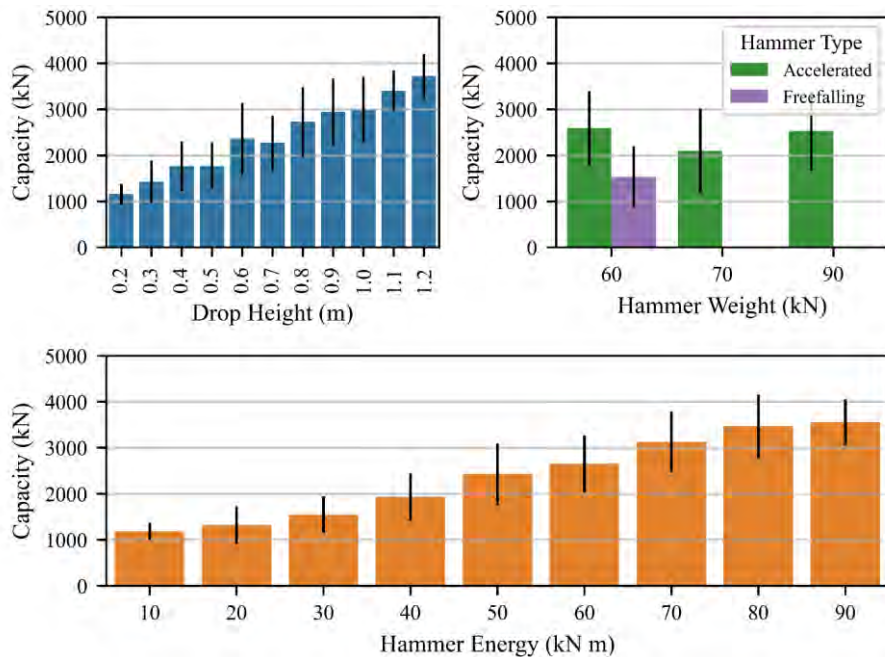


Figure 3 Exploration of initial and manipulated features.

The model building itself follows the iterative process described in Figure 4. For each iteration, the data set is split randomly with 80 % distributed into a training set and the remaining 20 % into a test set (the same random split is used throughout). The model is trained on the training set with a k -fold cross-validation scheme. This is done to provide robustness for the estimation of model performance, reduce the risk of overfitting, and assess how well the model will perform on unseen data. The main evaluation score is set as the coefficient of determination, R^2 , evaluated with Scikit-learn [8].

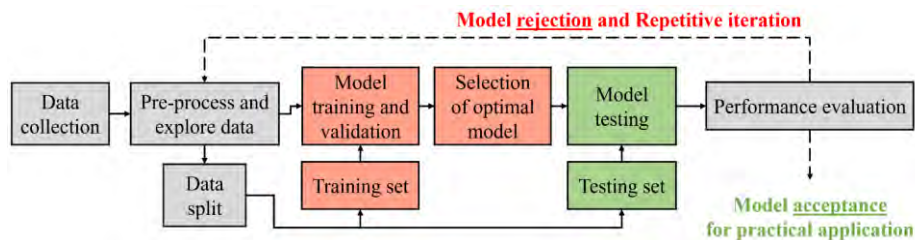


Figure 4 Development process of a supervised machine learning model.

For a random data split, two SVR models have been trained to predict pile capacity: A Baseline model with the same input features as in the Danish Driving Formula (see Eq. (1)), and a Feature-Engineered model. The hyperparameters have been optimized in a three-dimensional grid-search of $10 \times 8 \times 11$ ($C \times \varepsilon \times \gamma$) with a k -fold cross-validation scheme of a random

data split with 5 folds and 3 repetitions, where the models were evaluated for each iteration with the radial basis function kernel.

RESULTS

The feature selection relied on physical considerations of the pile that encapsulate the proportions of the pile geometry, the impact energy, and the soil resistance based on available data on the driven piles. The result of that process is the following features: The proportions and geometry are described in feature spaces of the pile dimension B (m), pile aspect ratio L_p / B (-), pile joint and hammer type (categoricals), pile reinforcement ratio $A_{\text{steel}} / A_{\text{concrete}}$, and the weight ratio of the hammer to pile G_h / G_p (-). The majority of engineering features are based on normalized parameters to spread out the data points of the feature spaces more evenly. The Hammer Energy $E_h = h \times G_h$ (kNm) represents the input energy, and the pile set per blow s (m) reflects the soil resistance.

The performance of the SVR models for pile capacity prediction as evaluated on training and test sets are shown in Figure 5, the score metrics are shown in Table 1, and the SVR models are compared against the Danish Driving Formula in Figure 6.

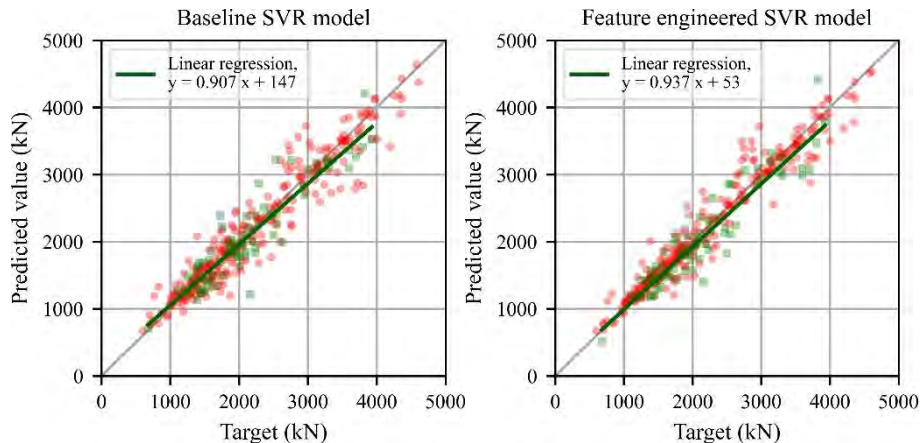


Figure 5 Pile capacity predictive performance of Baseline (left) and Feature-Engineered (right) SVR models. Models are evaluated on the training set (red) and test set (green). The regression line is the test set only.

From Figure 5 and Table 1, it is seen that the training scores are slightly lower than the test scores. The performance of the feature-engineered model is slightly better than the baseline model. In Figure 6, none of the methods is associated with a uniform bias. However, the Danish Driving Formula tends to overestimate the capacity for low capacities and underestimate the capacity for high capacities. This is not the case for the ML approach, though model uncertainty is also present here in the form of fluctuations.

Table 1 Performance metrics of the linear regressions between the model predictions and target. R^2 : coefficient of determination, RMSE: Root Mean Square Error.

Model, Hyperparameters	R^2		RMSE	
	Train	Test	Train	Test
SVR (baseline) $C = 10^{4.5}$, $\varepsilon = 70$, $\gamma = 10^{-1.4}$	0.923	0.846	258	288
SVR (feature-engineered) $C = 10^{4.5}$, $\varepsilon = 70$, $\gamma = 10^{-1.2}$	0.946	0.865	215	269
Danish Driving Formula	0.760	0.687	455	410

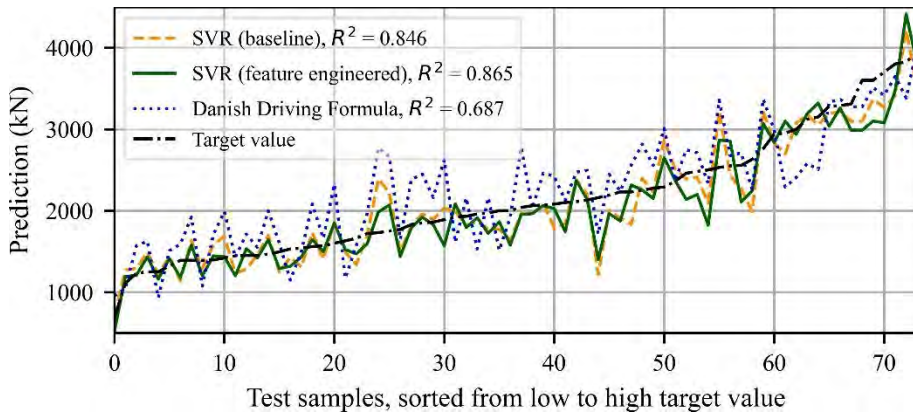


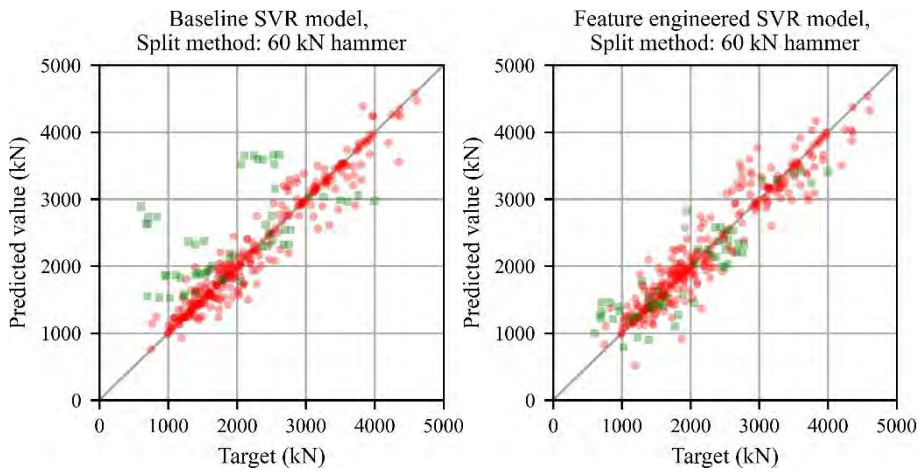
Figure 6 Comparison of the predictive capabilities of SVR models against the Danish Driving Formula on each sample in the test set.

In comparison, Pal and Deswal (2010) used eight input features (E , L_p , G_p , h , G_h , s and *hammer type* (*gravity* or *steam*)) to train an SVR model to achieve a coefficient of determination $R^2 = 0.855$ and RMSE = 372 kN on test samples [6]. The results of the baseline SVR model in this study are similar.

A second analysis is conducted with alternative split methods. The first retains all hammer weights of 60 kN (15 % of the data set), and the second retains all data from a single project (16 % of the data set) to mimic a real-world case and practical application. The results of the analyses are shown in Table 2 and Figure 7. The hyperparameters are tuned similarly besides $\varepsilon = 10$. A lower ε ensures that more predictions are penalized and more become support vectors.

Table 2 Metrics of the alternative split methods for the training process.

Split method	Model, Hyperparameters	Train, R^2	Test, R^2
60 kN hammer	SVR (baseline) $C = 10^4, \gamma = 10^{-0.75}$	0.942	-0.133
	SVR (feature-engineered) $C = 10^5, \gamma = 10^{-2}$	0.913	0.802
Single project	SVR (baseline) $C = 10^{3.5}, \gamma = 10^{-1}$	0.937	0.331
	SVR (feature-engineered) $C = 10^4, \gamma = 10^{-1.25}$	0.942	0.632



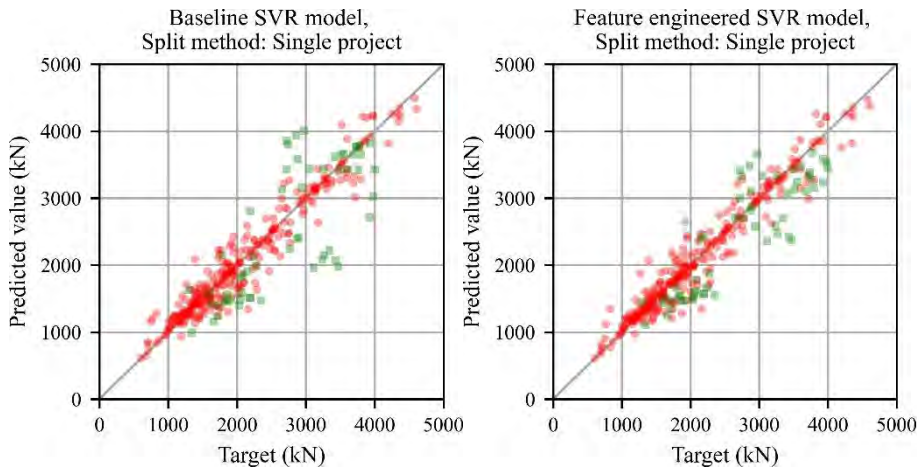


Figure 7 Pile capacity prediction performance of SVR models with 60 kN hammer data points in the test set (above), and a single project case in the test set (below). Model evaluated on training set (red) and test set (green).

DISCUSSION

The SVR models show increased accuracy and precision compared to the Danish Driving Formula. The performance gap for predictions on the test set compared to the training set indicates overfitting of the models. However, even the Danish Driving Formula shows poorer performance on the test set compared to the training set (Table 1), suggesting an uneven split with underlying bias or greater error sources in the test set. The relatively lower gap in the score metrics of the SVR models between the training set and the test set in Table 1 shows them to be well-balanced.

The data set comprises only 16 commercial projects, in which typically the same *hammer weight* is used for an entire project. Underlying tendencies for individual projects may be captured by the baseline model through this feature that causes overfitting. The feature engineering efforts, such as constructing *hammer energy*, mitigate this. In the alternative split method where 60 kN hammers were withheld, the baseline model is extrapolating as it was only trained on 70–90 kN hammers (Figure 3). On the other hand, feature engineering makes the model cover a wider parameter space that makes it interpolate within the range of *hammer energies*. The feature-engineered model works nearly as well as for the random split (Figure 7). Some of the performance loss may be attributed to the indirect removal of the *Hammer Type*.

In the single-project split method where data from a whole project site was withheld in training, the feature-engineered model performs better than the baseline model (Figure 7). In this split method, parts of the training set's

parameter space may be insufficiently covered in training for the model to accurately predict the target values of the test set. This causes the performance to decrease in comparison to the models trained with a direct random data split, which suggests a level of extrapolation of new projects. This observation emphasizes that the practical application of SVR requires a larger data set that must be representative of the population intended for its application.

As opposed to *Deep Learning* in which feature engineering is made by the algorithm, supervised *Machine Learning* allows the human behind to apply his/her domain knowledge and modify features based on known physics behind the data. This can generalize the model, mitigate overfitting, and make the model more trustworthy. In this research project, knowledge of the driving mechanism was the basis for a feature engineering process that led to the identification of three basic parameters that represent the hammer strike, pile geometry, and soil reaction, and, in addition, ratios between parameters that are more relevant than the original values. Equally important, the data exploration and feature engineering process make the model developer aware of any boundaries present in the parameter space.

The model performances would arguably be improved had the data set been soil-specific. However, the lack of restrictions on the soil type at the pile tip or along the pile shaft enables broader application.

CONCLUSION

This paper has shown that feature engineering can significantly improve the accuracy and generalization capabilities of an ML regression model based on the input data generally used for the semi-empiric Danish Driving Formula. This is done using domain knowledge while maintaining control over the learning process of the models. On a direct comparison of the coefficient of determination R^2 in predicting the pile bearing capacity as determined by the CAPWAP method, the SVR model, with a random data split, outperforms the Danish Driving Formula with 0.865 against 0.687.

There are several advantages to using data-driven methods in geotechnics. Complex relations may be made between two variables dependent on a larger multi-dimensional set of features. Supervised machine learning, as opposed to deep learning, allows for further control over the training enabling implementation of domain knowledge of the data and the physics behind them. This mitigates the issue of the “black box” that is inherent in employing machine learning.

The research has shown that the Feature-Engineered model can confidently interpolate over a limited space of original parameters with poor coverage. However, extrapolation and overfitting are shown to be mitigated by a generalization of the features. Yet, the model requires more training data

before practical application to enable use on new projects without recalibration.

Future research should involve training on a larger data set with dozens of project cases to learn whether the model can perform well on new project cases unseen in training. Additionally, using the same or similar feature-engineered parameters in a comparison between different ML algorithms is suggested for future research.

ACKNOWLEDGEMENT

Aarsleff provided the data of the 16 commercial project cases used in this study. This is greatly appreciated.

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PILED FOUNDATION FOR THE CONTAINER TERMINAL AT YDRE NORDHAVN

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Mette Hansen¹, and Jørgen S. Steenfelt¹**

KEYWORDS

Driven piles, Concrete slab, Limestone, Dynamic tests, Soil set-up, Global Mapper

ABSTRACT

As part of the development of the Nordhavn area in Copenhagen a new container terminal is under construction (2024). The new container terminal is located on a reclaimed land with a stock yard area of around 133,000 m² paved with a concrete slab supported by more than 4000 concrete driven piles. The pile sizes varied between 250 mm x 250 mm and 350 mm x 350 mm. Generally, the soil conditions at the site consisted of 15-23 m of reclaimed material (clay or sand fill) on top of gyttja underlain by limestone. Part of the gyttja layer was placed as a membrane in order to separate polluted reclamation fill from the naturally deposited gyttja underneath. The limestone elevation was found between -15 m DVR90 and -23 m DVR90. The borehole records indicate that the degree of induration varies from H1 to H5.

The aim of this paper is to describe the piled foundation design approach, where the limestone surface and required pile toe elevation for all production piles were established based on borehole and test pile data, respectively. The paper provides detailed site conditions, determination of soil set-up factor and the design approach. Finally, a statistical approach to treating, compiling, and evaluating the data for the installation of the 4000+ production piles is presented.

INTRODUCTION

Driven prefabricated reinforced concrete piles are the default choice for a piled foundation in Denmark unless prevented by environmental issues or capacity restrictions. To achieve a high required pile resistance, piles frequently penetrate very stiff or dense soils or rocks. This places high demand on the structural capacity of the piles.

For the new container terminal project, which is a part of the development of the Nordhavn area in Copenhagen, driven prefabricated reinforced concrete

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piles were chosen as foundation below a large concrete slab that supports the stock yard area of around 133,000 m². To limit the settlements of the slab the piles were required to be installed into limestone to rely on very high toe resistance.

To optimize the design of the involved 4000+ piles a major testing campaign was initiated. The goal was to establish robust driving criteria for the piles to avoid damage to the piles and at the same time ensuring that the target capacity could be reached. To achieve these goals test piles were driven and re-driven with complete driving records followed by PDA/CAPWAP testing. The latter allowed for estimation of the set-up after pile driving to be established.

The paper describes the design approach used for the project, the on-site handling of the pile driving and provides insight into the way the statistical data, based on the test piles and the installation of production piles allowed for overall management of the pile driving process.



Figure 49 Site location [4].

MATERIALS AND METHODS

Construction site

The construction site is an area in the harbour of Copenhagen reclaimed between 2012 and 2022. As illustrated in Figure 50, part of the reclaimed area was subjected to preloading by placing a temporary fill (marked with red lines on the figure). Another part of the reclaimed area (marked with light blue lines on Figure 50), partly overlapping the preloaded area, was initially covered with a layer of gyttja, as a membrane, in order to separate more or less polluted reclamation fill from the natural soil deposit underneath. The reasons for the shape of the preloading area and gyttja membrane are not known. However, it seems possible that the preloading area was meant to cover the area with the thickest gyttja layer as shown in Figure 51.

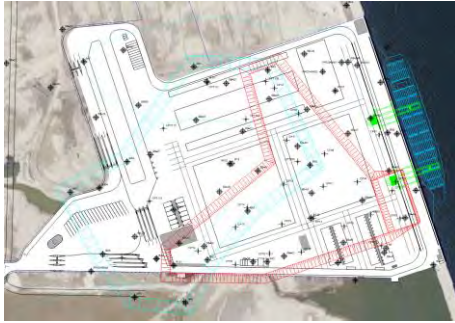


Figure 50 Construction site with the assumed location of the pre-consolidation (marked with red) and the approximate area of gyttja-

membrane (marked with light blue) [3].

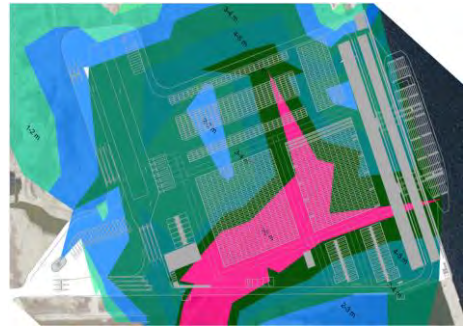


Figure 51 Variation of gyttja thickness at the construction site [3].

Soil conditions

Ground investigation campaigns between 2006 and 2022 indicated a soil profile consisting of a 15-23 m thick layer of reclaimed material (soft to stiff clay or loose to compact sand fill) underlain by up to 7 m thick layer of gyttja on top of limestone. Some boreholes indicate a late glacial or glacial gravel layer (up to 7 m thick) between gyttja and limestone. The undrained shear strength of gyttja derived based on the field vane test results and correlation factor of 0.8 was found between 30 and 110 kPa. The unconfined compressive strength of limestone is presented in Figure 52. The lower bound 95% confidence limit and mean value are 12.7 MPa and 19.9 MPa, respectively.

The top of the limestone across the construction site presented as an elevation heat map is shown in Figure 53. The heat map was created in a commercial software called Global Mapper [1] using the coordinate data from the available boreholes. As can be seen from the figure the top of the limestone is located between -14.6 m DVR 90 and -23.5 m DVR90.

Pile test programme

The test programme consisted of two phases of dynamic tests. In the first and second phase 30 and 97 test piles were installed and tested in two rounds, respectively. The first round of the dynamic tests was performed between 1 and 4 days after pile installation. The second round of the dynamic tests was conducted at least 7 days after the first round. Different pile sizes (250 mm, 300 mm and 400 mm) and lengths (21 – 31 m) were used in the test campaign.

The main goal of the testing campaign was to find the achievable pile resistance and potential soil set-up factor allowing driving and termination criteria for all the production piles to be established. The pile resistance was

determined with CAPWAP analysis. The soil setup, i.e. gain in pile resistance with

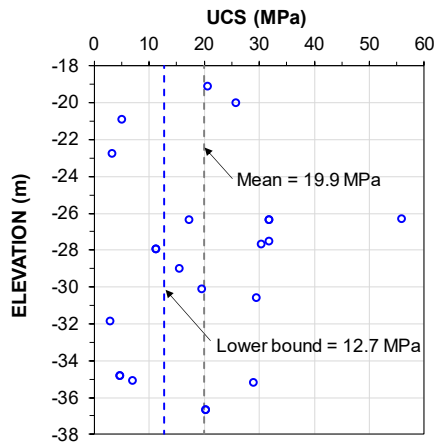


Figure 52 Unconfined compression strength of limestone.

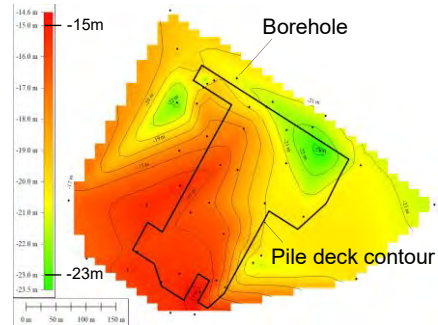


Figure 53 Top of limestone elevation with the pile deck contour and location of the boreholes.

time, was established based on two rounds of CAPWAP measurements and extrapolated to values 28 days after pile installation. Afterwards, the extrapolated pile resistance was divided by the initial end-of-driving resistance in order to obtain the soil set-up factor. The end-of-driving resistance was determined based on Danish Pile Driving Equation (DPDE). The driving and termination criteria (including the potential soil set-up factor) were determined to avoid hard driving and damage to the piles.

Data collection

Managing data from driving journals for over 4000 piles across various spreadsheets is cumbersome and resource intensive. To streamline this process, a single Excel tool has been created. This tool allows for easy importation of individual or multiple driving journals in Excel format with the click of a button. Utilizing Visual Basic for Application (VBA), the tool imports data and calculates pile capacity, ensuring convenient access to specific pile information. Additionally, it can import CAPWAP test results from PDFs, achieved through a combination of VBA and Power Query to convert the PDF into tabular format. The tool offers various features, including a plan for tracking pile status and the ability to create depth-based capacity charts.

Determination of the expected length of production piles

The main requirement for the installation of all the production piles was to embed them into limestone. The top of limestone was determined for all the production piles based on the map from Figure 53. The design axial

compression pile load in the ultimate limit state was 1400 kN. The elevation, at which a test pile reaches an equivalent pile resistance measured at the end-of-driving (including the potential soil set-up factor, correlation, and partial factors), was determined for every test pile as presented in Figure 54.

Afterwards, similar to the creation of the top of limestone elevation heat map, another heat map was established showing elevations at which piles were assumed to reach the required end-of-driving resistance (or ULS load).

Assessment of production pile installation

Driving records of all the production piles (4000+ piles) were stored in digitized driving journals. All the journals were imported daily to the project's dedicated excel spreadsheet to assess the installation of the production piles. The initial assessment was done automatically by the spreadsheet following the prescribed criteria:

- “Green” when a pile reached the required pile resistance and the expected top of the limestone.
- “Yellow” when a pile reached the required pile resistance, but its pile toe was located up to 0.5 m above the expected top of the limestone.
- “Red” when a pile did not reach the required pile resistance, its pile toe was located more than 0.5 m above the expected top of the limestone, or its pile head was located below the required elevation (i.e. the pile was installed too deep)

Every pile was checked by an engineer after the initial assessment.

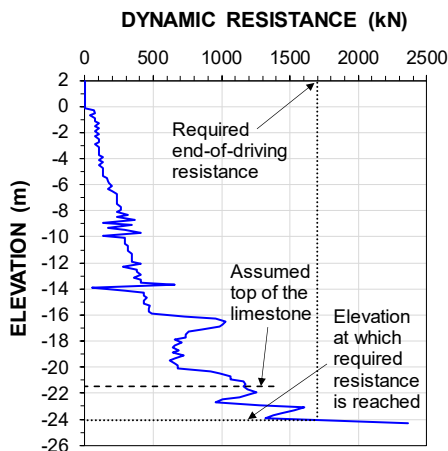


Figure 54 Determination of the expected pile toe elevation for a single test pile

2. RESULTS

Soil setup

Figure 55 presents an example of CAPWAP resistance extrapolation analysed for one of the test piles. The first round of dynamic test with CAPWAP analysis conducted 4 days after pile installation indicated a total pile resistance of 3190 kN. The second round of test revealed a pile resistance of 3680 kN. The extrapolated CAPWAP resistance on 28th day after pile installation was 4000 kN. Such analysis was performed for every test pile and created a basis for determination of the soil set-up factor.

The obtained soil set-up factor is shown in Figure 56. The vertical axis shows a pile resistance extrapolated on 28th day after pile installation based on two rounds of CAPWAP measurements. The horizontal axis represents a pile resistance determined by DPDE at the end-of-driving (EOD). It was decided to compare the CAPWAP determined resistance with DPDE at EOD resistance because it was planned to verify all the production piles using that method

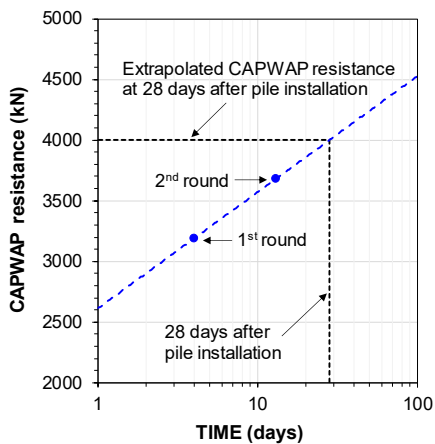


Figure 55 Example of CAPWAP resistance extrapolation.

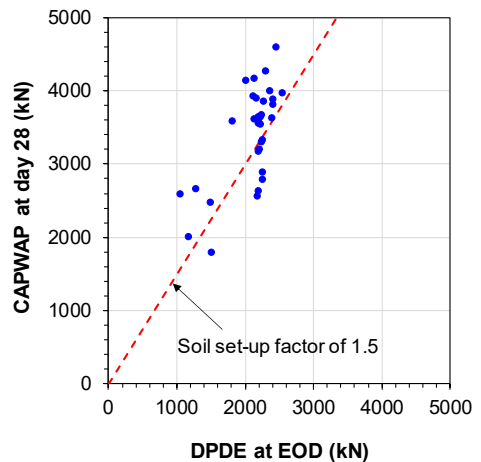


Figure 56 Soil set-up factor.

(i.e. DPDE at EOD). The average soil set-up factor was 1.63. However, it was decided to use a soil set-up factor of 1.5 as indicated in the figure by a red, dashed line. It is important to mention that in average the CAPWAP determined pile resistance was 15 % greater than the pile resistance determined by DPDE. This finding confirms that in case of high pile resistances DPDE underestimates the pile resistance [2].

Expected length of production piles based on test piles

Figure 57 shows the elevation heat map indicating the toe level corresponding to required end-of-driving resistance. The black dots show the location of the

test piles, and the red dots show the location of “dummy” piles. The elevation at which a “dummy” pile reached the required resistance was the same as the closest test pile and they were used to cover the entire pile deck area. As mentioned before the penetration resistance graphs created from the digitized driving journals were analysed in the excel spreadsheet to find the elevation at which the required pile resistance was reached. Using the required pile head elevation, the production pile lengths were calculated adding a robustness of 0.5 m to the so derived pile length. In the final step the pile lengths were rounded up to comply with the manufacturer’s pile length increment of 1 m.

Production pile installation – selected data

The installation of production piles started on 27 March 2023. Table 7 shows selected data from the production pile installation as of 25 March 2024. It is believed that the low percentage of broken piles is an outcome of the extensive pile testing campaign and the derived soil set-up factor. The difference between designed and as-built pile length of 5.3 % is also related to the manufacturer’s pile length increment of 1 m. Assuming an average pile length extension of 0.5 m due to the manufacturer’s pile length increment the difference between designed and as-built pile length could be reduced by almost 2 km.

Figure 58 shows an example of a pile installation overview beneath one of the 20 plates (Plate 4) designed for the project. The coordinate system is DKTM3. The fill of a marker denotes initial assessment of piles installation. Green fill denotes fulfilling the pile installation requirements in terms of pile resistance and placement. A few piles in the south-west corner of the plate had a yellow initial status because the piles were installed up to 0.5 m above the assumed top of the limestone elevation (see Figure 53). Since all these piles reached the required pile resistance and considering the uncertainty of the top of the limestone surface, all the piles were approved, which is denoted by a green border of the marker. Some piles located in the same corner had a red initial status because they were installed more than 0.5 m above the expected top of the limestone. For those piles the soil set-up factor was disregarded and the obtained pile resistance was compared to the required one. As presented in Figure 58 all those piles reached the required resistance even without the soil set-up factor, and, therefore, the green border was used for those piles. Four piles were broken during installation (marked with a cross). Replacement piles were installed at a distance of 2 diameters (centre-to-centre) and assessed as other piles. The same procedure was conducted for all 20 plates.

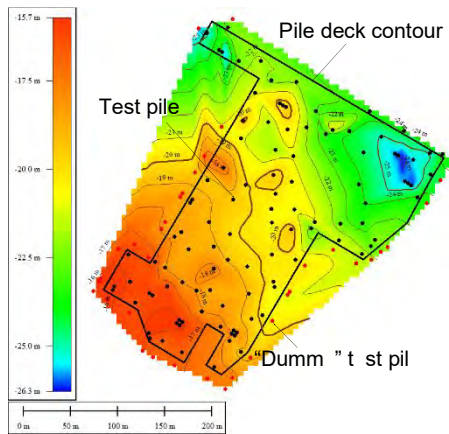


Figure 57 Expected pile toe elevation heat map.

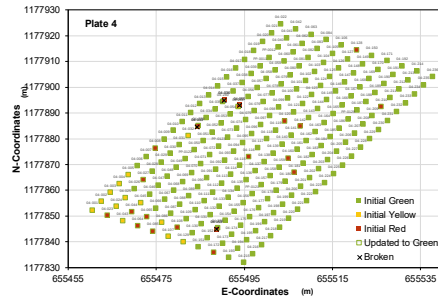


Figure 58 Overview of all piles installed beneath plate 4. Pile number is given above the mark.

Table 7. Selected data from the production pile installation.

Number of installed production piles	3989
Number of broken production piles	70
Percentage of broken piles [%]	1.8
Accumulated design length of installed production piles ¹⁾ [m]	96287
As-built accumulated length of installed production piles ¹⁾ [m]	91193
Difference between design and as-built pile length [m] ([%])	5094 (5.3)

¹⁾ Excluding broken piles

CONCLUSIONS

The following conclusions can be drawn from the present case study:

- The use of commercial and in-house made software greatly enhanced the design approach of large pile deck foundation and improve the efficiency during the execution phase of the project.
- The pile testing campaign allowed the use of a soil set-up factor > 1.5 . This allowed a reduced demand on end-of-driving resistance which greatly reduced the percentage of broken piles during installation as excessive hard driving could be avoided.
- Building and handling big data base requires following predefined procedures and methods by a supplier and receiver of the data.

ACKNOWLEDGEMENT

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<https://byoghavn.dk/nordhavn/nordhavn-udvides/>

RAILWAY ACTIONS ON EMBANKMENTS AND RETAINING WALLS – COMPATIBILITY BETWEEN LOAD MODELS

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KEYWORDS

Railway, Axle Loads, Geotechnical Loads, Load models, 2D/3D Failure

ABSTRACT

Load models for rail traffic on bridges are specified in EN 1991-2 (2003) comprising e.g. LM 71, SW/0, and SW/2 but these are applicable for bridges and not the adjacent geotechnical structures, e.g. embankments and retaining walls. This paper discusses the background for the train load models applied in Denmark for geotechnical structures. Reference will be made to EN 1991-2, EN 15528 and the technical specifications for interoperability (TSI INF). Possible revisions to the existing load models are discussed following the idea that loading on the rails originate from axle groups and that the equivalent line loads may be derived from these groups. Load models for geotechnical structures must therefore reflect an axle load and a line load scenario, but not a combination of axle and line loads within the same model as seen in LM 71.

INTRODUCTION AND BACKGROUND

Railway actions on bridges are covered by EN 1991-2, [1], e.g. load model 71 (LM 71). LM 71, which is shown on Figure 1, does not represent a real train but it includes a combination of load components being critical for local and global stability of bridges. Train loads on neighbouring geotechnical structures must respect EN 1991-2 but shall be carefully established to reflect critical loading aspects for geotechnical structures. The TSI INF requirements, [2], for the European railways state that LM 71 must be used for earth work structures. This is intended to ensure a compliant railway system across Europe, but it does not provide clarity as to how to implement load models for geotechnical structures. The scope of this paper is to present and discuss the approach taken by Rail Net Denmark to establish compliant load models for new geotechnical structures. In addition, suggestions have been put forward to

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discuss requirements in [2] and the approach taken by Rail Net Denmark in their railway standard BN1-59-4, [3] representing supplementary structural and geotechnical Danish requirements to [1] and [2]. Focus for this paper is to present the origin of the load applicable for design of geotechnical structures and to discuss geotechnical aspects related to railway actions.

DANISH LOAD MODELS

The Danish load models for new geotechnical structures are defined by the guidelines set out in [3] from Rail Net Denmark, which offers non-contradictory complementary information to the requirements defined in [1], [2], and EN 15528, [4]. The following rules apply (cf. [3]) for the design of new embankments, new retaining walls and sheet piles:

For stability of embankments i.e. 2-dimensional calculations, single tracks must be subjected to an infinite line load 175 kN/m. Double tracks or more parallel tracks with a line load of 175 kN/m in most critical track, a line load of 110 kN/m in the nearest track, and no load on other tracks.

Calculating the earth pressure on support walls and sheet pilings, including walls for frame bridges, the local increase of loads from boogie-shafts must take account of 3D-effects. Thus, for single tracks locally increased line load 250 kN/m over a 6.4 m stretch is applied and outside this stretch 135 kN/m as an infinite line load. For parallel tracks the loads model for single tracks is applied in the most critical track, combined with a line load 110 kN/m in the nearest track, and no load for other tracks.

The following goes for all line loads: a) They are distributed at the base of the sleepers (2.5 m width), b) Dynamic allowance is included and c) The loads must be used without a reduction in both drained and undrained cases.

The load models above are aligned with the guidelines in [1], where a separation between global and local structural effects is introduced and the geotechnical reflection is likely to distinguish between local failures (e.g. from concentrated axle loads i.e. a 3D case) and global failures, e.g. an average consideration of loads distributed over a length of a train (a 2D case). The proposed geotechnical distinction between local and global failures will align with the intention of [1] and aid the designer in selecting an appropriate design tool. The idea in [3] is that geotechnical load models must be simple to avoid misunderstandings, and that code requirements often can be met using rather simple means.

The basic model for vertical loads in [1] is LM 71 as illustrated on Figure 1 with four axle loads, each representing 250 kN, and with uniformly distrib-

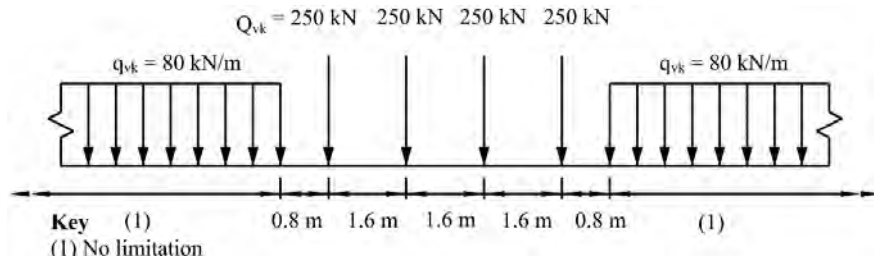


Figure 59. Load model 71 and characteristic values for vertical loads, EN 1991-2, Figure 6.1.

uted line loads outside the point loads. The point loads may reflect two closely spaced bogie axles and are intended to govern the design of local elements, whereas the global load effects are less influenced by these point loads. The distribution is, however, dependent on the loaded length and the size of the structure.

For 2D-analyses (plain strain) e.g. for geotechnical railway embankments, LM 71 has been interpreted based on the following principles, which laid the grounds for the loads presented by [3]: a) The 2D load model should cover the local failure (3-dimensional failure shape) associated with two closely spaced axle groups, b) a purely global failure (2-dimensional failure shape) can be covered by the infinite line load of LM 71 only, c) for loads on double tracks the failure, although considered as a global one, may start as a local failure in one track and develop into a global failure, and d) the 2D line load cannot exceed the line load of LM 71 if the loaded length exceeds 12.5 m (i.e. four point loads of 250 kN divided by 80 kN/m). This agrees with the principles of [4], i.e. corresponding to real rail traffic actions. For 2D-analyses, the load model for heavy trains, SW/2, defined by [1], is considered as an infinite line load corresponding to a global failure mode. In general, the loading presented by [3] is set out as an envelope, covering selected design cases, presented in [1], [2] and [4]. The following subsections illustrate this approach.

The equivalent line loads given in [3] for 2D analyses includes an α -value of 1.33. Compared to the point loads of LM 71, the loaded length assumed to reach the loading of 175 kN/m is 7.6 m ($= 1.33 \cdot 4 \cdot 250 \text{ kN} / 175 \text{ kN/m}$) compared to combined length over the axles of 6.4 m in LM 71 i.e. additional 0.6 m used for distribution of load at both ends of the axle group. Compared to LM 71, also including the line loading when distributing the point load, this load corresponds to an equivalent loaded length $L = 9.6 \text{ m}$ ($175 \text{ kN/m} \cdot 9.6 \text{ m} = (9.6 \text{ m} - 6.4 \text{ m}) \cdot 110 \text{ kN/m} + 4 \cdot 1.33 \cdot 250 \text{ kN}$). Considering the stress distribution through rails, sleepers, ballast, and blanket layer, an equivalent length larger than the 6.4 m from LM 71 is physical reasonable.

For the heaviest reference vehicles in [4] (axle loads of 330 kN, $\alpha = 1.00$ and a line load of 110 kN/m) the equivalent line load of two closely spaced bogies

is $157 \text{ kN/m} [= 4 \cdot 330 \text{ kN} / (2 \cdot 1.8 \text{ m} + 2 \cdot 1.5 \text{ m} + 2 \cdot 0.5 \cdot 1.8 \text{ m}) = 1,320 \text{ kN} / 8.4 \text{ m}]$, i.e. less than 175 kN/m adopted in [3] ($\alpha = 1.33$ included). [1] defines that SW/2, amounting 150 kN/m must be checked, but with a lower partial safety factor. For single-tracked railways this combination is less critical than the load defined in [1]. For railways with multiple tracks, the SW/2 is to be applied in one track, and LM 71 in the other. As the SW/2 is a line load, this situation corresponds to a global (plane strain) failure, which is why the LM 71 approaches a 110 kN/m line load ($1.33 \cdot 80 \text{ kN/m}$).

Based on the discussion above, the Danish rules from [3] complies with requirements in [1], [2], and [4]. Thus, the loading of 175 kN/m envelopes a) LM 71 (156 kN/m or 119 kN/m dependent on approach), b) Heaviest reference vehicle combined with future-proofing (using $\alpha = 1.33$), reaching 157 kN/m , and c) Load case SW/2 required by [1], amounting 150 kN/m .

If the LM 71 axle loads are used for local analyses to investigate 3D load effects on structures, the line load associated with the four axles is $1.33 \cdot 4 \cdot 250 \text{ kN} / 6.4 \text{ m} = 208 \text{ kN/m}$ along 6.4 m . [3] requests to use 250 kN/m along the 6.4 m i.e. approx. 20% higher than what can be derived directly from [1]. The load model for concentrated axle loads shall be used for abutments, walls of frame bridges and wings walls, allowing load spreading of the concentrated axle load through the soil parallel and perpendicular to the track. Parallel to the track, load spreading of the line load outside the 6.4 m is limited 135 kN/m in [3].

ANALYSES OF LM 71 FOR GEOTECHNICAL PURPOSES

The train loads experienced by a geotechnical structure originates from the axle load which is distributed through the ballast and the blanket layers to the subgrade level. Loading on the subgrade level will thus be represented by a series of vertical surcharge loads distributed along the rails, and with each surcharge load area being defined by the footprint of the axle group (plus load spreading to subgrade level). No loading will be seen between the distributed surcharge loads. The consequence of such a load configuration has been investigated by FEM using 2D and 3D analysis of a railway embankment as illustrated on Figure 2. Load spread from base of sleeper to subgrade level has been set to 0.6 m in the analysis while a sleeper width of 2.5 m has been used.

The analysis has been limited to cover one track along a straight line, excluding nosing force for simplicity. The train load is modelled through the load q_{vk1} on Figure 2 (and then multiplied by a partial factor of 1.40) while ballast, blanket layer, sleepers and rails are represented by q_{vk2} rather than discrete soil volumes (partial factor of 1.00). The length B on Figure 2 represents half the width of the subgrade layer, and it has been set to 3.45 m and 4.25 m , which are Danish minimum rules for train speeds below and above 200 km/h , respectively.

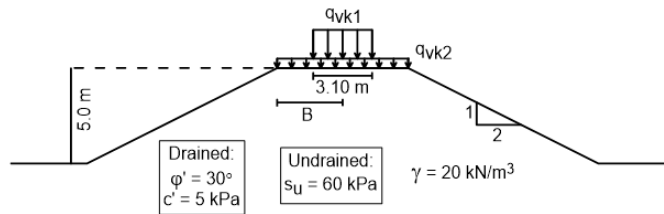


Figure 60. Embankment geometry and characteristic soil properties for a Mohr-Coulomb model. Non-associated flow with $\psi = 0^\circ$ has been used for the drained state.

The cases investigated are defined in Table 1 and are as follows: Case a) A 2D analysis using the 80 kN/m line load from LM 71, Case b) A 3D analysis using one axle group from LM 71, Case c) A 3D analysis using 10 axle groups from LM 71, Case d) A 3D analysis using the Danish line load and Case e) A 2D analysis using the Danish line load.

Table 8. Analysed cases for assessment of load models.

Case	Environment	Design value of surcharge load ($\alpha = 1.33$)
a)	Plaxis 2D	80 kN/m line load from LM 71. $q_{vk1} = 80 \text{ kN/m} \cdot 1.33 / (2.5 \text{ m} + 0.6 \text{ m}) = 34 \text{ kPa}$
b)	Plaxis 3D	Single axle group LM 71: $q_{vk1} = 4.250 \text{ kN/m} \cdot 1.33 / [(6.4 \text{ m} + 0.6 \text{ m}) \cdot (2.5 \text{ m} + 0.6 \text{ m})] = 61 \text{ kPa}$
c)	Plaxis 3D	10 LM 71 axle groups: As case b) above. Edge-to-edge distance of 5.5 m accounting for load spread
d)	Plaxis 3D	175 kN/m line load (DK-approach with $\alpha = 1.33$ included). $q_{vk1} = 175 \text{ kN/m} / 3.1 \text{ m} = 56 \text{ kPa}$
e)	Plaxis 2D	As case d) above but conducted as a 2D analysis.

Convergence analyses were performed to ensure adequate mesh density. The safety of the system was estimated using the ϕ' -c-reduction method leading to a resulting safety factor ΣM_{sf} . Input values for the analysis were the design value of the train load and the characteristic values of the soil parameters. Hence ΣM_{sf} represents a back-calculated partial safety factor on $\tan \phi'$ and c' (or s_u for the undrained investigation).

Figure 2 represents the 2D embankment geometry while the 3D geometry is illustrated on Figure 3, Left (axle group configuration from case c) included).

Back-calculated partial safety factors on the shear strength parameters of the soil are shown in Table 2 for drained and undrained analyses of the five cases investigated. The higher the value, the safer the structure appear. Drained results in Table 2 exceeding 1.32 (or 1.98 for undrained) reflects a safe structure in accordance with EN 1997-1 DK NA.

Table 9. Back-calculated partial safety factors on $\tan\phi'$ and c' (drained materials) and s_u (undrained materials) for the investigated cases.

B [m] on figure 4	Drained materials		Undrained materials	
	4.25	3.45	4.25	3.45
a) 2D, line load (LM 71)	1.59	1.51	2.46	2.45
b) 3D, one set of axle loads	1.80	1.71	3.05	2.99
c) 3D, 10 sets of axle loads	1.75	1.66	2.67	2.65
d) 3D, line load (Danish)	1.67	1.56	2.36	2.37
e) 2D, line load (Danish)	1.49	1.39	2.13	2.11

The conducted Plaxis analyses do not serve as a complete basis for establishing firm conclusions, but the following indications are included.

As the width of the subgrade layer increases (parameter B), the safety level increases, as expected. Comparing one LM 71 axle group [Case b)] with ten axle groups [Case c)] implies that the series of axle groups is more critical than a single axle group. In addition, the line load part of LM 71 [Case a)] implies a lower safety than the LM 71 axle group part [Case c)]. The line load part of LM 71 [Case a)] leads to a higher safety level than found when using [3] [Case e)], which can be explained by the lower line load (110 kN/m versus 175 kN/m). The friction angle of the soil causing failure for cases a) and e) are 20.0° and 21.2° , respectively, which is a small difference given the uncertainty in establishing a characteristic friction angle. The investigated embankment could have steeper slopes, which likely would cause a more severe load effect when comparing Cases a) and e).

The analyses using load models from [3] [d) and e)] yields lower safety factors compared to the LM71 axle models [b) and c)]. This may indicate that the loading set out in [3] is a safe approach, compared to the LM 71 axle groups.

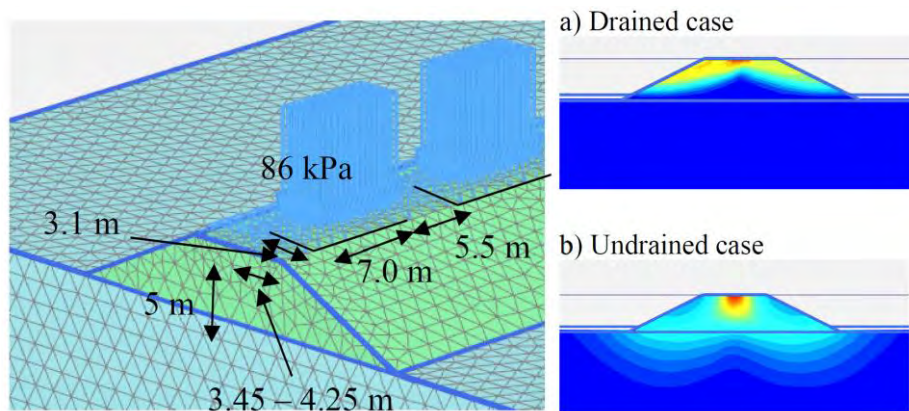


Figure 3. Left: Plaxis 3D model used for evaluation of LM 71 on geotechnical structures ($86\text{ kPa} = 1.4 \cdot 61\text{ kPa}$). Green layer is embankment fill, blue layer is intact

soil. Main dimensions are shown. Right: Failure mechanisms from Plaxis 3D. Cross section taken through axle no. four from the model perimeter.

Finally, line loads applied in 2D will not provide the same safety level as when used in 3D [d) versus e)]. This observation has not been studied at this stage, but the effect of the intermediate effective principal stress component should be investigated.

PRACTICAL DETAILS

The centrifugal forces shall be estimated in accordance with [1] where two fundamental load cases are considered if train velocities exceed 120 km/h: A slower heavy train (freight trains at 120 km/h) and a fast, lighter train (running with the allowable speed along the line). The heavy train in Denmark is modelled following [3], while the light train is modelled as the heavy train divided by α ($= 1.33$). Centrifugal forces will not influence a slope stability analysis provided that the radius of the curvature is sufficiently high, and a radius of 4,000 m seems to be a threshold value. [1] has not ascribed a value to the influence length for the centrifugal force, however multiple Danish railway projects have been designed using an influence length of 10 m for 2D calculations.

A clay embankment shall be investigated for drained and undrained conditions applying the centrifugal force for a curved track. The centrifugal force is linked to the mass of the train and the recommendation therefore align with the single source principle from EN 1990 [8]. Thus, one may argue that the centrifugal force should be included due to the single source principle, when investigating a drained failure in a clay embankment (short load duration indicating undrained conditions). However, disregarding the centrifugal force for drained failure would represent a load case with a parked train in the tracks.

Nosing force is caused by oscillations of train wheels due to their conical shape, and unevenness in rails and tracks, e.g. from attrition. Nosing force is a dynamic load event with a magnitude dependent on the inertia, i.e. mass and velocity. As such, the nosing force constantly changes direction and point of attack. Moreover, the magnitude of nosing will be dissipated through the ballast mat. Thus, for global failures this force is of little magnitude and could be disregarded, which is similar to [1] (Clause 6.4.5.4) stating "... abutments, foundations, retaining walls and ground pressure may be calculated without taking into account dynamic effects."

DISCUSSIONS / CONCLUSIONS

[1] deals with load models for bridges only, while [2] specifies that earth pressure effects shall be specified, applying LM 71. The background for the geotechnical load models from Rail Net Denmark, cf. [3] has been presented

and explained in the context of [1], [2] and [4] to illustrate that load models for geotechnical structures must respect the bridge load models principles.

Based on finite element modelling, it has been shown that geotechnical load models in [3] may be on the safe side but the critical issue with a certain load model may change with the soil type, i.e. clay versus sand, and the full picture is not available at this stage. It seems that concentrated axle loads are less critical than line loads when embankments of a cohesive material are considered, and this observation somehow contradicts the use of LM 71 for geotechnical structures. Rails are loaded by axle loads and an equivalent line load may be derived from these axle loads, but line loads and axle loads acting at the same time is neither physically possible nor suitable in load models for (most) geotechnical structures. It is stressed that the conclusions offered in the present paper is valid for a selected number of cases, and that further analyses must be performed to reach firm conclusions.

The use of $\alpha > 1.0$ may be explained for bridges as train loads historically tends to increase with time and bridges are thus designed with an extra margin to absorb this, but also an increase of robustness on the main railway network in Europe. The shear strength of soil will, however, also increase with time caused by consolidation and compaction effects from repetitive axle passages, and a reduced α -value for geotechnical structures may therefore be considered.

The implementation of geotechnical load models is discussed in EN1991-2 (2023), which is yet to be implemented in Danish design practice.

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RESILIENT MODULUS OF PAVEMENT MATERIAL FROM ORGANIC MATTER ENRICHED PUMICE

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KEYWORDS

Pavement material, Pumice, Organic, Cyclic triaxial testing, Resilient modulus

ABSTRACT

Green areas in connection to densely build up areas are often desired. These can be made, e.g. on top of garage basements or buildings. In order to integrate layers where planting is possible (“growth layers”), and layers with a certain stiffness that would enable some traffic such as e.g. parking, special material is required.

The currently presented materials are made from pumice mixed with organic matter, where the pumice shall provide the required stiffness, and the added organic material shall provide the requisite fertility. Besides having an appropriate stiffness, the pumice also enhances the performance of the growth layer with respect to moisture transport and oxidation of the layer.

Four materials, including clean pumice and pumice enriched with organic matter, are tested in order to determine resilient modulus as a measure of stiffness. The tested material is in variable grain size of up to 20 mm.

In the first, conditioning phase the specimens were exposed to 20 000 cycles. Subsequently, cyclic phases at different cell pressures were performed in order to determine the resilient moduli as a function of overlaying stress and the cyclic load.

The article presents the results and comparison of the cyclic triaxial tests on clean and organic enriched pumice-based pavement materials.

INTRODUCTION

Natural pumice is a porous stone that can soak up to 50 % of water and still include more than 25% of air. While the use of pumice for soil stabilization is well documented in landscaping and horticulture, the material is also used for

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soil improvement in road building (e.g. Çimen et al. 2015 [9]). Based on this, it is presumed that an organic-enriched pumice-based material can have resilient moduli sufficient to sustain light traffic, while being able to support growth of plants with respect to nourishment, moisture transport and oxidation of the layer. This would allow for use of the present material where planting and some traffic such as e.g. parking are simultaneously desired.

The tested pumice-based materials are composed of pumice gravel in variable grain sizes of up to 20 mm, and organic matter. Both clean pumice and organic-enriched materials are tested.

Preliminary testing of the material included limited determinations of grain density cf. DS/EN 17892-3 [1], grain size distributions cf. DS/EN 17892-4 [2] (proprietary data), and maximum dry density and water content by means of vibration table cf. DS/EN 13286-5 [3].

Mineralogy of the tested pumice has not been determined. In general, pumiceous sands and gravels are siliceous materials with lower specific gravity than silica sands, typically up to 2.5 (Asadi et al. 2017 [5], Pender et al. 2006 [8], Chaney et al. 2001 [6]). Specific gravity of the presently tested clean pumice is 2.38. It is underlined by e.g. Chaney et al. 2001 and Pender et al. 2006, that due to the closed porosity of pumice, typical grain size and the very procedure of testing may influence the test value.

Vibration table test of clean pumice at full saturation resulted in a soft, low density, structureless specimen. The test at natural water content resulted in a firm, high density specimen. Specimens for subsequent cyclic triaxial testing are therefore built in by tamping the material at natural water content to 95% of the maximal dry density determined in the vibration table tests.

The grain size distribution testing prior to, and after the vibration table test on clean pumice showed an increase in the content of fines from about 4% to about 10%, attributed to particle crushing.

One cyclic triaxial test was performed on material that was also including a fraction of scoria. Scoria is material that is typically denser and with a higher strength than pumice, with larger pores and thicker pore walls. Variety of uses of scoria for e.g. landscaping, horticulture and roadbeds is documented by Dehn and McNutt 2015 [7].

Finally, it should be noted that the tests were carried out over a span of several years and without a scientific purpose.

CYCLIC LOAD TRIAXIAL TEST FOR UNBOUND MATERIAL

Testing program and sample conditioning

Four drained cyclic triaxial tests were conducted on materials summarized in Table 1 in order to determine resilient moduli. Note that Test 2 has been rerun in the resilient phase due to technical issues.

Table 1. Overview of materials for drained cyclic triaxial testing

Test no.	Material	Fraction [mm-mm]	Organic content [%]	Dry density [g/cm ³]	Saturation [%]	H/D [mm/mm]
1	Clean pumice	0-10	0	0.84	100 ^a	140/70
2	Pumice with organic content	0-18	15	0.59	n.a.	200/100
3		10-20	10	0.51	100 ^a	200/100
4	Pumice with scoria and organic content	0-7 (scoria) 10-18 (pumice)	10	0.72	49 ^b	200/100

^aEstimated and referring to the open pores space.

^bCalculated based on the material crushed after test.

Conditioning of specimens was carried out at the effective axial stress of about 170 kPa and constant cell pressure (confining stress) of 70 kPa; 20 000 cycles was carried out with the cyclic stress deviator of 200 kPa corresponding to the low stress level of the Method B of DS/EN 13286-7 [1]. Characteristic permanent axial strain cf. Annex C of DS/EN 13286-7 is presented in Table 2.

Table 2. Characteristic permanent strain based on the conditioning phase

Test no.	Permanent axial strain		
	end of conditioning, $\epsilon_{1P(20000)}$ [%]	after 100 cycles, $\epsilon_{1P(100)}$ [%]	characteristic $\epsilon_{1c} = \epsilon_{1P(20000)} - \epsilon_{1P(100)}$ [%]
1	1.50	n.a.	<1.50
2	1.24	0.61	0.63
3	1.98	0.72	1.26
4	2.89 ^a	2.37 ^a	0.52

^a>1 % occurring during the first cycle of conditioning

Following DS/EN 13286-7, materials 1, 2 and 3 have been validated for repeated loading for resilient testing as the axial strain of 2 % was not exceeded in the conditioning phase. In the Test no. 4, a significant portion of axial strain ($>1\%$) occurred during the first conditioning cycle. This was considered to be an effect of bedding, and consequently, the material is validated for resilient testing.

Study of resilient modulus

The testing was conducted principally following the Constant confining pressure procedure (Method B of DS/EN 13286-7). At each confinement stress, 100 cycles were applied at each of several cyclic deviator stress levels (hereafter “series”). In order to comply with 100 cycles requirement, Test 2 has been rerun after initial series at 20 and 35 kPa confinement stress that were carried out with 41 to 47 cycles. An example of stress conditions for all series in Test 1 is plotted against double amplitude cyclic axial strain at the end of each series in Figure 1. Double amplitude cyclic axial strain and cyclic stress deviators are defined in Figure 2.

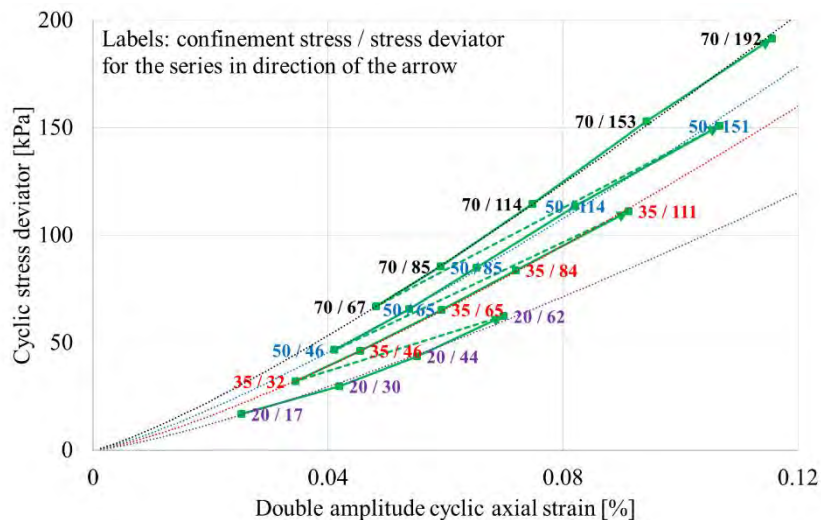


Figure 61 Testing levels and resilience example from the test on clean pumice (1)

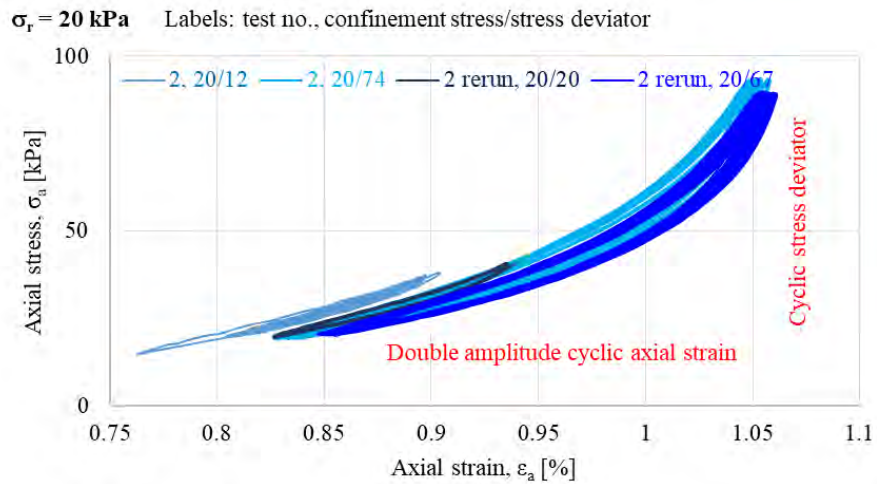


Figure 62 Possible effect of rerun depicted on selected series within Tests 2 and 2,rerun.

Following the standard, all four tests have been carried out at the confinement stress levels of 20, 35, 50 and 70 kPa, see Figure 3. In addition, Test 4 has been carried out at confinement stress levels of 100 and 150 kPa. All the series carried out at constant confinement stress levels are marked with arrow lines in Figure 3. Double amplitude cyclic axial strain in Figure 3 is measured in the last cycle of every series against the applied cyclic stress deviators. Upon the last cyclic state at constant confinement pressure, the tests continued at the next confinement state; this is indicated with dotted lines in Figure 3.

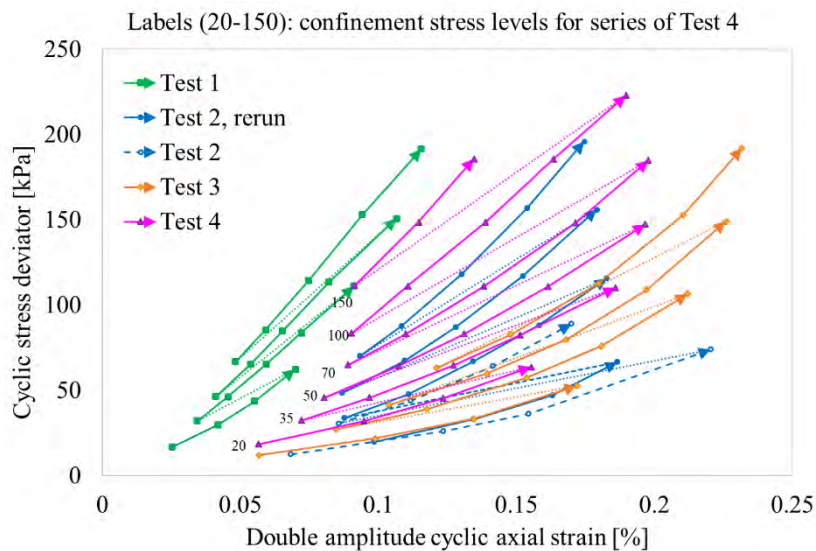


Figure 63 Simplified stress strain paths.

RESULTS AND DISCUSSION

As depicted in Figure 2, the stress strain response during the resilient cycling is typically nonlinear, with a small hysteresis. Similar is observed both for clean pumice and pumice enriched with organic material.

Resilience for the clean pumice (Test 1) can be seen on Figure 1 showing the final stress-strain states for each of the series in the space of cyclic stress deviator and double amplitude cyclic axial strain. Figure shows that all strain development curves originate at 0, indicating that the full unload of specimen to no confinement and no cyclic deviator state results with full reversal of strains, thus confirming resilience.

Resilient modulus, E_r , is defined as a ratio of cyclic deviator stress to resilient strain. The resilient strain is a measure of deformation occurring during elastic loading, i.e. fully reversible. The variation of E_r within the space of the elastic loading and occurring resilient strain may take any trend while depending on the load level or other factors causing elastic nonlinearity. E_r is determined as a function of confinement pressure and the cyclic stress deviator and presented in Figure 4.

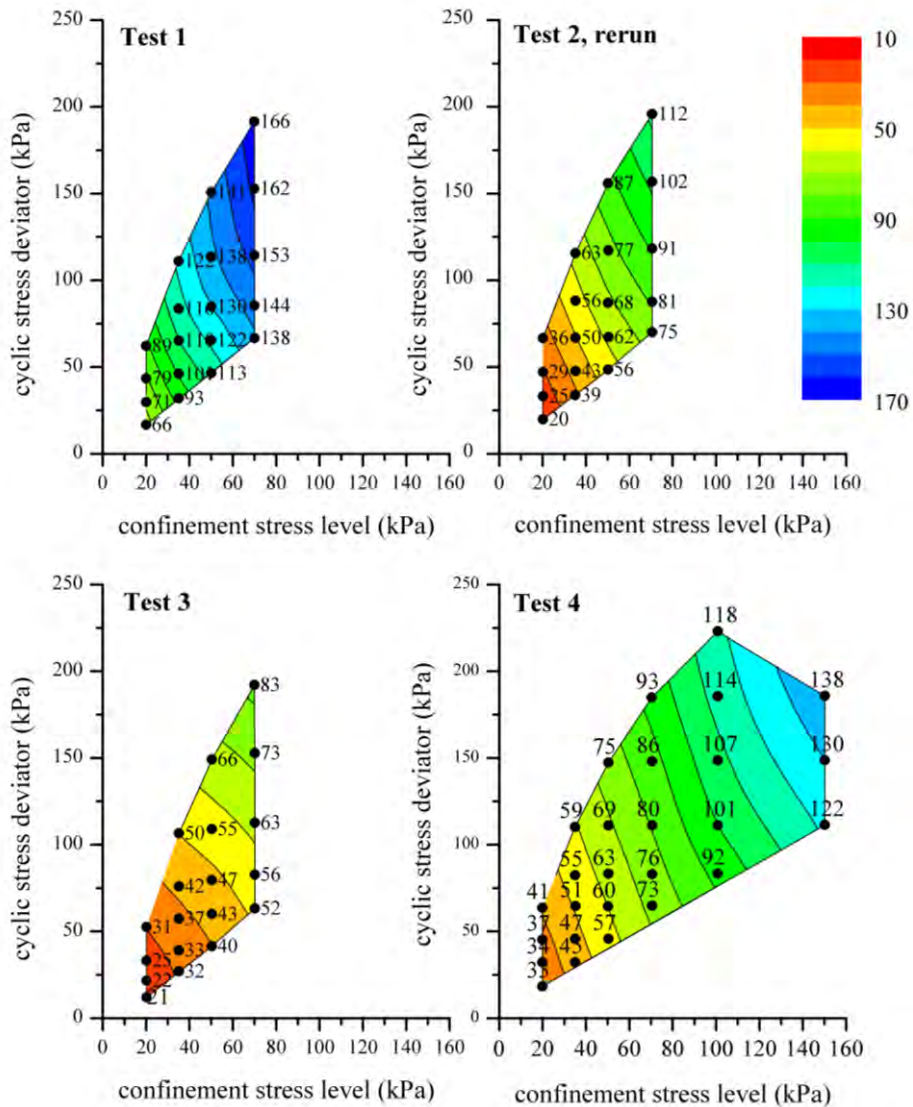


Figure 64 Resilient moduli contours as function of cyclic stress deviator and confining stress level. Labels: E_r test results

The reduction of E_r increase with increasing confining pressure in clean pumice (Test 1) is found in overall agreement with the results of Asadi et al. 2017 [5].

A comparison of the four materials is presented in Figure 5, per confinement stress level. The clean pumice in Test 1 has the highest stiffness. For all the tests with organic material, E_r is significantly lower. At higher confinement stresses, the nonlinearity of E_r against the cyclic deviator stress is the highest

in clean pumice. The lowest increase of E_r with cyclic stress deviator is seen in pumice enriched with scoria and organic matter (Test 4).

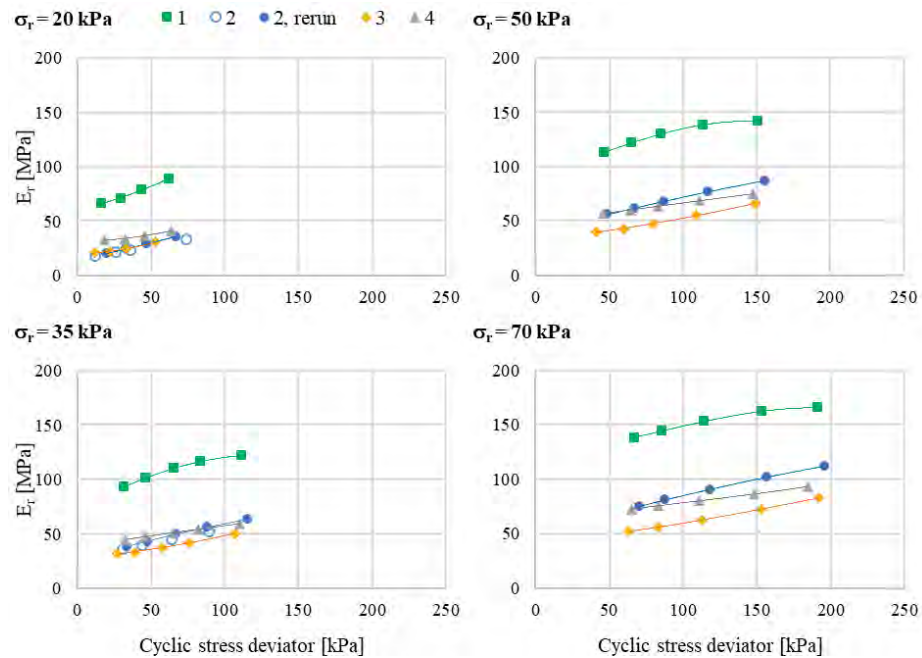


Figure 65 Resilient moduli as function of cyclic stress deviator per confining stress level, σ_r , ranging from 20 to 70 kPa

A possible effect of a rerun is observed on a medium coarse specimen with 15 % organic material (Test 2 and Test 2, rerun). Figures 2 and 3 show that the moduli in the series at 20 and 35 kPa confinement increased in the rerun, possibly at the expense of increased average axial strain. However, the evaluation of E_r may also be influenced with the lower number of cycles in the series of the original run of Test 2.

CONCLUSIONS

The four presented tests showed that the testing standard DS/EN 13286-7 is generally applicable for testing of resilient modulus on clean pumice gravel and pumice enriched with organic material.

Testing showed nonlinear stress strain response in resilient loading cycles, and confirmed the resilience of cyclic strain.

The highest E_r moduli are evaluated for clean pumice gravel, which is finer than the gravel used in samples enriched by organic materials. At low confinement stress of 20 kPa, there is no significant difference in E_r between the Tests 2 and 3, whereas Test 4 on the material enriched with scoria showed

higher moduli. Generally, the lowest E_r moduli are evaluated for the coarsest material (Test 3).

Characteristic permanent strain based on the four tests generally classifies materials in Class C3 cf. DS/EN 13286-7 Annex C.

ACKNOWLEDGEMENT

Authors are thankful to Byggros for allowing the presentation of the test results.

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SECANT PILE WALL FOR CONSTRUCTING THE NEW METRO STATION SOFIA IN STOCKHOLM

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KEYWORDS

Deep excavation, Ground Conditions, Secant Pile Wall, Rock Socket, Ground Anchors, Large Boulders

ABSTRACT

Station Sofia in Södermalm, Stockholm, Sweden is part of the expansion of the new blue metro line towards Gullmarsplan (southbound) and Nacka (eastbound).

Retaining walls were necessary to support the excavation works for the rock foundation of the station building. The retaining walls consist of a drilled soldier pile wall above the groundwater level and a watertight secant pile wall below the groundwater level. Both walls are rock anchored with self-drilling ground anchors.

Construction of Station Sofia also includes the rock excavation for an approximately 90-meter deep elevator shaft (20x20 m) down to the station space in the rock. Consequently, the station becomes one of the deepest in the world.

This paper reports on the construction of the secant pile wall, which consists of 90 overlapping bored piles, Ø 1200 mm, with lengths ranging from 6 m to 20 m. The excavation depth is approximately 19 m, of which about 15 m are below the groundwater table. The secant pile wall has been embedded into the rock and has up to two anchor levels work with anchor grout lengths in rock ranging from 4 to 5 m.

The ground conditions have been extremely challenging, with extensive large boulders, significant soil depths, groundwater-lowering measures, a strongly inclined rock surface, and a weakness zone, significantly impacting the difficulty of executing the secant pile wall.

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Despite the challenges, it can be concluded that a continuous and robust secant pile wall with the required watertightness has been achieved.

INTRODUCTION

Station Sofia is part of the expansion of the new blue metro line towards Gullmarsplan and Nacka. See Figure 66 and Figure 2. Owner of and responsible for the Stockholm's new metro is Region Stockholm.



Figure 66 Location and cross-section of the new metro station Sofia.

The contractor Implenia has been responsible for various tasks, including the construction of retaining walls, earth and rock excavation for the foundation of the new station building, and the rock excavation for an approximately 90-meter deep elevator shaft (20x20 m) down to the station space in the rock.

The type of contract for the retaining walls is design and build.

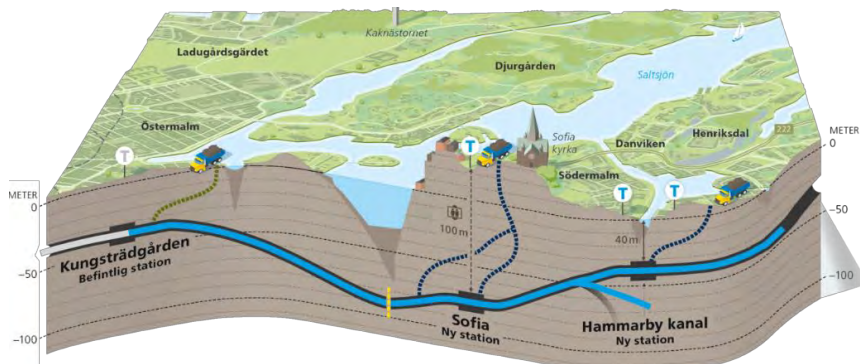


Figure 67 Profile of the eastbound part of the new blue line with Station Sofia.

GROUND CONDITIONS

Topographic conditions

The immediate surroundings consist of an urban park environment surrounded primarily by streets and old buildings.

Soil conditions

The soil profile generally consists of fill on clay underlain by frictional soil and moraine on bedrock. In the northern part of the area, bedrock is exposed.

The thickness of the fill varies between 0.5 and 4 meters and consists mainly of frictional soil, boulders, building debris, and organic material.

The clay layer varies in thickness between 0 and 3 meters.

The thickness of the friction soil varies across the area between 0.5 and 12 meters and consists of silty sand transitioning to gravelly sand.

The thickness of the moraine reaches up to approximately 13 meters. The moraine is sandy/silty and is assessed as firmly deposited.

Both boulders and large boulders have been encountered at various depths in the soil profile.

According to the soil-rock investigations, the depth to estimated bedrock varies between exposed bedrock (outcrop) in the north to about 20 meters in the south. The bedrock surface generally slopes at an average of 45° in a north-south direction. See Figure 3 for point of the compass.

Rock conditions

The bedrock in the area is predominantly medium-coarse grained sedimentary vein gneiss, which transitions to coarse-grained granite at depth.

Along Folkungagatan (Figure 3 and Figure 12), there is also a longitudinal weakness zone with a RMR_{bas} -value ≤ 30 , indicating poor to very poor rock quality.

Hydrogeological conditions

Within the work area, there is only a lower groundwater aquifer in the frictional soil beneath the clay. The groundwater pressure level varies between +15.5 and +17.5.

The groundwater generally flows through the area in an eastward direction along Folkungagatan and further descends towards “Saltsjön” in the east. The hydraulic gradient is moderate within the working area.

A groundwater recharge also occurs through precipitation from higher elevations in the north down towards Folkungagatan (south side). See Figure 3.

Environmental conditions

Within the working area the soil has been assessed to be contaminated down to level +19. The groundwater has also been assessed to contain contaminants.

DESIGN OF THE SECANT PILE WALL

To perform excavation works down to the foundation level of the station building and for the construction of the elevator shaft in rock, two types of retaining walls has been required along three sides of the foundation pit.

No retaining wall has been required on the northern side due to exposed bedrock (outcrop), which however constitutes a rock slope ≥ 20 m. See Figure 3 and Figure 4.

The size of the pit is 35 m (west/east) times 45 m (north/south).

Additionally, a watertight retaining wall was partly required to minimize the risk of impact on groundwater-sensitive objects in the adjacent surroundings.

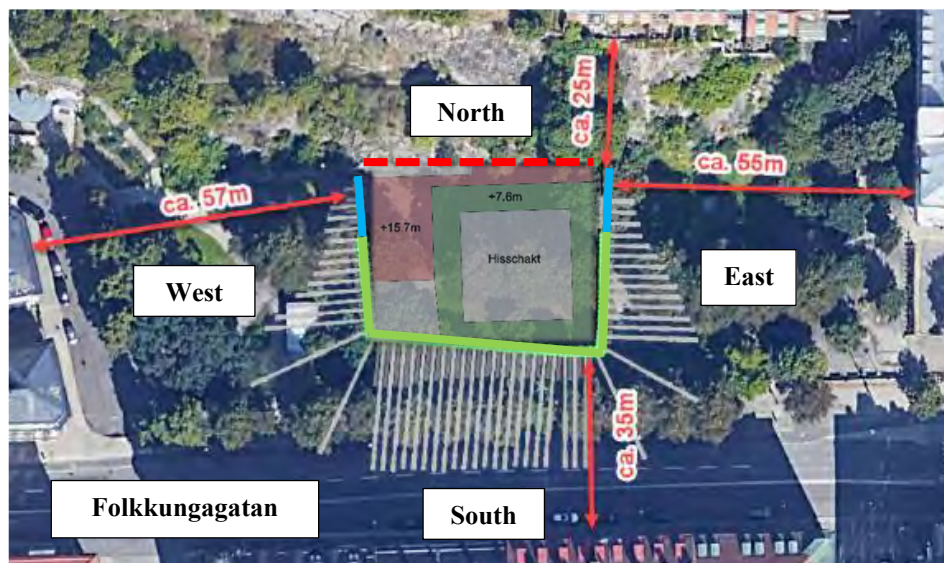


Figure 68 Lay-out of the secant pile wall (green) and soldier pile wall (blue), as well as the rock slope along the north side of the pit (red dotted line).

Based on the ground conditions, which necessitates a drilled solution, and the need for ground water lowering during excavation along with a high requirement for watertightness down to the bedrock a secant pile wall was chosen.

The secant pile wall is designed according to Geotechnical category 3 (GK3) and safety class SK2. The design life is 10 years.

The secant pile wall consists of 90 bored piles with lengths ranging from approximately 6 to approximately 20 meters, see Figure 4.

In Figure 4, the extent of the secant pile wall and the rock foundation levels for the station building, as well as the location of the elevator shaft, are shown.

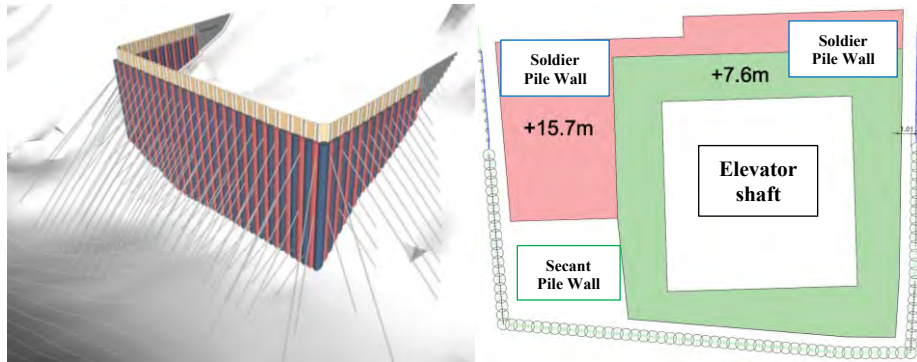


Figure 69 Lay-out of the secant pile wall, the soldier pile wall and rock foundation.

The secant pile wall consists of bored piles, $\varnothing$ 1.2 m, with center-to-center spacing of 0.9 m. Every secondary pile is reinforced. The overlap between each pile is 0.3 m (Figure 5).

Each pile is embedded in the bedrock to ensure a watertight connection to the bedrock surface. Additionally, every secondary pile had a minimum rock socketing depth of 0.3 m ensuring the entire cross-section is fully engaged to handle both vertical loads and horizontal earth pressure. See Figure 5.

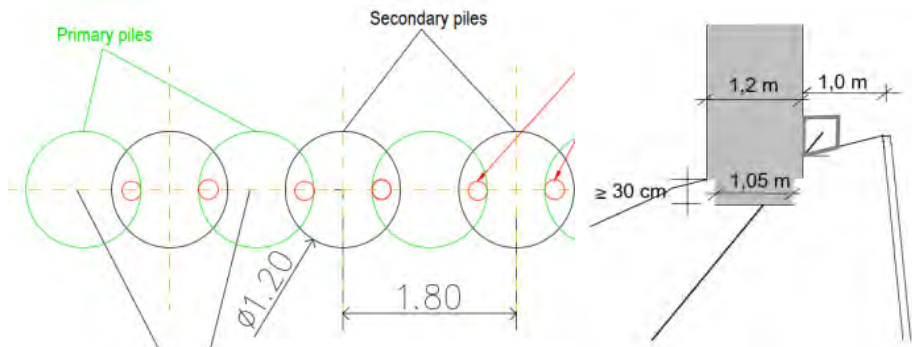


Figure 70 Left - lay-out of the secant pile wall including steel casing in each secondary pile for further curtain grouting. Right - requirement for rock socketing.

Ground anchors

The secant pile wall is anchored with tie-backs in bedrock with up to two levels according to Figure 4 and Figure 13. The bond lengths in rock vary between 4 and 5 meters.

The ground anchors consist of self-drilling Ischebeck Titan rods with dimensions of 103/72, 103/51, and 103/43. The prestressing load varies between 900 kN and 1435 kN.

Sealing demands

Sealing demands for the secant pile wall have only been specified as the maximum allowable groundwater drawdown during a pumping test, which needed to be approved by the Owner before pumping of groundwater could commence.

EXECUTION OF THE SECANT PILE WALL

The ground conditions have been extremely challenging, significantly impacting the difficulty of installing the secant pile wall. Challenges include drilling into steeply inclined rock, drilling through both boulders and large boulders (>630 mm), and meeting drilling tolerances to ensure overlapping secant piles from top to bottom.

The secant piles have been installed through rotary drilling, minimizing environmental impact on the surroundings compared to percussive drilling.



Figure 71 Picture showing time for installation of anchor level 2 as well as ongoing raise boring for the elevator shaft.

Considering the sloping rock surface, a pile test was conducted to optimize the approach for installing each secant pile with the required rock socketing in sound rock (Figure 5). Despite the pile test, achieving the necessary rock

socketing has been one of the major challenges in installation, both in terms of time, verification, and with extremely high costs for rock drilling.

However, the results after excavation have shown that all secondary piles are properly rock socketed with an average depth exceeding 0.3 meters. The high likelihood that all secondary piles were embedded in sound rock before rock excavation has also been verified during drilling for curtain grouting, via the pre-installed steel casings (Figure 9).

Furthermore, drilling for tie-back anchors, besides facing the same difficult soil conditions, posed challenges in determining the transition between soil/rock and drilling and anchoring in the weak zone along Folkungagatan (Figure 13).

Drilling rigs and drilling equipment

For the installation of the secant pile wall, two drilling rigs have been used as follows.

One drilling rig from the manufacturer Liebherr, LB36-410, equipped with a Kelly-system (telescopic drill string) for rotary drilling with casings down to the rock surface. The machine has a mast height of approximately 26 meters and weighs around 115 tons.

After the installation of the casings a second drill rig have executed the socketing. For rock socketing a drilling rig, BG28, from the manufacturer Bauer, equipped with an air-powered down-the-hole hammer (DTH), Ø 1050 mm, has been used. The machine has a mast height of approximately 22 meters and weighs around 84 tons.

Considering the extent of frictional soil, casings (outer diameter 1220 mm and inner diameter 1180 mm) have been required to stabilize the borehole, thus enabling concreting of each secant pile.

Tools for excavating inside the casings have included various types of augers and drilling buckets. DTH drilling has also been required to penetrate or break up large boulders inside the casings, if necessary, that could not be excavated with standard tools. See Figure 7 below regarding the extent of boulders.



Figure 72 Pictures showing the extent of boulders and of large boulders (>630 mm).

Sequence of installation

In connection with a secant pile wall, the unreinforced primary piles are first constructed, followed by the reinforced secondary piles, see Figure 8.

Concrete quality for the primary piles is C12/15 and for the secondary piles is C30/37.

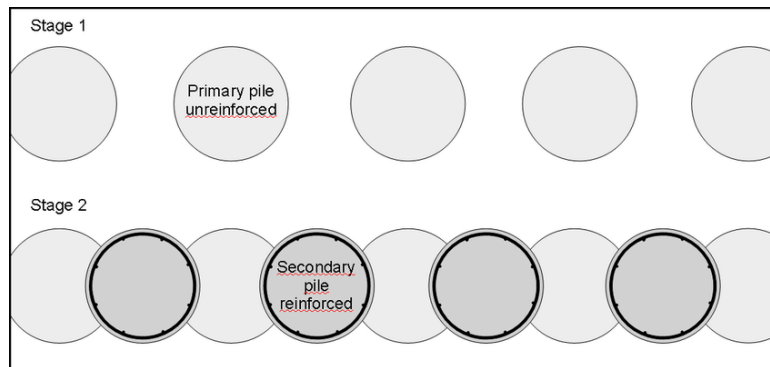


Figure 73 Installation stages for the secant pile wall.

Reinforcement cages

The reinforcement cages have been prefabricated and delivered to the construction site. Cages with a total length exceeding 12 meters were delivered in two parts (upper cage/lower cage), which were assembled on-site before installation. See Figure 9.

Additionally, welded spacers were attached to the reinforcement cage to ensure its verticality and centering within the casing before concreting, as well as to meet the requirements for concrete cover.



Figure 74 Pictures showing reinforcement cages incl. steel casing for curtain grouting.

Tremie concreting

The piles have been cast using ready-mixed concrete of prescribed quality for both primary and secondary piles. Casting was performed using underwater tremie concreting method, employing a system consisting of jointed steel pipes, Ø 220 mm, in sections of 3 m.

After cleaning the borehole, the tremie pipe was lowered to the bottom of the secant pile. Subsequently, the pile casting was carried out by gradually raising the tremie pipe and simultaneously extracting the casing as concrete was poured in.

Drilling tolerances

To achieve a continuous and watertight secant pile wall, the following drilling tolerances have been required for the construction of the secant pile wall:

- Collaring deviation ± 30 mm.
- Drilling and angular deviation 1.0%.

To facilitate and ensure the alignment of each secant pile, a concrete drilling template has been constructed prior to installation. See Figure 10.



Figure 75 Pictures showing a typical drilling template for aiding alignment.

During construction, each secant pile has been surveyed at the pile top and checked for drilling deviation through inclinometer measurements.

The results show that the drilling tolerances have been achieved (Figure 10).

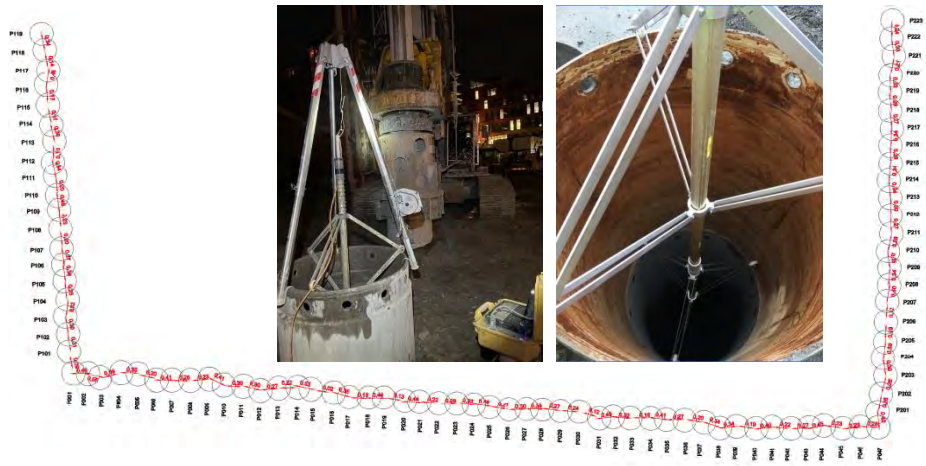


Figure 76 Result of deviation measurements at hole bottom, showing 100 % overlap.

Ground anchors

To install the anchors, it is first necessary to drill two holes through each secondary pile. The holes are made using core drilling, as shown in Figure 12.

Each anchor has been installed according to SS-EN 1537. Drilling was carried out using top hammer drilling with water flushing, and no casings were used.



Figure 77 Left and middle– holing in each secondary pile for further anchoring. Right- ongoing drilling for anchoring.

The anchors are grouted into the rock with a theoretical volume (plus 10 %) with a cementitious grout, w/c ratio of 0.45, and shrinkage-reducing admixture. Type of cement CEM II 42.5 R. Grouting has been made via the anchors.

Acceptance testing of each anchor (method 1) was conducted according to SS-EN ISO 22447-5:2018. All anchors have passed the acceptance test.

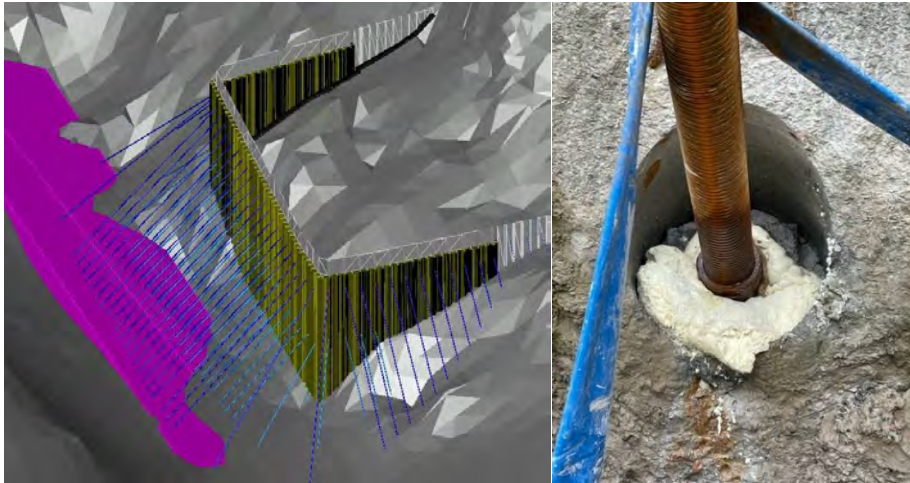


Figure 78 Left – assessed location of the weakness zone (magenta). Right – permanent sealing of the anchor hole using polyurethane resin.

For anchor level two, sealing against groundwater ingress through the tie-back penetrations in the secant pile wall has been performed both temporarily during drilling and permanently after acceptance testing. Permanent sealing of leaking tie-back holes has been done using polyurethane resin, see Figure 13.

Geotechnical monitoring

Monitoring (Figure 14) during installation and excavation has shown ground movements of approximately 1–5 mm, manhole covers approximately 1–3 mm, and in the street of Folkungagatan approximately 1 mm. Settlements of around 1–2 mm has been measured at block Buketten 15 to 18 (groundwater-sensitive buildings).

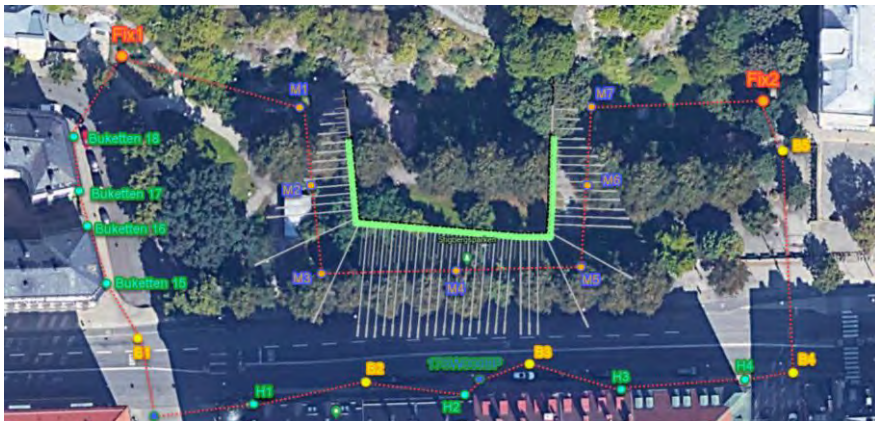


Figure 79 Scope of measuring points for geotechnical monitoring.

Thus, the results of geotechnical monitoring have shown that no harmful impact on the surroundings has occurred, as well as no impact on the ground water levels.

CONCLUSIONS

Despite challenging ground conditions, it can be concluded that a highly robust and continuous secant pile wall has been achieved with the required watertightness as shown in Figure 15 and Figure 16 below.

It has taken approximately a year to install the secant pile wall, which clearly reflects the difficulty posed by the ground conditions. Furthermore, no adverse effects on the surroundings have occurred, neither in terms of ground movements, settlements, nor groundwater.

Executing the secant pile wall considering the difficulty, requirements, and ground conditions has been an unparalleled challenge. However, with great dedication and fighting spirit from all parties involved, a fantastic result has been achieved.

A secant pile wall is an extremely unbeatable technical solution when the ground conditions demand a drilled retaining wall and with high demands on watertightness.



Figure 80 Secant pile wall, facing east and south.



Figure 81 Secant pile wall, facing south and west, rock slope facing north.

ACKNOWLEDGEMENT

We'd like to acknowledge Region Stockholm for letting us present this paper.

SETTLEMENT ANALYSIS OF EMBANKMENT USING FOAMED GLASS AGGREGATE OVER A SOFT CLAY

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KEYWORDS

Foamed glass aggregate, bulk lightening, Plaxis, Soft soil creep, Mohr coulomb. Light weight fill.

ABSTRACT

Analysis of settlements under any structure is very crucial in preventing future damages or failures. This paper investigates the consolidation settlements in the soft clay layer during and after constructing embankments and its stability using a layer-wise foamed glass aggregate (lightweight fill) with Engineered fill over existing fill. Foamed glass aggregates are produced from cleaned recycled glass and have multiple uses, including bulk lightening, frost-heave insulation, dry layering, etc. This material has a bulk density of 4 kN/m³ and granular size ranging from 10-60mm. Finite element analyses were carried out to evaluate the consolidation settlement of soft clay and the stability of the embankment. Advanced models were used for soft clay as a soft soil creep model and Mohr-Coulomb for sandy soil/lightweight using 15-noded triangular elements in Plaxis software. The results show that there is a substantial reduction in consolidation settlement of the soft soil layer by using lightweight fill (Foamed glass aggregate) layer-wise during embankment construction.

INTRODUCTION

Lightweight fill or foamed glass aggregate replacement is a popular method to reduce the vulnerability of additional loads in the presence of soft, compressible soils or utilities. Lightweight fill is frequently used in place of traditional soil or aggregate to minimize settling, safeguard utilities, or lessen driving forces on slopes or walls. Today's market offers a wide range of lightweight fill materials, such as expanded shale, foamed concrete, shredded tires, lightweight aggregate made of clay or slate, geofoam, and now foamed glass aggregate (FGA).

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Foam glass aggregate, which ranges in size from 0 to 60 mm, is similar to pumice and is utilized as a lightweight aggregate in the construction and infrastructure sectors. Glass cullets that are no longer needed are combined with a foaming ingredient, usually silicon carbide, to create foam glass aggregate. "Reject glass" produced at glass recycling facilities is kept out of landfills via the manufacturing of foam glass. There is no organic matter in the foam glass product because any organic matter contaminants in the raw material burn throughout the production process. The pros and disadvantages of these low-weight materials include their recycled content, sustainability, durability, flammability, buoyancy, permeability, ease of installation and time, availability, and cost. They are also available in a range of unit weights. Figure 1 shows the standard product of FGA for geotechnical applications.



Figure 1 Foamed glass aggregate

Technical Properties of Foamed glass aggregate:

The unit weight of foamed glass aggregate is relatively low with a high friction angle, making it ideal for embankments. The technical characteristics of Hasapor foam glass (FGA) are presented in below Table 1.

Table 1. Technical properties of Foamed glass aggregate.

Light fillings/frost insulation	Test method	Size
Grain size	NS-EN 933-1	10-60 mm
Dry density	NS-EN 1097-3	180 kg/m ³
Grain density, prd	NS-EN 1097-6	380 kg/m ³
Resistance to crushing (at 20% compression)		770 mPa
Design specific gravity in fill	NS-EN 1097-3	3.0 kN/m ³
Design weight density against uplift	NS-EN 1097-3	2.2 kN/m ³
Capillary rise height	NS-EN 1097-10	170 mm
Compression Volume Change	NS-EN 1097-10	15-25 %
Friction angle		45°
Thermal conductivity (dry)	NS-EN 12667	0.097 W/mK
Thermal conductivity (wet, drained)	NS-EN 12667	0.107 W/mK
Frost/Resistance	EN 13055-2	0.2 %

This paper will present the FEM analysis of the settlement and stability performance of the embankment considering the placement of foamed glass aggregate in the embankment area above Fill material.

DESCRIPTION OF THE AREA

The five-mile-long railway stretch for goods routes through great importance for freight transport. Today, the railway is heavily loaded, and the tracks are crowded. The risk of disruption on single-track sections is high. It is expanding so that more trains can run on the track and transport times will be reduced. With increased capacity, more traffic can be transferred from road to rail. It has positive effects on the environment because carbon dioxide emissions are reduced. The planned area where the analysis is carried and presented in this paper as shown in Figure 2.



Figure 2 Planned construction area

The planned construction for the area illustrated in Figure 2 involves the chainage km 4+180, where a retaining wall is planned on the southern side to separate the existing entrance group's tracks from the new double track and the new double track continues the southern side of the existing freight line, expanding the embankment up to 20 meters to the south. After the construction of the retaining wall, the leveling by filling with Engineered Fill material resulted in increased settlement in the area, so it is planned to replace the top layer of Engineered Fill with foam glass aggregate to reduce the settlement of the embankment.

SOIL CONDITIONS

The stratification of this area consists of filled up soil ranging from 2-3m and silty Clay/clayey to a thickness ranging from 1 to 10m, followed by sand and silt. At this location, sandy silty/silty Sand is underlain by weathered rock. The clay soil is normally consolidated with an OCR of 1-1.2 and undrained shear strength of 10 to 15 kPa. A typical CPT profile and undrained shear strength plot is presented as shown in Figure 3.

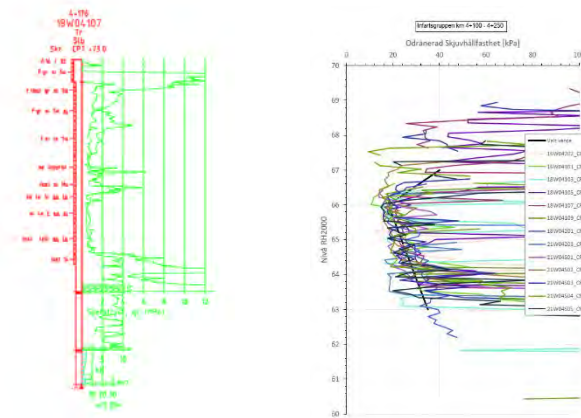


Figure 3. Plot of CPT and Undrained shear strength.

SOLUTIONS

Regular Engineered filling was out of the question because it would have increased the settlement, increase in lateral earth pressure on retaining wall and cause stability problems. These factors led to the use of light weight fill to reduce the weight of the embankment on the subsoil.

Light weight fill was selected due to its technical and structural attributes, as well as its suitability for use and recycling. The low unit weight of foamed glass reduced the loads on the subsoil, and the combination of low unit weight and high friction angle reduced the stresses on the retaining walls and reduces the settlement.

NUMERICAL ANALYSIS ASSUMPTIONS

Settlement calculations have been performed using the FEM-software PLAXIS 2D, version 2023.1.0.136 with cross sections. For settlement calculations in the longitudinal direction, PLAXIS assumes the same conditions as in the transverse direction regarding soil layers and reinforcement measures.

The settlement model is used to assess total settlement, lateral displacements, and differential settlements during construction and the permanent stage.

Material Properties:

The Mohr-Coulomb model is accepted as a first-order approximation of real soil behavior for frictionless soils. This elastic-perfect plastic model requires five basic input parameters: Young's modulus (E), Poisson ratio (ν), cohesion (c), friction angle (ϕ), and dilatation angle (ψ). The properties of the soils used in this analysis are presented in Table 2.

Table 2. Material Properties.

Identification	Lera 1 Z>65 SSC(A)	Lera 2 Z>65 SSC(A)	Silt / Sand above clay	Friction Soil beneath the clay	Existing Fill	Engineer ed Fill	Foam Glass Aggregate (10-60mm)
γ_{sat} , kN/m ³	17.5	18.5	18	20	20	20	4
E'_{ref} , kN/m ²	-	-	5000	20000	20000	40000	3333
ν	-	-	0.3	0.3	0.3	0.3	0.3
c' , kN/m ²	1	1	1	1	1	1	1
ϕ'	30	30	30	34	35	40	44.5
ψ'	0	0	0	4	5	5	0

CALCULATION MODEL

Settlement analyses for the embankment (including construction sequence) and earthworks were carried out using the finite element program, PLAXIS-2D version 23.01.00.136. The initial stress conditions were modeled using the in-situ type analysis mode. Stress distribution and deformations of the earthworks were modeled using the Mohr-coulomb (perfect linear elastic-plastic) soil model for granular and cohesionless materials. Clayey soil is modeled as a soft soil creep model. The existing fill (post-filling), and the Engineered fill with FGA were constructed assuming layered fill with varying thicknesses (1.5 to 2m) with different pause periods.

The construction sequence and analysis stages considered in the models is shown in Figure 4.



Figure 4 Construction sequence and stages

RESULTS

The finite modelling analysis section with final construction phase replacing the top layer of Engineered Fill with foamed glass aggregate to reduce the deformation is shown in Figure 5 and the consolidation results are presented in Figure 6 and 7. The stability for the final construction phase is also checked as shown in Figure 8. The results of the analysis with immediate and consolidation settlements are presented in Table 3.

From the results of the analysis, it is seen that the settlement is significantly reduced within limit by using the foamed glass aggregate as a replacement for the Engineered Fill layer which is feasible in the site and is durable.

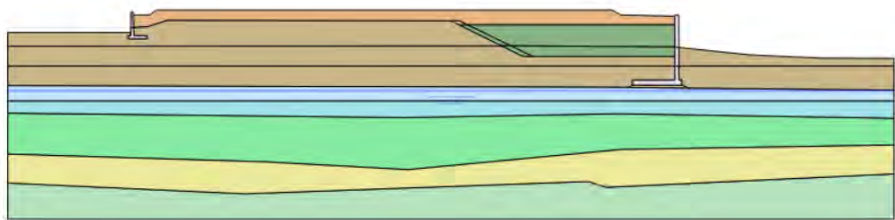


Figure 5 Final construction phase for section 4+180

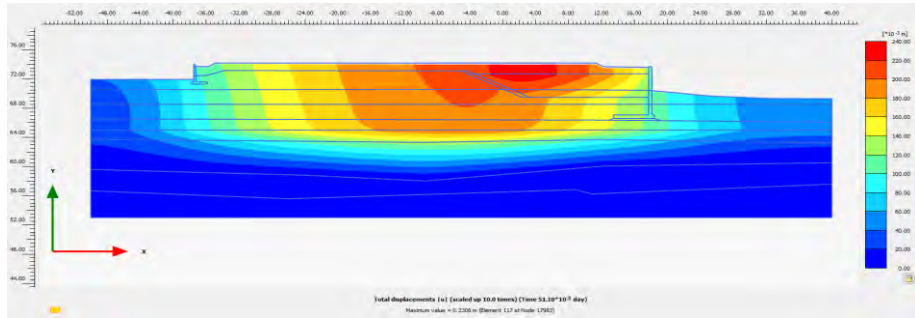


Figure 6 Final stage consolidation for section 4+180

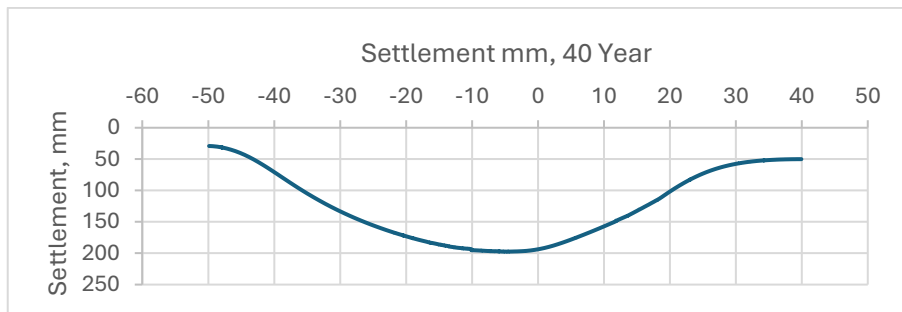


Figure 7 Section 4+180 - Consolidation Settlement vs time plot for 40 years of consolidation. The X-axis represents the distance from track 307.

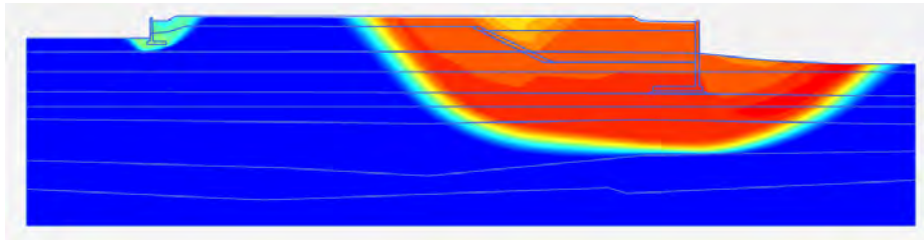


Figure 8 Final stage of stability control for section 4+180

Table 3. Summary of deformation results of each stage.

Stages		Deformation (mm)		Time (years)
		Immediate	Consolidation	
Engineered Fill & Lightweight Fill (FGA)	1st	26.59	0.98	-
	2nd	39.65	5.67	-
	6 months	-	43.51	-
	1 year		22.4	1
	5 years		49.9	4
	10 years		15.8	5
	40 years		26.1	30
Total Settlement		66.24	164.36	40

CONCLUSION

The lightweight characteristics of the foamed recycled glass, together with its positive engineering and environmental outcomes, suggest the material is perfect for its use as a lightweight fill material, according to analyses completed for the study of the effect of foamed glass aggregate (FGA) and from the standpoint of an engineering material. Throughout this study, analysis is performed using PLAXIS 2D software to determine the settlement of embankment by replacing with FGA. It is concluded that replacing filling material in the embankment area with FGA significantly reduces settlement.

Finally, the impact of replacing the fill/embankment under the lightweight material has been analyzed in the paper. Thanks to technological advancements, engineers may now use FGA, a technically sound yet lightweight insulating fill material for use in Engineering projects like Railway tracks, Retaining walls, etc.

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SOIL BEHAVIOUR TYPE CLASSIFICATION FOR TOTAL SOUNDINGS

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KEYWORDS

Soil investigations, SBT classification, Total sounding, Machine learning

ABSTRACT

The total sounding method is the most frequently used site investigation method in Norway; however, only a few classification models exist to interpret its results. This work proposes a new soil behavior type classification model for total soundings. A nearest neighbor classifier with the dynamic time warping distance is proposed, utilizing archived soundings and laboratory results from about 4500 boreholes across Norway. The presented model capability in classifying different soil types is shown to be promising.

INTRODUCTION

The Norwegian Total Sounding is a soil investigation method, developed in cooperation between the Research council of Norway, the Norwegian Public Roads Administration (NPRA), and the Norwegian Geotechnical Institute. It combines elements from the rotary pressure sounding and rock control drilling and is usually conducted prior to any advanced test to get an idea of the soil stratification and depth to bedrock. This method has been the topic of previous papers in this conference (e.g. [1], [2] and [3]). In Sweden, selected phases from the total sounding have been combined with elements from Jb-2 method to define the Jb-tot method (e.g. [1] and [4]).

The total sounding has 5 phases: a normal-, increased rotation-, flushing-, hammering- and rock drilling phase. The method definition given in [5] gives details on when each phase shall be applied, but the first two are identical to the rotary pressure sounding procedure. Interpreting soil stratification with total soundings is done qualitatively based on visual observations of sounding profiles and examples of interpretation procedures are given in the method definition. Despite its widespread use in Norway, only a handful of classification models have so far been proposed (e.g. [3] and [6]). The penetration resistance is measured at terrain level and includes both the tip-resistance as well as the accumulated rod friction from the surface to the tip.

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Resistance curves from all material types are found to span the entire presentation range for the push force 0-30kN, so defining models to identify materials based on resistance values alone is problematic. This paper defines a machine learning approach for Soil Behavior Type (SBT) classification for total soundings, using short intervals of the push force as a basis for the classification, combining the dynamic time warp distance [7] with a k-nearest neighbor classifier [8]. Field and laboratory results from over 4500 boreholes were collected to define the model, but this dataset was later reduced to representative soundings for each class to make the model implementation more practical.

DATASET

Total sounding registrations were linked by depth to grain size analysis (GSA) results in the same position from numerous NPRA projects across Norway. The depth interval around each soil sample was selected as 1m giving about 6500 short push force curve segments. Figures 1A and 1B show the locations of boreholes and how curves were extracted from depths around samples.

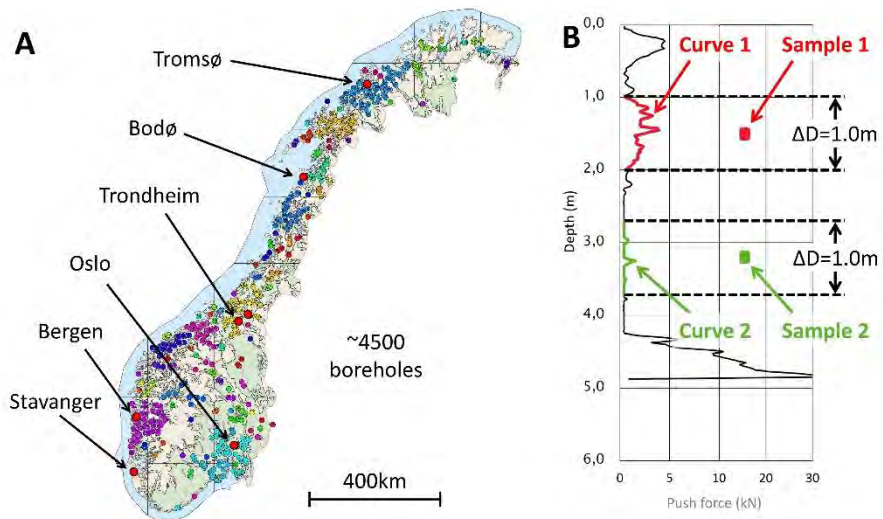


Figure 1 A) Location of boreholes in the training set. B) Push force curve segments were collected in 1m intervals around sample depths.

The classes are generated by interpreting GSA curves according to standard Norwegian practice [9]. The number of classes were reduced to a practical limit of 10 by only allowing main fractions (clay, silt, sand and gravel), possibly with one neighboring secondary fraction as an adjective (e.g. clayey silt, gravel, gravelly sand). Curves corresponding to other labels are disregarded (e.g. clayey sand, gravelly silt). About 4100 curves fit these criteria and are illustrated in Figure 2 colored by class.

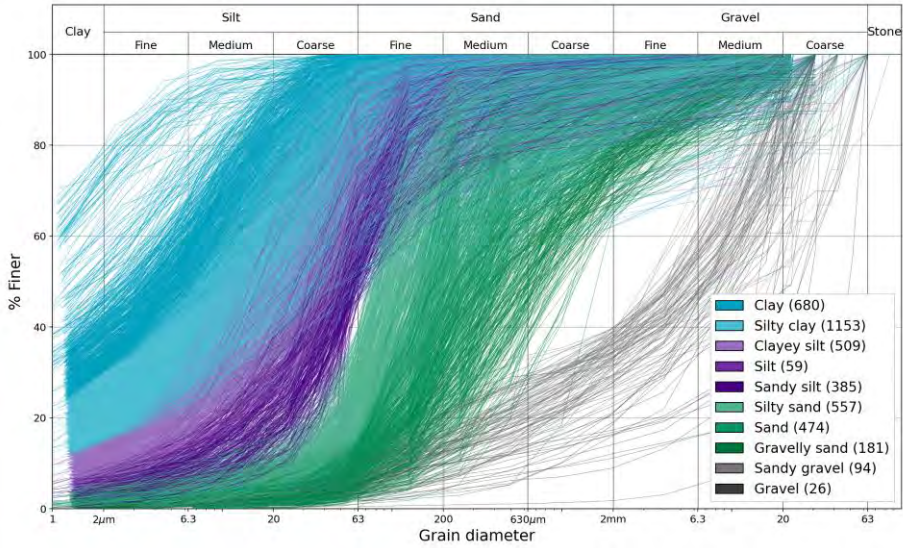


Figure 2 GSA curves used to generate labels for the total sounding data. Each color is a model class and numbers in parenthesis show the number of samples for the class.

Dynamic time warping (DTW)

To compare curve segments from new tests to those in our dataset, the dynamic time warping (DTW) distance is used. This method was proposed by Vintsyuk [7] to identify similarities between audio signals for speech discrimination. A key ingredient of DTW is the alignment process, where optimal alignment of series is identified before their similarity is evaluated. In the original context, the alignment process was useful to account for different dialects or cadence in spoken language, but the algorithm has been found useful across a host of other domains, a few are listed in [10].

Let $x = (x_1, x_2, \dots, x_m)$ and $y = (y_1, y_2, \dots, y_n)$ be two sequences that are not in sync, (Figure 3B upper). The warp cost between any element i and j of x and y can be calculated as

$$\text{cost}(i, j) = (x_i - y_j)^2 \quad (1)$$

for $i = 1, \dots, m$ and $j = 1, \dots, n$. A D-matrix is constructed as the accumulated cost between all elements in both series (Figure 3 A). The value of D for elements x_i and y_j is defined as

$$D(i, j) = \min \begin{Bmatrix} D(i, j-1) \\ D(i-1, j-1) \\ D(i-1, j) \end{Bmatrix} + \text{cost}(i, j) \quad (2)$$

Eq. (2) defines a dynamic programming scheme, where the matrix is constructed from the lowest index ($i = j = 1$) and expanded along either rows

or columns towards the end. Backtracking indexes from the final step to the origin reveals the optimal alignment path (Figure 3B lower). The DTW distance between the two sequences is then calculated as

$$DTW(x, y) = \sqrt{D(m, n)} \quad (3)$$

Variants of DTW include the endpoint relaxation, where alignment can start at index p , in either sequence and can finish q indexes from the end of either sequence with $p, q \in \{0, 1, \dots, r\}$, (e.g. [7] and [10]). It has been suggested that better classification performance can be obtained by windowing the warping path, disallowing paths that stray far from the diagonal (e.g. [11]). This method also reduces the number of calculations needed, thus speeding up the classification. Figure 3 shows an example of a DTW D-matrix for two sequences and their resulting alignment. The DTW distance in this example is $DTW(x, y) = \sqrt{D(10, 9)} = \sqrt{171} \approx 13.08$.

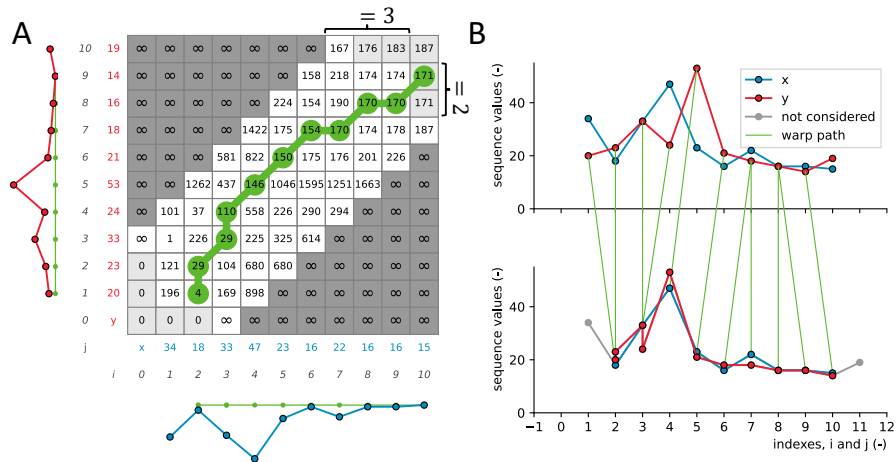


Figure 3 shows both a A) DTW D-matrix using endpoint relaxation $r=2$ and window $w=3$ and the B) original (upper) and aligned (lower) sequences.

The DTW algorithm is flexible and can produce degenerate alignments. It is therefore important to evaluate appropriate settings for the model constraints for each problem (e.g. [12]).

k-Nearest neighbors

We have used a k-Nearest neighbor (k-NN) classifier to produce classifications for new soundings using the DTW distance described in the previous section. The k-NN method was first proposed by Fix and Hodges [8], for binary classification problems using the Euclidean distance as a metric. Figure 4A shows how the original definition can be used to decide the

class of the blue cross, which is set to belong to the majority class for points within a given radius.

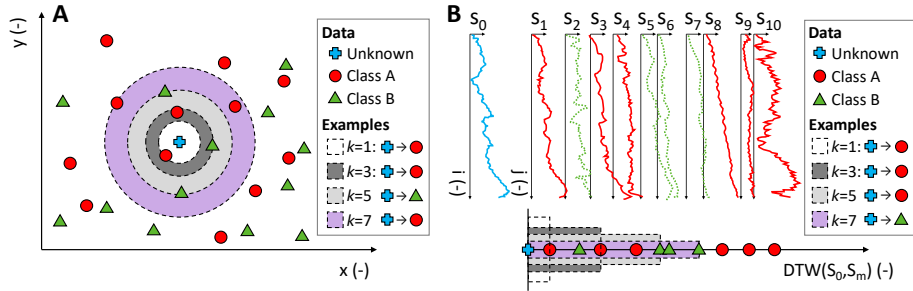


Figure 4 k-NN classification using A) Euclidean distance in 2D and B) DTW distance in 1D.

If the radius in Figure 4A is chosen so the circle contains 3 neighbors ($k = 3$), then 2 are red and 1 is green. The blue cross is then assigned the red class. Increasing the radius so that the circle contains 5 neighbors, then 3 are green and 2 are red. The blue cross is in this case assigned the green class.

The same logic can be applied using DTW, as illustrated in Figure 4B. First all curve segments in the training dataset are compared to the blue curve, ordered by ascending DTW distance. The decision threshold can be adjusted as before. If it is set to contain 3 neighbors ($k = 3$), then 2 are red and 1 is green. Here the blue curve gets assigned the red class. Rather than adjusting the value of the decision boundary (radius or distance in the above examples) in k-NN, the neighbors are simply sorted by the chosen measure and the majority class of the k-nearest neighbors is used as the answer.

DATASET REDUCTION

With the implementation of the DTW algorithm given earlier, the calculations are slow. For practical purposes it is desirable to save calculation time by reducing the dataset to representative segments for each class.

Our initial study agreed with the conclusion in [3], where normalizing the push force was found to give a better basis for classification. It is defined as

$$q_n = \frac{F_{DT}}{A \cdot \sigma'_{v0}} \approx \frac{F_{DT}}{A \cdot \gamma' \cdot z} \quad (4)$$

where F_{DT} is the push force, A is the drill-bit cross section area, $A = 2.55 \cdot 10^{-3} m^2$. σ'_{v0} is the effective vertical stress, z is the depth and γ' is the effective unit soil weight, which is here set to $\gamma' = 7 kN/m^3$.

The set reduction is implemented by first defining two sets for each soil class. One containing all q_n curves, S_A , and another empty set, S_B . Members are iteratively moved from S_A to S_B , in each step selecting the one that minimizes

$$d_{S,i} = \sum_{a \in S_A} \inf\{DTW(a, b) \mid b \in S_B\} \quad (5)$$

for steps $i = 1, 2, \dots, n$. Figure 5A shows the first 100 steps of the reduction for each class. A threshold was set at $t = 20$, where all curves still had members and were past their elbow point. It is beyond the scope of this paper to discuss model parameter tuning, but the AUC_{total} -, F_1 - and accuracy measures are used as described in [13]. As the soil classes are interrelated, predicting the nearest neighboring class during validation is counted as the correct answer. Figure 5B is presented to show that all the measures used support selecting the parameters $w = r = 20\%$ of the curve length and $k = 17$.

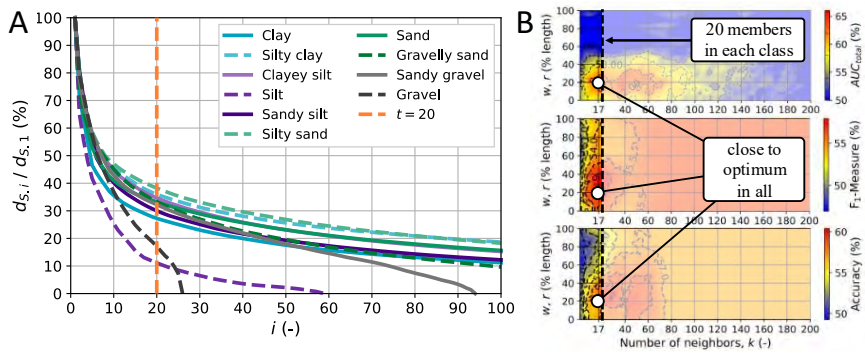


Figure 5 A) Normalized average distance between all elements in S_A to any element in S_B as a function of the iteration step. B) AUC_{total} -, F_1 score and classification accuracy as a function model parameters k , w and r .

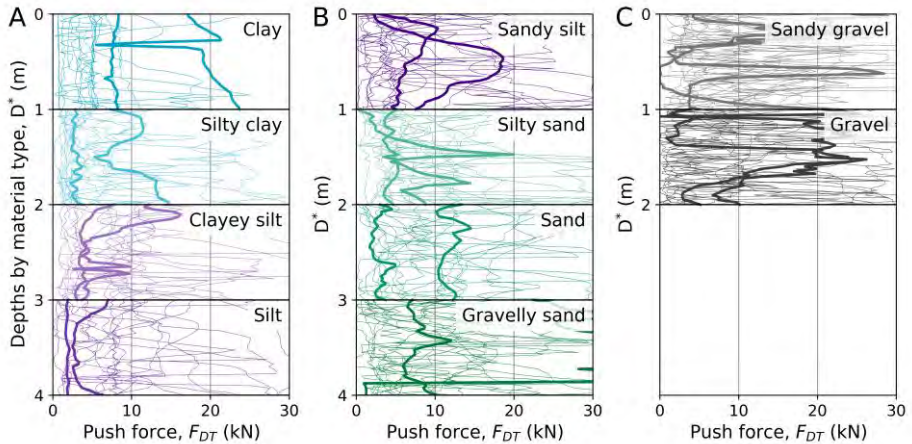


Figure 6 A-C) Reduced training set with 20 representative push force curves for each material type. Each material plotted in 1m depth window. The first two curves selected for each class in Figure 5A are drawn with thicker lines.

CLASSIFICATION EXAMPLES

Two total soundings are selected from the NPRA database to showcase the presented model capability in different soil types. Intervals with flushing or hammering are washed out as the model is not defined for these phases.

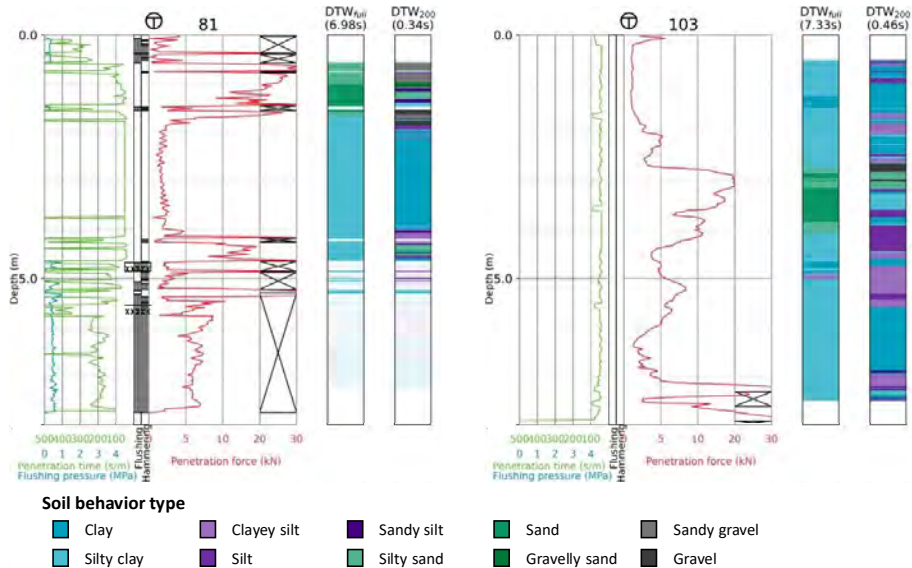


Figure 7 Classification examples using the proposed model with the full- and reduced version of the dataset. Time needed to calculate each profile is given in parentheses.

3. CONCLUSIONS

This work demonstrated that an SBT classifier for the total sounding can be defined with k-NN and the DTW distance using sequences of q_n as features. This approach shows promise and should be investigated further.

The k-NN DTW approach should be applicable for other methods as well, such as the jb-tot, rotary pressure sounding and cone penetration tests.

Classification can be significantly sped up using representative datasets for each class. Examples are given on classification of soundings for the full- and trimmed dataset along with calculation times needed to generate both.

The quality of machine learning models is fully dependent on the quality of the training data. These could be improved using proper estimates of σ_{v0} and u_0 , rather than a fixed γ' . Efforts could be put into removing outliers or curves from improperly performed tests from the training set.

There is room for improvement on the total sounding method as well. A fundamental method improvement would be to implement a load cell at- or close to the tip, thus removing interference from friction against the rod system.

ACKNOWLEDGEMENT

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SPATIAL VARIABILITY IN LIME-CEMENT MIXED CLAY AT MULTIPLE LENGTH SCALES

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Jelke Dijkstra¹**

KEYWORDS

Lime cement columns, variability, soil stabilisation, scale of fluctuation

ABSTRACT

Lime-cement columns, widely used for soil improvement in Nordic countries, exhibit heterogeneous strength and stiffness. Characterising this variability is crucial to optimise their performance. This paper introduces an integrated approach to characterise strength and stiffness variability across various scales in lime-cement columns in natural sensitive clay at the Centralen project site in Gothenburg, Sweden, using laboratory-based methods and field measurements. The aim is to uncover diverse variability estimates, providing valuable insights into the challenges associated with characterising lime-cement mixed soils.

INTRODUCTION

In Sweden, lime-cement columns (LCC) are commonly used to improve the engineering properties of soft soils. However, lime-cement mixed clay in the columns is highly heterogeneous in strength and stiffness. Factors such as type of binder [1], non-uniform mixing of lime and cement [2,3], variations in soil composition and moisture content [4], and heterogeneous distribution of in-situ stresses [5,6] contribute to the complex spatial variability observed in these stabilised soil systems.

This paper outlines an integrated method for analysing spatial variability in both field and laboratory scale, using field mixed samples and quality control data from a Gothenburg excavation site. The results provide valuable insight into the spatial variability of lime-cement mixed clay at different scales, and highlight the challenges involved in characterising this variability.

TEST SITE

The data presented in this study was obtained from an excavation site (E02 Centralen, West Link railway project) located in Gothenburg (Figure 1a). The

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approach used to derive the scale of fluctuation of the stiffnesses along the height of the samples is outlined in detail by Wong et al. [8].

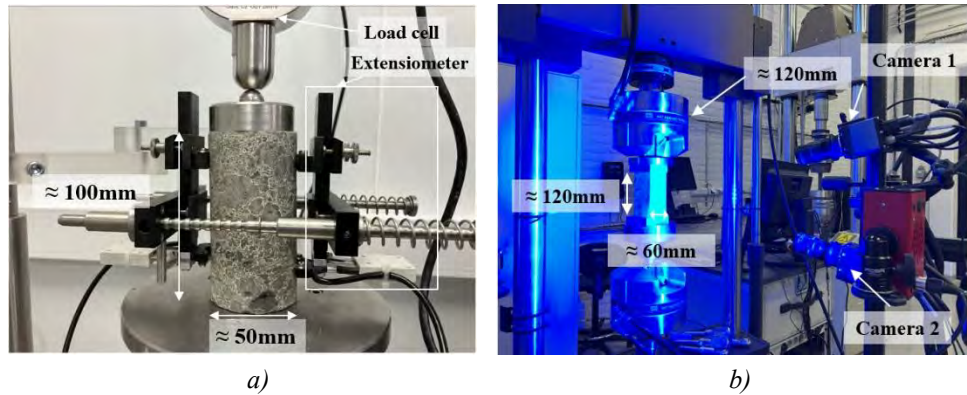


Figure 2: Setup of a) UCS test with local strain measurements b) UCS test with DIC measurements.

Field study

Results from predrilled column penetration tests (FKPS) and reverse column penetration tests (FOPS) conducted 1 to 5 days after construction were utilised to evaluate spatial variability in lime-cement columns at the field scale. Following [8], FKPS data points within the range of 2 to 8 m depth were selected to determine the SOF as the FKPS test is most reliable near the surface. Conversely, FOPS, being a bottom-up technique, offered a reliable dataset within the depth range of 10 to 18 m.

RESULTS AND DISCUSSION

In the following section only selected findings are presented, for complete set of results refer to [8].

Unconfined compressive strength

The variation of unconfined compressive strength q_u with axial strain ϵ and average local axial strain (ϵ_{local}) is depicted in Figure 3a. The q_u ranged from 1.03 to 2.56 MPa (mean value = 1.96 MPa, standard deviation = 0.65 MPa). The variability in q_u values at failure is attributed to inherent variability in the field-mixed samples. Young's modulus E for the tested samples ranged from 199 to 398 MPa (mean value = 286.20 MPa, standard deviation = 67.95 MPa). E_{local} ranged from 589 MPa to 3122 MPa (mean value = 1640 MPa, standard deviation = 921 MPa), averaged approximately 6 times E , indicating its greater representation of small-strain material stiffness. This observation is supported by subsequent DIC measurements of local stiffness values. A E_{50}/q_u ratio of 133 was determined which is in good agreement with the literature of cemented clays [10,11] as seen in Figure 3b.

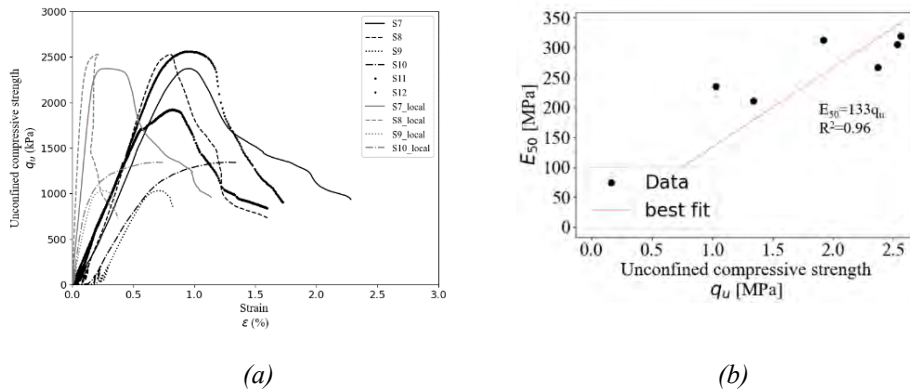


Figure 3: a) q_u vs ϵ and b) E_{50} vs q_u from UCS tests.

Uniaxial compression tests with DIC

The stiffness values obtained from the measured axial load and external Linear Variable Differential Transformer (LVDT) measurements for the two cubic samples tested was 229 MPa and 319 MPa. The globally measured stiffness does not provide information on the variability within the specimens. Visual examination reveals distinct areas of cemented and uncemented material, prompting further investigation with simultaneous surface DIC.

Figure 4a shows the strain distributions on the front surface of sample S14 obtained from DIC corresponding the linear region of loading curve (Figure 4b). A noticeable non-uniform distribution of strains with localisation on areas of uncemented clay is observed. The variation in local stiffness across the height of the samples S13 and S14, estimated from DIC strain measurements, is shown in Figure 5. The stiffnesses obtained from this analysis was in the similar range to those obtained from local transducers (Figure 3a). The mean stiffness was 1028 MPa and the standard deviation was 792 MPa. Following the approach outlined in [8], the vertical scale of fluctuation (SOF_y) for the two samples are summarised in Table 1.

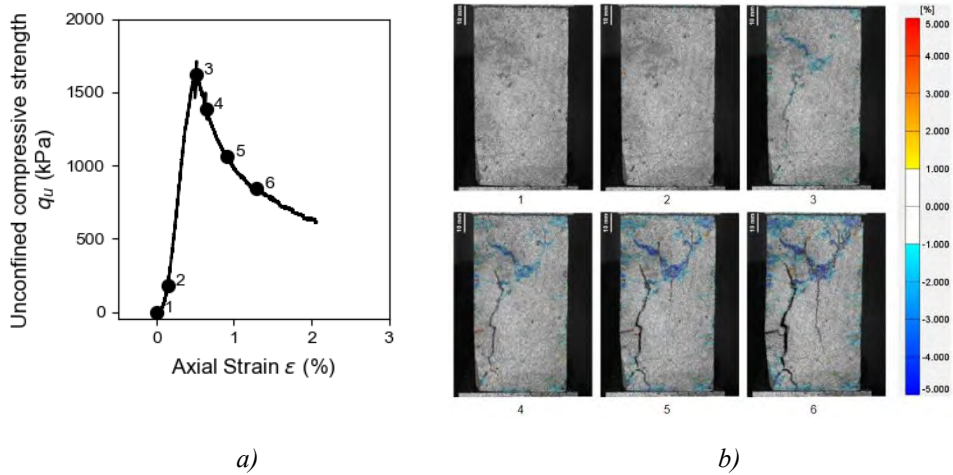


Figure 4: a) q_u vs ϵ for S14 b) Vertical strain field for S14.

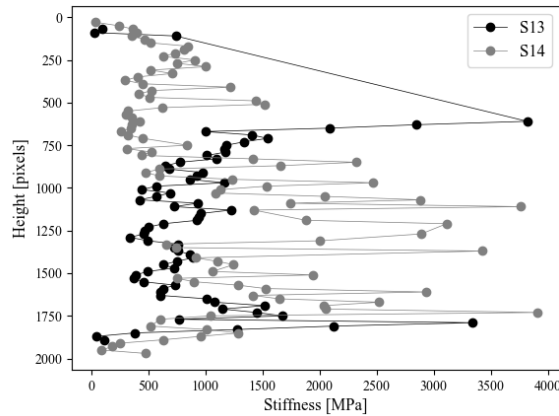


Figure 5: Stiffness distribution along the height of the sample determined using DIC.

Field scale

Natural clay

Figure 6a shows the variation in shear strength with depth of the original clay deposit (as derived from nine sets of cone penetration test (CPT) data prior to construction).

Lime cement column

Figure 6b shows the variation of shear strength with depth obtained from FKPS (202 data sets). Figure 6c shows the corresponding plot for pull-out force obtained from FOPS (29 data sets). The mean of both data sets is shown by the solid black line. The vertical scale of fluctuation for the natural clay and the lime-cement columns, estimated following the procedure outlined in [7] using the data shown in Figure 6, is summarised in Table 1.

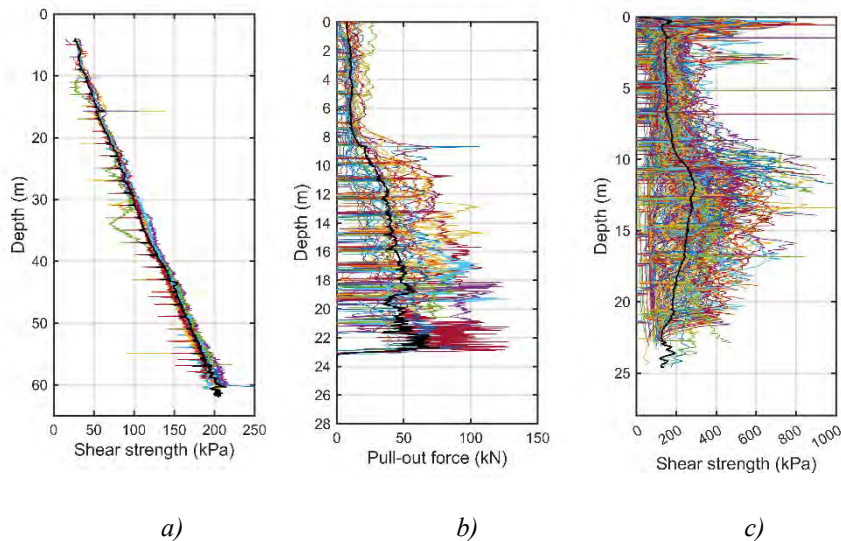


Figure 6: a) CPT data from the in-situ clay deposit b) FOPS data tests from LCC (c) FKPS data from LCC.

Table 1. Summary of $SO F_y$ from different test methods for LCC and the in-situ clay

Test type	Material	$SO F_y$
Laboratory DIC	LCC (S13)	5.4 mm
	LCC (S14)	13.0 mm
FKPS	LCC	0.4 m to 3.5 m, 1.4 m (average)
FOPS	LCC	0.3 m to 2.0 m, 0.9 m (average)
CPT	in-situ clay deposit	2.7 m to 6.3 m, 4.1 m (average)

CONCLUSION

In this paper, various techniques are proposed to characterise the spatial variability of lime-cement mixed clay at the laboratory scale and at the field scale. The key finding is that different magnitudes of SOF exist for lime-cement mixed clay at the laboratory scale and at the field scale, this provides a more nuanced picture on the inherent variability of lime cement columns.

A key challenge is understanding the relationship between the SOF measured at different length scales. However, bridging the results from these tests can be challenging due to differences in timing and conditions. Field tests on lime-cement columns are typically conducted shortly after construction, while laboratory tests involve excavating samples after a 1-year curing period. Nevertheless, the results shed light on the extent of inherent spatial variability in LCC and highlights the challenges in accurately characterising this variability.

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STABILITY OF NATURAL SLOPES - ROBUSTNESS AND SAFETY

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KEYWORDS

Slope stability, Quick clay, Natural slopes

ABSTRACT

Methodology for assessing the stability of natural (existing) slopes in current Norwegian practice is mainly based on undrained total stress analysis (robustness requirement) and drained effective stress analysis (material factor requirement). The two methods are not expected to provide the same material factor (FOS or γ_m). However, when a slope is standing in drained state, but is really at the same time calculated to be at undrained failure ($\gamma_m = 1.0$), the material factor should ideally then be uniquely defined at 1.0. Meaning that there should be no calculated margin left to failure in drained state as well. In practice total stress analyses usually give FOS of 1.0 or even less, while the drained effective stress analyses give a significantly higher factor of safety. Based on current regulations and guidelines, the most unfavorable result must then be taken as a basis for design. This frequently mean that extensive measures must be taken to improve the stability, without necessarily being the optimal use of resources. As of today, the rules do not suggest evaluating analysis results from the two analysis methods considering each other. We believe that one must be able to get a uniform picture of the robustness of a natural slope with both methods. Therefore, there is a need to look more closely at the underlying premises and conditions for the slope stability analyses. This work looks at the historical development in the practice of stability analyses. We find some interesting changes in what has been emphasized over time. There is a varying use of undrained total stress analyses with and without the use of anisotropic models, undrained effective stress analyzes, and drained effective stress analyses.

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INTRODUCTION

Slope stability analysis in geotechnical engineering provides a basis for checking that the required level of safety in the associated regulations is met or to design measures necessary to satisfy the given requirements. Accordingly, the calculated factor of safety (FOS) is of great importance for reliability, economy, and feasibility of a project.

Natural (existing) slopes involving quick clay, by definition, are in a consolidated state, standing under drained conditions with no excess pore water pressure. However, if a failure is triggered by an initial slide, it could lead to a fast and extensive retrogression process making the failure occur under undrained condition. This has led the regulations, from NVE (2020), requiring both the drained and undrained slope stability evaluations, where the undrained slope stability evaluations should be conducted using the total stress approach. The required material factor for the drained analysis is $\gamma_{m,DR} > 1.25$ and for undrained analysis $\gamma_{m,TOT} > 1.20$. With satisfaction of these criteria, the slope is considered safe under existing conditions, i.e. without the need for extra safety measures.

In the recent years we have observed that when conducting area stability evaluations, the consultants responsible for the analyses typically face a contradictory situation of $\gamma_{m,DR} > 1.60$ and $\gamma_{m,TOT} \approx 1.0$. On one hand, the natural slope is evaluated as safe, yet on the other hand, the natural slope is failing under existing conditions. For a failing natural slope or close to failure, the drained or undrained conditions has no relevance, since natural slopes does not carry excess pore water pressure under their existing condition.

Grimstad et al. (2023) demonstrated this issue using undrained effective stress analyses. They found that a slope with undrained material factor $\gamma_{m,UNDR} \approx 1.20$ could have a drained factor, $\gamma_{m,DR}$, as high as 1.60 (but likely it will be lower), meaning that $\gamma_{m,DR} > 1.60$ should ensure $\gamma_{m,UNDR} > 1.20$. Since this result counters the current Norwegian state-of-practice leading to far-reaching implications, more studies are required before decisive conclusions. Furthermore, we need a clear understanding of why generally the calculated $\gamma_{m,TOT}$ indicates a critical condition while $\gamma_{m,DR}$ indicates a (very) safe situation for natural slopes.

Based on our understanding of how the analyses are done in the current practice and their underlying assumptions, our hypotheses for the discrepancies are briefed as follows:

- 1) The total stress ADP (Active-Direct-Passive) method gives too conservative results for $\gamma_{m,TOT}$ for natural slopes. The reasons could be:
 - a. The ADP factors were originally developed and meant to be used for flat terrain and not for the stress situation in slopes.

Aamodt et al. (2021) demonstrated that a modified ADP model for sloping terrain increases the material factor by 0.10 to 0.20 depending on slope angle.

- b. Inputs for active and passive strengths are determined for triaxial condition but used for plane strain problem (Krabbenhøft et al. 2019).
 - c. The undrained shear strength profiles selected as design profiles are inherently conservative, typically due to sample disturbance.
 - d. The interpretation of CPTU does not give the peak undrained shear strength for quick clays.
 - e. Interpretation of CPTU results in slopes is not directly comparable to those established for flat terrains (K_0 consolidation) under the hydrostatic pore pressure condition.
- 2) Since ADP method usually gives conservative results, the drained analyses are commonly undervalued. This led to a practice where various practitioners oversimplify the drained analyses:
- a. Practitioners interpret the friction angle and attraction, for the drained case, directly from undrained triaxial compression tests. Even though studies show that higher strength mobilization levels, for normally consolidated (NC) quick clay, result in tertiary creep that eventually lead to failure. Pusch et al. (2016) found this mobilization level to be $f_{su} \approx 0.85$ based on s_u (which is not directly transferable to the mobilization based on friction angle). Torpe (2014) found that a drained mobilization ($f = \tan \rho / \tan \phi$) as low as $f = 0.70$ still results in tertiary shear creep rupture within laboratory time frame in undrained triaxial tests on NC quick clay (OCR around 1.4). However, it is very difficult to find the critical mobilization from laboratory tests as any sample disturbance will result in over predicting the initial creep rate.
 - b. The interpretation of critical friction angle and attraction (cohesion) should also be evaluated in the light of geological history. For instance, if the OCR is much higher than two and a sample is sheared (i.e. on the “dry side”), as results shown by e.g. Leroueil (2001), the Coulomb line (critical state line) will practically be a limitation due to undrained creep swelling followed by tertiary creep. This is very relevant for ravines, see section 2.

- c. Practitioners usually overlook the critical situation in terms of pore pressure. Often only the ground water level (GWL) is identified, and hydrostatic pore pressure distribution is assumed. A ground water flow analysis is seldom conducted despite being highly relevant for sloping terrain. Critical deviations related to seasonal variation and precipitation are not considered.
- d. Despite the “independent” interpretation of shear strength in drained and undrained (total stress) condition, the critical cross sections for drained case are not identified independently from the undrained (total stress) case.

The primary purpose of this article is to present a critical review of the current state-of-the practice looking at historical development of the provided guidelines for assessment of stability of natural slopes in quick clay. Secondly, the work aims to establish the basis for a bigger project aimed at giving input for new guidelines for assessing the safety and robustness of natural slopes in quick clays.

NATURAL SLOPES IN QUICK CLAY

Lyche¹ (NVE 2015) categorized natural slopes into four groups along with the following geotechnical and geological characteristics (NVE 2015).

1) “Even terrain slopes”

Geologically:

- Formed by deposition in an earlier seabed basin, governed by "underlying" topography (rock).

Geotechnically:

- Normally not preloaded (OCR only due to creep) but weathered with 1-3 m of dry crust. Often homogeneous deposits, but proximity to mountains or watercourses often results in more variable conditions regarding layering and pore pressure.

2) “Ravine slopes”

Geologically:

- Formed by erosion in a clay deposition (for example in an even terrain slope)

Geotechnically:

¹ Presentation at workshop at NPRA Trondheim, 2014, by Einar Lyche (NVE)

- Ongoing erosion process, including continual development of small slides and surface slips.
- Marginal stability against shallow slides (liable surface)
- Often critical in triggering quick clay landslides.

3) “Terrace slopes”

Geologically:

- Erosion from large rivers/watercourses ("mega ravines") in marine deposits.
- Completed deposition steps under the withdrawal of the ice.

Geotechnical: (as for ravines)

4) “Submarine slope” (“Marbakke”)

Geologically:

- Formed similarly to a terrain slope above water.

Geotechnically:

- Exposed to wave erosion and to marine erosion forces, such as shore currents or propeller currents (in ports)
- Mapped often in connection with larger development projects.
- Quick clay occurs also off the shore.
- Warning signals about possible landslides are invisible.

Total stress ADP analyses

Roughly before 2000, the use of ADP analyses for natural slopes were quite sparse and mostly used for stability evaluations of cuts and fillings. The reference to use of ADP, with NGI code BEAST v2003, for natural slopes came with the development of the GeoSuite package. The main available reference regarding ADP analyses for natural slopes is the stability seminar held at Hell, Norway in 2003¹. The handbook of the Norwegian Public Road Administration (SVV 1992) “HB016 – Geoteknikk in vegbygging” page 105

¹ Karlsrud, Kjell (2003) “Stabilitetsanalyser av skråninger, skjæringer og fyllinger”. Kurs 20.-22. mai 2003, Rica Hell Hotell

states: “*Totalspenningsanalyse ... må bare brukes innenfor sitt gyldighetsområde*” – meaning “*total stress analyses... must only be used within its valid domain*”. This means that it is up to practicing engineers to define that domain. In the authors’ understanding, the common practice for evaluating the stability of natural slopes varied with respect to how much emphasize was given to the total stress analyses and how they were performed. These variations were observed between different consultant firms, state agencies and regions.

Drained effective stress analyses

Over the last years, it seems that in many cases the effective stress analyses of natural slopes in quick clay has been reduced to a mandatory exercise done as formality. SVV (2018) suggest using a friction angle (ϕ) as low as lower than 20° for quick clay. With such a low value for ϕ , it would be surprising if the total stress analyses would still give the lowest results for the calculated material factor. However, this low ϕ is not the common practice, yet, based on the authors observations, the friction angle interpreted for high strain levels from undrained triaxial test is used (orange solid line in Figure 82). This is typically in the range of 32° - 36° (combined with zero cohesion). The friction angle used in effective stress analyses should consider possible tertiary shear creep (point 2a.).

Undrained effective stress analyses

Undrained effective stress analyses (or ESAU) (Svanø and Nordal 1987) has not become a standard method for stability evaluation. Hence, undrained analyses with use of Janbu D parameter have not been seen in practice. These type of analyses with $D < 0$ always give some undrained margin of safety if the slope is standing under drained condition. The selection of friction angle from undrained triaxial testing is still questionable, but less than for the drained state since there is some control on the undrained shear strength through D . However, the role of shear creep is still not properly accounted for.

Expectations

It is expected that total stress analyses should yield the most critical results for slopes on even terrain. On the other hand, for ravine slopes, total stress analyses should give safer results than long term (effective stress) analyses. Due to the possible creep swelling (point 2e.), the drained analyses would be more reliable for ravine slopes. If the pore pressure is not properly evaluated, then the effective stress analyses are surely directly affected (point 2c.). Also, since the interpretation of CPTU are dependent on the initial pore pressure it might also give unreliable results (point 1e.).

With finite element analyses, pore pressures can be better estimated in seepage calculation and drained/undrained safety analyses are both available

in mechanical part. However, it is not possible to make direct comparison between drained and undrained failure for the same mechanism in finite element analyses. But, with the method of slices material factor for the same mechanism can be compared across the different analyses.

METHODOLOGY FOR FRICTION ANGLE DETERMINATION

Figure 82 shows a schematic response of undrained triaxial tests run with three different axial strain rates, $\dot{\epsilon}_{a1} < \dot{\epsilon}_{a2} < \dot{\epsilon}_{a3}$, where $\dot{\epsilon}_{a1}$ represents the slowest possible rate that can still be practically considered as undrained. The figure also shows schematically the response of three different undrained creep tests with different initial mobilization (initially at the same volume). There will be an initial mobilization for which the minimum strain rate during the undrained creep phase will be equal to $\dot{\epsilon}_{a1}$. Setting this as a critical mobilization line (envelope), here solid green line, it provides the value of φ to be used in drained analyses, as higher average mobilizations along a mechanism that would cause creep failure. The dashed green line gives the mobilized friction angle at “point of no return” (for $\dot{\epsilon}_{a1}$).

Further, as mentioned earlier, tertiary creep limit (critical mobilization) will depend on OCR. In Figure 83a schematic test result on normally consolidated (NC) quick clay is compared to a schematic result of an over-consolidated (OC) quick clay, with the same initial effective stress. As seen, it is expected that the critical mobilization is a function of OCR which in practice means on the natural slope type. “Even terrain slopes” would have an OCR due to aging alone (around 1.4), while in ravine slopes the OCR will depend on the depth of the ravine and position of the point (depth) in relation to the ravine surface. The OCR will depend on depth (the effective stress) in the ravine slope. Alternatively, it is possible to make consideration for this variation by using a representative cohesion and a single value for φ . This principle is demonstrated in Figure 84.

Figure 85 gives the expected variation in the critical φ as a function of OCR. It is assumed that a NC clay (1 day old) will creep to failure even in K_0 condition. With inspiration from SHANSEP (Ladd et al. 1977), the following is used:

$$\sin \varphi = \min \left(\frac{1 - K_0^{NC}}{1 + K_0^{NC}} \cdot OCR^m, \sin \varphi_{CAU} \right) \quad (1)$$

Where φ_{CAU} is the “conventional” friction angle from undrained triaxial compression tests (orange solid line in Figure 82). The figure gives some indications of the expected variation in OCR for the categorized slopes and variation in friction angles as hatched areas.

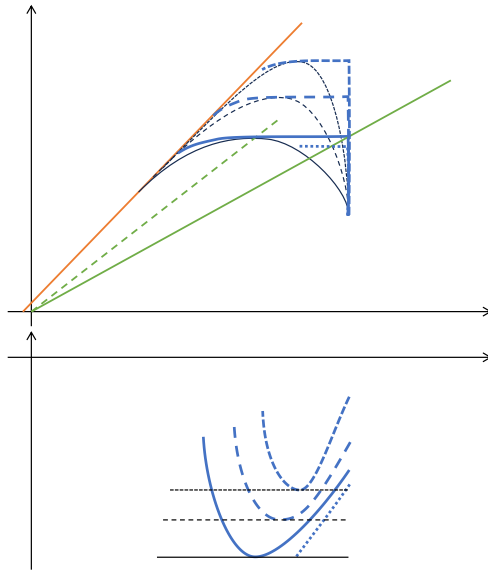


Figure 82 Schematic behavior of NC quick clay in undrained triaxial tests. Three tests with varying rate $\dot{\epsilon}_{a1} < \dot{\epsilon}_{a2} < \dot{\epsilon}_{a3}$, Four undrained creep tests with varying initial mobilization. Results given in $p' - q$ and $p' - \ln \dot{\epsilon}_a$

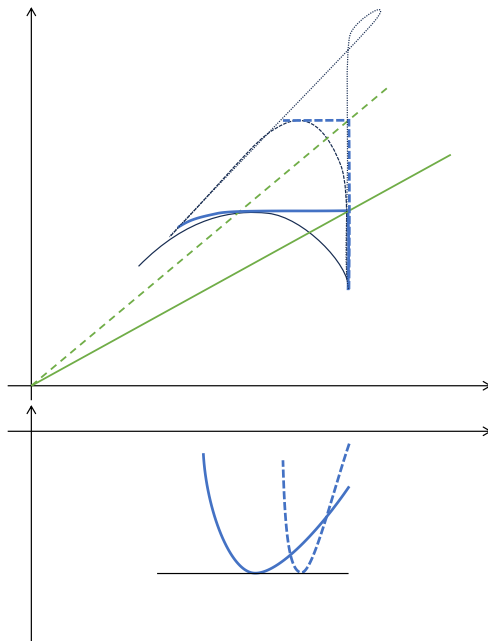


Figure 83 Schematic behavior of NC and OC quick clay in undrained triaxial tests, both constant rate and undrained creep tests. Results given in $p' - q$ and $p' - \ln \dot{\epsilon}_a$. Critical mobilizations dependent on OCR.

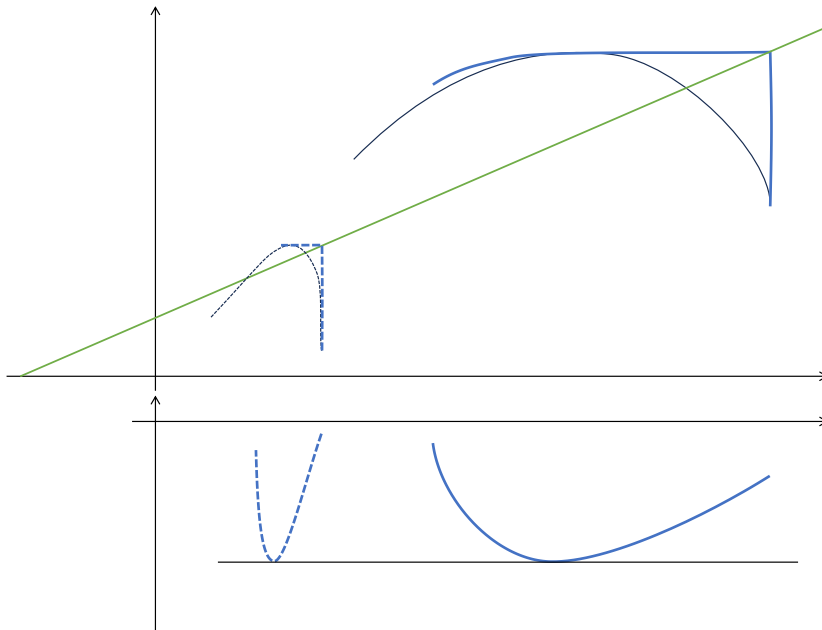


Figure 84 Schematic behavior of NC and OC quick clay at two different depths (stress level) in undrained triaxial tests, both constant rate and undrained creep tests. Results given in $p' - q$ and $p' - \ln \epsilon_a$. Interpreted as constant ϕ and with cohesion/attraction.

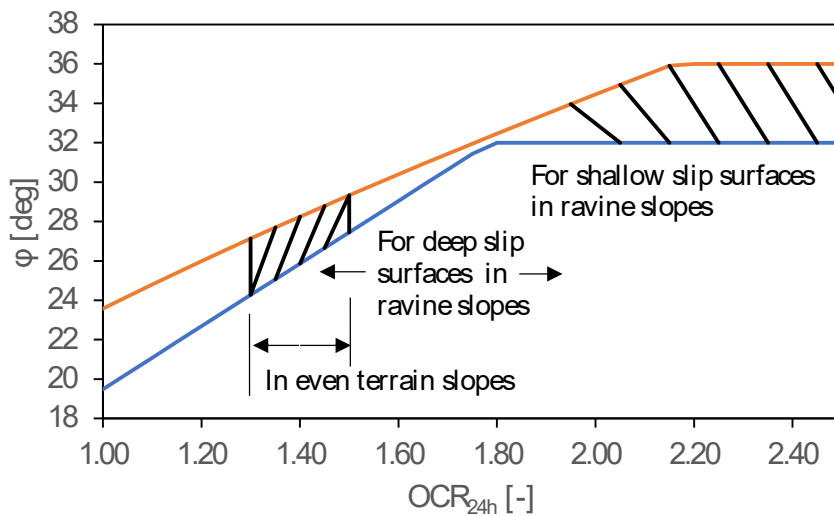


Figure 85 Friction angle for drained analysis as a function of OCR corresponding to 1 day reference time (with low values for cohesion)

EXAMPLE ANALYSIS

In this section, a hypothetical even terrain slope is considered to numerically illustrate the presented methodology. An even terrain slope consisting of dry crust, clay, quick clay, and a permeable layer is considered and analyzed in Plaxis 2D, Figure 86.

Two cases are analysed with varying friction angles. From Figure 85 we see that for an even terrain slope in quick clay a friction angle of 25° is a reasonable value in drained safety analyses, $OCR \approx 1.4$. While for a sensitive clay, but not quick, 27° could be more appropriate (upper estimate at same OCR). The hypothetical case of Grimstad et al. (2023), Figure 86, is re-analyzed with these input parameters (see Table 1). The calculated drained material factor is $\gamma_{m,DR} = 1.47$ and the undrained material factor was found to be $\gamma_{m,UNDR} = 1.23$, using a Janbu $D = 0.0$ and the same values for φ (ESAU approach). This result is as expected for the even terrain slope since the undrained analysis gave the lowest calculated safety factor.

For a second case, more unconservative values for the drained analysis, like $\varphi = 29^\circ$ for both quick clay and clay gave $\gamma_{m,DR} = 1.65$. If we assume that drained and undrained analyses give unique simultaneous $\gamma = 1.0$ at failure, linear interpolation implies that $\gamma_{m,DR} > 1.60$ would ensure $\gamma_{m,UNDR} > 1.20$ for cases similar to the (extreme) second case. With a similar line of reasoning $\gamma_{m,DR} > 1.41$ would ensure $\gamma_{m,UNDR} > 1.20$ in the first case.

In case one, the drained failure mechanism (Figure 88) is similar to the undrained failure mechanism (Figure 87). In the second case (Figure 89) the drained failure surface is not in the quick clay and shallower and more local.

Table 1. Parameters

	Quick clay	Clay	Perm. layer	Dry crust
γ [kN/m ³]	19.0	19.0	19.0	19.0
G' [kPa]	2000	2000	2000	2000
K' [kPa]	4000	4000	4000	4000
φ [°]	25 (29)	27 (29)	35	30
$k_x = k_y$ [m/day]	0.001	0.001	0.1	0.1
K_θ	$1 - \sin\varphi$	$1 - \sin\varphi$	$1 - \sin\varphi$	$1 - \sin\varphi$

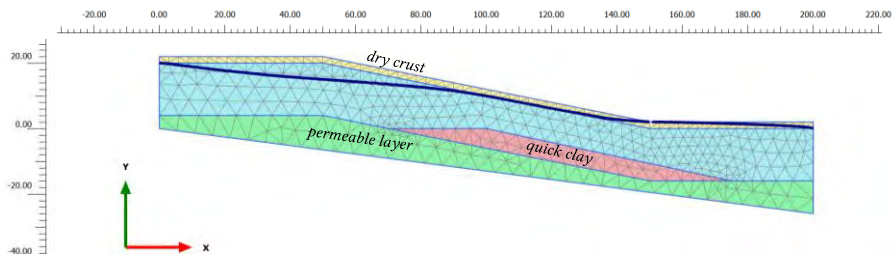


Figure 86 Geometry of the even terrain slope.

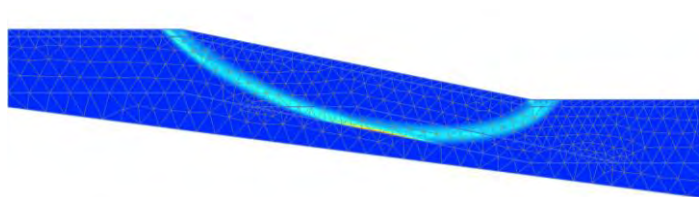


Figure 87 Undrained failure mechanism (shear strain)

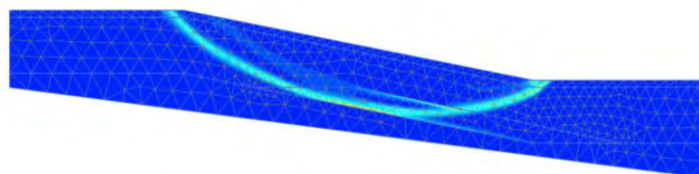


Figure 88 Drained failure mechanism (shear strain) for case one.

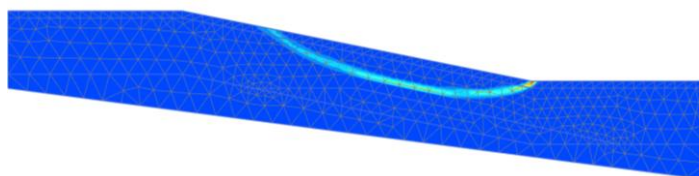


Figure 89 Drained failure mechanism (shear strain) for case two.

CONCLUSIONS

The current practice for stability evaluations often leads to situation where an existing slope in drained state is evaluated as “unsafe” from a total stress based undrained analysis. Possible reasons for such discrepancies are discussed in the article. The main message to take from this review is that ADP analyses have led to too little focus on critical pore pressure evaluation and relevant interpretation of friction angle, to use in drained stability analyses. This also led to a practice where measurement of pore pressure profiles is seldom carried out in projects. The example shown in the article demonstrates that a calculated drained factor of safety higher than 1.6 ($\gamma_{m,DR} > 1.60$), using the proposed interpretation of friction angle, also satisfies the robustness criterion set with undrained factor of safety larger than 1.20 ($\gamma_{m,UNDR} > 1.20$).

Further studies are being carried out in the project to identify an acceptable drained safety level that also ensure sufficient robustness.

ACKNOWLEDGEMENT

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STABILIZATION OF SULFIDE-RICH SEDIMENTS WITH INDUSTRIAL SIDE STREAMS

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KEYWORDS

Acid sulfate soils, Stabilization, Industrial side streams, Leaching.

ABSTRACT

The dredged spoils in the coastal areas of Finland are very likely potential acid sulfate soils that, when spread on land, begin to oxidize and form sulfuric acid. The drop in pH dissolves harmful elements from the clay that are transferred to the environment with the runoff water.

Chemical analyses were performed to determine the pre-incubation characteristics of the fresh sample and the sample after 19 weeks of incubation. The total concentration of the elements in the spoil was analyzed after digestion in aqua regia. The leaching of metals and metalloids was determined by using the two-stage shaking test method.

Seven recipes with industrial side stream (ash, blast furnace slag and lime residues) additions were developed, with an emphasis on reducing the amount of cement needed for clay stabilization. Both laboratory and field tests were performed. The unconfined compression strength (UCS) was determined after 7, 28 and 91 days. The leaching of elements from the stabilized samples was determined.

The measured strength development and successful immobilization of harmful elements suggest that sulfate-rich sediments can be used as building ground materials. Industrial side streams can substitute part of the cement traditionally used for stabilization. Accordingly, a significant reduction in carbon footprint, calculated as CO₂ emissions, can be achieved.

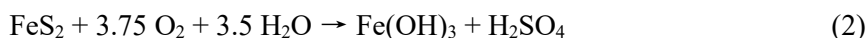
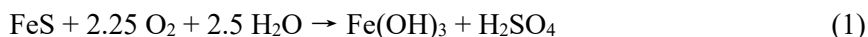
INTRODUCTION

Dredging is necessary for deepening and maintaining waterways and port areas. However, disposing of the large amounts of dredged sediment is challenging. Dredging and dumping the sediments at sea can negatively affect the aquatic environment. Thus, increasing interest is paid in utilizing dredged spoils for land construction. However, the beneficial use of dredged spoils

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requires the stabilization of clays in the sediment. Legislation must also be considered to determine the risks of the total and soluble concentrations of elements or compounds leaching from the dredged spoils, industrial side streams, and treated dredged spoils.

In the Baltic Sea countries, sediments deposited in the Littorina Baltic Sea may be rich in acid-generating sulfides. When the sulfide-rich sediments are disposed on land, the sulfides in the usually metastable iron sulfide, denoted FeS in reaction 1) and pyrite (FeS₂) in reaction 2), start to oxidize, leading to the formation of sulfuric acid and subsequent release of harmful elements [1].



If the pH drops below 4 due to the oxidation of sulfide minerals, an actual acid sulfate soil is formed.

In geotechnical engineering, stabilizing different clays is widely applied [2-5]. Stabilization solidifies soil materials otherwise unsuitable for construction due to their poor load-bearing capacity. Stabilization hardens the clay soil, thus making it suitable for construction grounds. Stabilization immobilizes harmful substances and prevents the acid formation characteristic of sulfide-containing soils. Cement is typically used for stabilization. However, cement production negatively impacts the environment due to high CO₂ emissions and energy consumption. Thus, using industrial waste and side streams (e.g., slag, ash, and lime residues) as part of the stabilization recipes significantly reduces CO₂ emissions compared to stabilization with cement alone.

In this work, industrial side streams were used to stabilize sulfide-rich sediments. The amount of cement was reduced by using a high amount of fly ash in combination with blast furnace slag, bottom ash, and lime residues. Another goal was to prevent the dissolution of harmful metals from the stabilized soil, thus enabling the safe use of sulfide-rich sediments in the soil underlying the construction foundations. The work was carried out both in the laboratory and in field experiments.

MATERIALS AND METHODS

The dredged spoil used in this study was an organic-rich silty clay typically found in the coastal area of southern Finland. Dredging was used to deepen

and maintain the waterways of a narrow bay called Matalahti (Naantali, Finland). The dredged sediments were deposited on land close to the bay.

Chemical analyses were performed to determine the pre-incubation characteristics of fresh dredged sediments and oxidized samples after 19 weeks of incubation. Titratable incubated acidity (TIA) was determined by mixing the soil with KCl and then titrating it with NaOH to a pH of 5.5 [6]. Total element concentrations in the sediments and side streams used for the stabilizations were determined after digestion in aqua regia by ICP-MS or ICP-OES. The leachable concentrations of elements were determined by using the two-stage shaking test (method SFS-EN 12457-3).

Cement (Plus, CEM II/B-M S-LL 42,5N, Finnsementti Oy) was used as a reference binder in the stabilization tests. The Plus cement is a blended cement consisting of 65-79% clinker and 21-35 % blast furnace slag and calcite. Fly ash and three alternative stabilizing materials were chosen: 1) A commercial ground granulated blast furnace slag (GGBFS) product (KJ 400, Finnsementti Oy). 3) Fly ash received from a combined heat and power (CHP) plant using a mixture of biomass (60-75%), coal (25-40%), peat (1-2%), and refinery gas (1-2%) as fuel. 4) Lime kiln dust (LKD) collected from the lime kiln in lime production (Nordkalk Oy). 5) Municipal solid waste incineration bottom ash (MSWI), received as a 0-2 mm fraction from Suomen Erityisjäte Oy.

Laboratory experiments were first performed to optimize the stabilization recipes for the field test. In the field tests, the dredged spoil was spread on the test area and left to dry for 6 months before mixing with the binders. The area was divided into squares and in each square, the fly ash was spread and mixed with the spoil. The following day, the other binders were mixed into a depth of 1 meter (Figure 1). Samples were taken from each test square and test cylinders for unconfined compression strength (UCS) measurements were prepared.

The stabilization recipes were calculated as the mass of binders in 1 m³ of spoil (Table 1). In total, 350 kg binders, including an ash content of 100-200 kg/m³ were added to 1 m³ spoil. In one recipe (R7), only 600 kg ash per 1 m³ spoil was added. The raw materials were weighed and mixed in a Hobart mixer for two minutes. Small, compacted cylinders, 56 mm in height and 28 mm in diameter (2:1) were prepared by tapping the tube against the table. The sample cylinders were tightly plugged at both ends. The samples were stored for 2 days at room temperature (23 ± 1°C) and then moved to a refrigerator (6 ± 1°C) to wait for the compression strength measurements. The UCS was measured after designated curing times of 7, 28, and 91 days.

UCS was measured using a universal testing machine (L&W Crush tester) operated at a speed of 1 mm/min. The maximum load (N) at the breaking

point was used to calculate the strength (kPa). The average strength was based on triplicate measurements/cylinders for each mixture. The mixed samples from each test area in the field tests were directly put into cylinders for the strength measurements as given above.

The two-stage shaking method was used to measure the leachable concentration of the elements from the stabilized samples after 91 days.

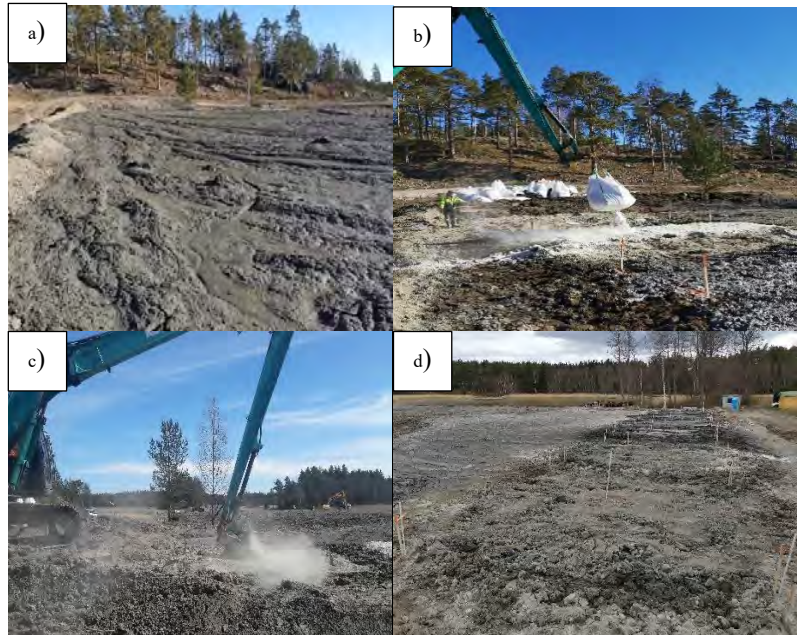


Figure 1. Field test: a) dredged spoil before stabilization, b) adding binders, c) mixing, and d) test field ready.

Table 1. Stabilization recipes calculated for 1 m³ wet sediment with the average mass of 1300 kg.

Recipe	Ash kg/m ³	Cement kg/m ³	GGBFS kg/m ³	LKD kg/m ³	MSWI kg/m ³
R1	200	150	0	0	0
R2	200	50	100	0	0
R3	100	150	0	0	100
R4	100	50	100	0	100
R5	200	50	50	50	0
R6	200	25	75	50	0
R7	600	0	0	0	0

RESULTS AND DISCUSSION

Properties of dredged sediment

The pH of unoxidized dredged spoil was 7.8 but decreased to 2.7 during the incubation for 19 weeks. The material was fine-grained, with clay particles < 2 µm around 52%. The measured loss on ignition (LOI) was 5.1%. For the unoxidized material, the total sulfur and sulfide contents were 1.9% and 1.6%, respectively. The incubated acidity (TIA_{pH5.5}) was 158 mmol H⁺/kg. This value exceeds the limit of 100 mmol H⁺/kg, which is defined as a high risk of acidification in fine-grained soil materials, thus being classified as acid sulfate soil material.

The total concentrations of elements in the fresh dredged spoil after digestion in aqua regia and the leaching of harmful metals are shown in Table 2. Element leaching from the fresh unoxidized dredged spoil was very low; only molybdenum (0.9 mg/kg) showed significant leaching. However, several harmful elements leached from the oxidized spoil sample with a pH of 3.4. The measured soluble element concentrations were compared to the Finnish decree on the recovery of certain wastes in earth construction (MARA, 843/2017) [7]. The solubility limits in the decree can be used to assess whether stabilizing or neutralizing soils with industrial side streams possesses environmental risks. The soluble concentrations were compared to the maximum permitted solubility in covered and paved field structures. The leachable concentrations of Cd (0.20 mg/kg), Ni (6.4 mg/kg), and Zn (27.7 mg/kg) exceeded the maximum limits given in MARA decree (Cd 0.06 mg/kg, Ni 1.2 mg/kg, and Zn 12 mg/kg) for a paved field structure.

Stabilisation results

Strength, given as the UCS values, for the laboratory and field samples after 7, 28, and 91 days are shown in Figure 2. The average UCS for the different stabilized field samples varied between 750 kPa and 2500 kPa after 91 days. These values were doubled compared to those received for the samples sediments stabilized in the laboratory (400 to 1200 kPa). The standard deviation was relatively high (400 -700 kPa) for the field stabilization samples but clearly lower (< 100 kPa) for the laboratory stabilization samples. The high standard deviation implies that mixing the binders with the sediment was not homogeneous in the field tests. The significantly higher strength values in the field tests compared to the laboratory tests can be explained in different ways. The inhomogeneous mixing and lower water content of the dredged spoil in the field test likely influenced the results. Binders on top of the soil and mixing from above did not provide a homogenous material in the depth direction but probably gave a higher content of binders close to the surface. The dredged spoil had time to dry before adding the binders. Thus, the relative share of binders was likely higher in the field tests than in the laboratory tests. The water content had decreased from 280% when deposited

on land to around 110% after 6 months. Most likely, the water content varied locally in the field squares. In the laboratory experiments, the water content was more even, about 130%, thus explaining the higher strengths in the field experiments. It was assumed that the exact location of the sampling point for the field tests affected the results, as suggested by the relatively high standard variation of the measured strengths.

The laboratory experiments were assumed to describe better the impact of the different side streams on stabilization. Substituting slag GGBFS (recipes R1 vs R2 and R6) for about 65% of the cement provided similar strength values after 91 days. In contrast, a slower early strength development was measured for the slag mixtures. GGBFS has been successfully used to substitute cement in concrete and, together with limestone, in blended types of cement [8]. Substituting MSWI bottom ash for 100 kg/m³ fly ash (stabilization recipes R5 and R6) did not affect the strength. R8, with the lowest cement content, showed the lowest strength. The strength of the test area with ash as the stabilizer (R7) was about 100 kPa. However, the dredged spoil mixed with 600 kg/m³ fly ash was hard enough to enter even with a caterpillar after 14 days, most likely due to the drying and compaction of the mixture.

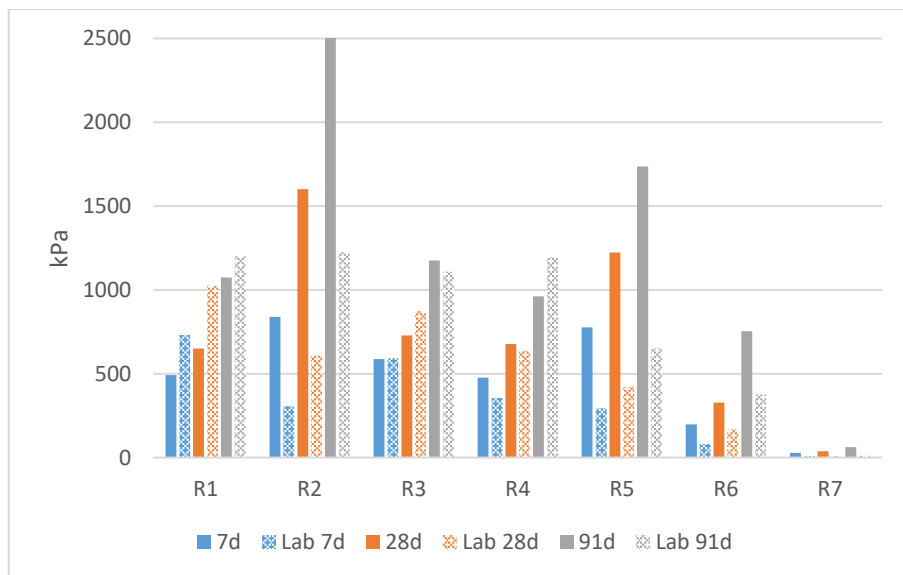


Figure 2. The unconfined compression strength (UCS) of the soil and binders mixed in the laboratory and in the field.

The leaching experiments were performed for the field test samples. The leaching of elements after stabilization was generally low. Cd, Ni and Zn leach from acid sulfate soils upon oxidations. However, the concentrations of these elements leaching from the stabilized samples was low. In contrast, the solubility of Mo increases with increasing pH; thus, the leaching of Mo

increased from the stabilized samples and exceeded the limit allowed for a covered field structure (0.5 mg/kg). The industrial side streams used as binders affected the leaching results. The total concentration of copper (1730 mg/kg) was very high in MSWI. However, only a small part of copper (2.7 mg/kg for sample R3) was leached out, but yet exceeding the limit for a covered field (2 mg/kg). On the other hand, the value was below the limit of a paved field structure (6 mg/kg). The release of vanadium for mixtures containing slag (GGBFS) exceeded the MARA limits. The total concentration of vanadium in GGBFS was 396 mg/kg. As vanadium concentration was very low in the dredged spoil, the leached vanadium must come from the slag used for the stabilization.

Table 2. Total concentration of the dredged spoil, leaching of elements from the dredged spoil fresh and incubated for 19 weeks and from the field stabilisation recipes after 91 days.

Element	Total conc.	Leaching								
	Fresh mg/kg	Sediment		Recipes						
		Fresh mg/kg	incubated mg/kg	R1 mg/kg	R2 mg/kg	R3 mg/kg	R4 mg/kg	R5 mg/kg	R6 mg/kg	R7 mg/kg
Sb	0.62	0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
As	9.9	0.1	<0.05	0.1	0.1	<0.05	0.1	0.053	0.051	0.1
Cd	0.33	<0.05	0.20	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Cr	81	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Cu	47	<0.5	0.5	2.2	1.8	7.2	0.9	2.2	1.8	<0.5
Pb	17	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Mo	2.4	0.9	<0.05	1.2	1.3	2.7	0.7	1.4	1.1	1.3
Ni	51	0.1	6.4	0.8	<0.07	0.8	<0.07	0.8	0.6	<0.07
Se	3.6	<0.05	<0.05	<0.05	0.1	<0.05	0.1	0.1	0.1	0.1
Zn	181	<0.6	27.7	<0.6	<0.6	<0.6	<0.6	<0.6	<0.6	<0.6
V	86	0.4	<0.2	0.6	2.2	0.5	3.3	2.0	3.0	0.7

CONCLUSIONS

The presence of acid sulfate soils and potential acid sulfate soils should be identified and considered in land construction at an early stage, preferably even before excavating or dredging. Acidic sulfate soils are challenging for land construction because of poor geotechnical properties, e.g., high water content and organic matter. Notably, such soil oxidation might lead to the release of potentially harmful elements in run-off waters. The dredged sediment studied contained acid sulfate soil material. Thus, harmful elements (Cd, Ni, and Zn) leached from unstabilized sediments during incubation.

Four industrial side streams were tested as stabilizers: blast furnace slag, ash, lime kiln dust and MSWI bottom ash. The field and laboratory tests showed that dredged sediment could be stabilized with cement and industrial side

stream mixtures. This implies that cement can partly be replaced with the side streams, depending on their composition. All mixtures achieved good geotechnical properties. The stabilization effectively neutralized the acid-producing capacity of the sediment. Thus, leaching of harmful substances from the stabilized masses was minor. The amount of industrial side stream needed suppressing the leaching must be optimized for each side stream and sediment to fulfill the limits in legislation for a particular application.

ACKNOWLEDGEMENT

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STRENGTH DEVELOPMENT AND SHEARING BEHAVIOR OF CEMENT STABILIZED GYTJJA

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KEYWORDS

Cement stabilization, organic soils, gyttja, shearing behavior, triaxial testing

ABSTRACT

This paper examines the shearing behavior of cement stabilized soil and the effect of curing time, curing temperature, and cement content on the strength and stiffness development through a series of triaxial tests. The soil used is a marine, postglacial, soft, organic, clayey, silty gyttja, extracted from Aarhus University's soft soil test site situated in Randers Harbor, Denmark. A total of 34 unconsolidated undrained (UU) triaxial tests were performed where cement contents of 75 and 150 kg/m³, curing temperatures of 8 and 22 °C, and curing times from 1 to 56 days were examined. Furthermore, a total of 9 isotropically consolidated undrained (CIU) triaxial tests were performed on specimens with a cement content of 150 kg/m³ that were cured for 7, 14, and 28 days and sheared after isotropic compression to effective stresses of 30, 60, and 90 kPa.

The results from the UU tests show that the undrained shear strength and the stiffness of the cement stabilized gyttja generally increases with the cement content and the curing time and that the relationship between the undrained shear strength and the curing time is described well by a logarithmic expression. Analysis of the maturity of the specimens using an equivalent 20 °C curing time indicates that specimens would reach the same strength as maturity develops. In the CIU triaxial tests, the stress path reaches the tension cut-off without failing, indicating a tensile strength in the material. The cement stabilized gyttja obtains effective friction angles in the range of 30-33° with an effective cohesion that increases with the curing time. The observed tendencies from the performed laboratory tests match well with findings from the existing literature.

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INTRODUCTION

Dealing with soft organic soil deposits is a significant challenge when planning large infrastructure projects such as roads, railways, and port facilities. The soft soil is traditionally excavated and replaced with transported materials. This process results in large CO₂-emissions and requires large amounts of sand and gravel materials which are becoming scarce resources [1].

One method to minimize the CO₂-emissions and material consumption when encountering soft soils deposits for large infrastructure projects is the use of cement stabilization. Cement stabilization of soft soils such as gyttja is a soil stabilization technique that eliminates the need for removal and replacement of soft soils with transported materials. Instead, the existing soft soil can be improved on site by adding cement and thereby increasing the strength and stiffness of the soil. Upon the addition of cement, chemical reactions occur between the cement and the water present in the soil forming cementitious products such as calcium silicate hydrates (C-S-H) and calcium aluminate hydrates (C-A-H) that bind the soil particles together [2]. The technique of cement stabilization is common practice in Sweden, Norway, and Finland but is rarely used in Denmark, where only a handful of projects in the last couple of years have used cement stabilization [3].

This study aims to investigate the shearing behavior of cement stabilized gyttja through a series of triaxial tests where the effect of curing time, curing temperature, and cement content is examined. Cement contents of 75 and 150 kg/m³ are investigated, and the specimens have been cured at either 8 °C or 22 °C. The curing times investigated for the triaxial tests are 1, 3, 7, 14, 28, and 56 days.

TESTING MATERIAL AND SPECIMEN PREPARATION

The soil used for the experimental tests in this project is a marine, postglacial, soft, organic, clayey, silty gyttja, which has been extracted at a depth of 2 m from Aarhus University's soft soil test site situated at Randers Harbor, Denmark (UTM32E89 X: 565577 (m) Y: 6257802 (m)). The soil has a grey and slightly green color and contains numerous shells and shell fragments. Table 1 shows the determined classification parameters for the natural gyttja.

Table 110. Soil classification parameters - natural gyttja

Soil classification parameters			
Unit weight [kN/m ³]	15 [4]	Organic content [%]	7.7
Water content [%]	106 - 119	Undrained shear strength [kPa]	21 [4]
Plastic limit [%]	45	Clay fraction [%]	38.3
Liquid limit [%]	102	Silt fraction [%]	57.5
Plasticity index [%]	57	Sand and gravel fraction [%]	4.2

The natural water content was generally just above the liquid limit, and the liquid limit has been determined in accordance with DS/EN ISO 17892-12:2018 using the fall cone method, while the plastic limit is determined as the water content where a thread of 3 mm in diameter starts to crumble. The organic content has been determined based on the loss on ignition, and the particle size distribution is determined by a hydrometer analysis combined with dry sieving.

Cylindrical specimens with the dimensions 100×50 mm ($H/D=2$) were used for the triaxial tests performed in this study. The preparation procedure of the specimens is inspired by the guidelines in Helle et al. [5]. All specimens were prepared at the natural moisture content of the gyttja.

The natural gyttja was homogenized and remolded by stirring it in a mixer for 2×30 seconds. The cement was added as dry powder, and the mixing procedure of 2×30 seconds was then repeated. The cement used in this project is called FUTURECEM cement from Aalborg Portland A/S, which is a type of Portland cement produced with a reduced clinker content, which means that the CO_2 -emissions in the manufacturing process can be reduced up to 30 %. Cement contents of 75 and 150 kg/m^3 were used in this study. After the mixing of the gyttja and the cement, the stabilized soil was manually compacted into acrylic cylindrical molds using a wooden muddler. The soil was compacted in 6 layers of approximately the same height, and each layer was scored to ensure bonding between the layers.

The quality and the uniformity of the compaction was monitored by comparing the weight of the compacted specimens. A maximum deviation of 2.1 % in weight was measured for specimens with a cement content of 75 kg/m^3 , while a maximum deviation of 4.0 % was measured for specimens with a cement content of 150 kg/m^3 . The small deviations in the weight of the compacted soil indicate that the chosen compaction method ensured consistent and satisfying results.

During curing, the specimens were placed individually inside two closed plastic bags with a damp cloth on top of the sealed specimen to ensure a relative humidity of 100 %. The specimens were cured at either room temperature corresponding to approximately 22°C or at 8°C in the refrigerator. The gyttja used for the specimens that were cured at 8°C had been stored in a refrigerator for a minimum of 24 hours prior to mixing.

EXPERIMENTAL PROGRAM AND TEST METHODS

A series of 34 unconsolidated undrained (UU) triaxial tests were performed to analyze the influence of the curing time, curing temperature, and cement content on the strength and stiffness development for the cement stabilized gyttja. Due to the partially saturated nature of the cement stabilized

specimens, all of the UU tests were performed at a confining cell pressure of 100 kPa.

Additionally, a total of 9 isotropically consolidated undrained (CIU) triaxial tests were performed to determine the effective strength parameters and investigate the shearing behavior of the cement stabilized gyttja in more detail at low to intermediate stress levels of 30, 60, and 90 kPa. Full saturation of the specimens was ensured by applying a high back pressure of 760 kPa for 24 hours prior to testing resulting in B-values in the range of 0.88-0.96. For specimens that did not obtain a B-value exceeding 0.95, the B-value for three cell pressure increments did not show any significant increase and the specimens were therefore assumed to be fully saturated. Table 2 shows the experimental program for the performed triaxial tests.

Table 211. Experimental program for UU and CIU triaxial tests (*duplicates have been performed)

UU tests	Curing times in days					
	1	3	7	14	28	56
8 °C – 75 kg/m ³	X*	X	X	X	X*	X*
8 °C – 150 kg/m ³	X	X	X	X	X*	X*
22 °C – 75 kg/m ³	X	X	X	X	X	X*
22 °C – 150 kg/m ³	X*	X*	X	X	X*	X*
CIU tests						
22 °C – 150 kg/m ³ (30 kPa)	-	-	X	X	X	-
22 °C – 150 kg/m ³ (60 kPa)	-	-	X	X	X	-
22 °C – 150 kg/m ³ (90 kPa)	-	-	X	X	X	-

RESULTS AND ANALYSIS OF THE UU TRIAXIAL TESTS

To analyze the development in undrained shear strength of the tested specimens, the undrained shear strength is plotted against the curing time. The data is fitted with the following logarithmic expression proposed by Mitchell [6] to better visualize the tendencies.

$$c_u = c_u(d_0) + K \cdot \log\left(\frac{d}{d_0}\right) \quad (4.1)$$

Where $c_u(d_0)$ is the undrained shear strength at the curing time of d_0 days, and K is an empirical factor. The factor K has been determined as the value that provides the best fit for the test data and the factor is found to be affected by both the curing temperature and cement content. A reference curing time of $d_0 = 28$ days is used. Figure 1 shows that the logarithmic expression provides a reasonable fit for the four test series except at low curing times and for the two specimens cured for 56 days at 8 °C. These two specimens obtain a significantly larger strength than expected. This could be explained by an error in the sample preparation procedure, where the soil used for specimens

cured at 8 °C for 56 days was not precooled to 8 °C. From Figure 1 it is evident that the addition of cement to the gyttja ensures a significant increase in the undrained shear strength compared to the natural gyttja. The figure also shows that the undrained shear strength increases with an increase in the curing time and an increase in the cement content. This is explained by new cementitious materials being formed with time from the hydration and pozzolanic reactions, and a higher cement content results in the production of more C-S-H gel, which binds the soil particles together and increases the strength.

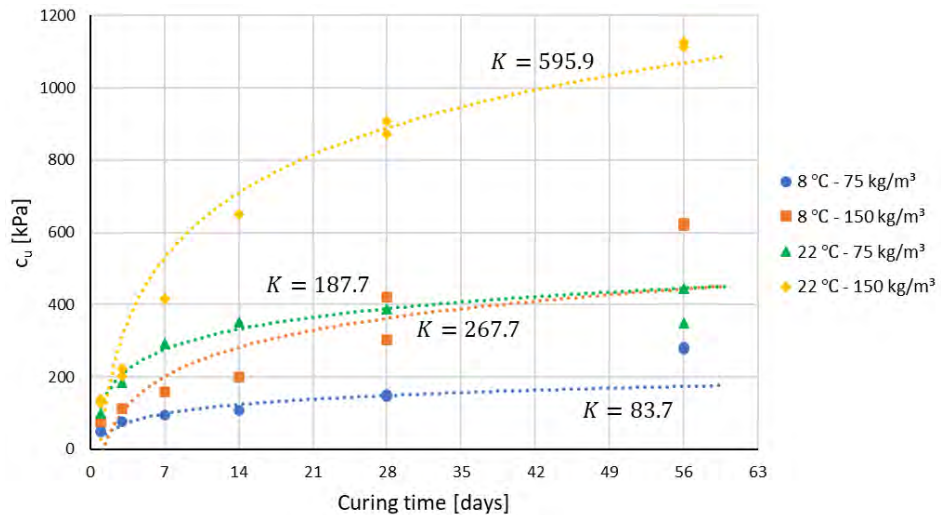


Figure 90. Undrained shear strength vs. curing time - UU triaxial tests

From the slope of the fitted curves, it is evident that the largest increase in the undrained shear strength takes place at the beginning of the curing period up to about 7 days of curing. After 7 days of curing, the slope of the curves gradually decreases. However, the slopes of the fitted curves for the four test series never reach zero and are always positive, indicating that the chemical reactions were still taking place even after 56 days of curing. Figure 1 also shows that specimens cured at 22 °C generally obtain a notably higher undrained shear strength than specimens cured at 8 °C. The curing temperature affects the rate of the chemical processes taking place as a higher temperature will speed up the chemical processes, resulting in a faster rate of strength increase with time [7]. Based on the ratios of K-values, the results indicate that the influence of curing temperature is independent of cement content, and that the influence of cement content is independent of curing temperature.

Maturity of tested specimens

In concrete technology, the maturity is measured as the equivalent curing time at 20 °C. A similar expression for the equivalent 20 °C curing time of cement

stabilized soil, t_{eq} , is proposed by Åhnberg and Holm [8] and modified by Vervoorn and Barros [9].

$$t_{eq} = \frac{(20 + K_t(T_c - 20))^4}{(20 + K_t(T_{ref} - 20))^4} \cdot t_c \quad (4.2)$$

Where K_t is an empirical factor not to be confused with the factor K from equation (4.1), where a value of 0.5 provides the best fit for Swedish soil types [8]. T_c is the actual curing temperature in °C, T_{ref} is the reference temperature of 20 °C, and t_c is the actual curing time in days. The factor K_t was found to be influenced by the cement content. For the cement content of 75 kg/m³, $K_t = 0.7$ was found to be suitable, while $K_t = 0.5$ was found to provide a good fit for the higher cement content of 150 kg/m³. The expression originally proposed by Åhnberg [10] and modified by Helle et al. [11] in equation (4.3) is used to describe the development in undrained shear strength as a function of the equivalent 20 °C curing time.

$$c_u = 0.3 \cdot \ln(t_{eq}) \cdot c_{u,28} \quad (4.3)$$

Where $c_{u,28}$ is taken as the undrained shear strength after 28 days of curing for the curing temperature of 22 °C in this study. The undrained shear strength is plotted against t_{eq} and fitted with equation (4.3) in Figure 2.

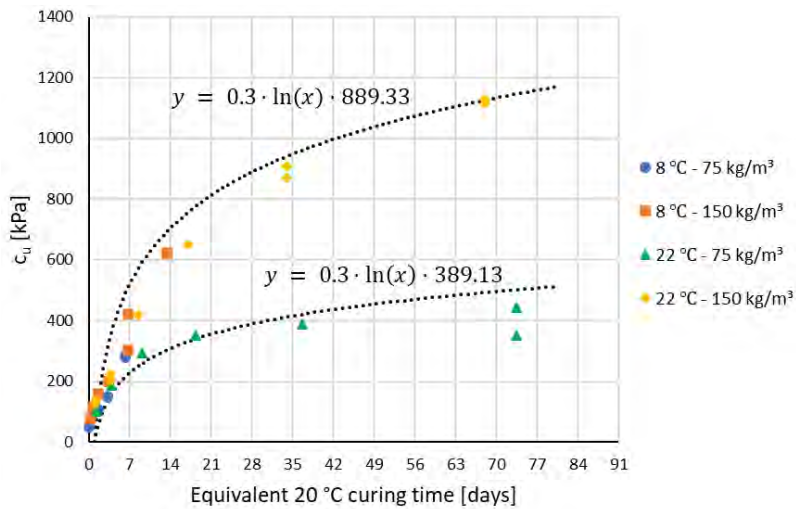


Figure 91. Undrained shear strength vs. equivalent 20 °C curing time - UU tests

Figure 2 shows that the development in undrained shear strength against the equivalent 20 °C curing time follows the logarithmic expression from equation (4.3) quite well. The graph also indicates that the reason for why specimens cured at 8 °C have not reached the same strength as specimens cured at 22 °C is the different level of maturity reached due to the chemical

reactions taking place being retarded at lower curing temperatures. However, the results suggest that specimens cured at 8 °C would reach the same strength and maturity as specimens cured at 22 °C for extended curing times.

The development in undrained shear strength for the performed UU tests, excluding the two outliers cured for 56 days at 8 °C, is now normalized with respect to the 28-day strength and compared with similar studies found in the literature. Tanderup [12] has investigated the undrained shear strength of FUTURECEM cement stabilized Aalborg gyttja with a cement content of 150 kg/m³ and cured at 20 °C using CIU triaxial tests. Åhnberg [10] has investigated the unconfined compressive strength of cement stabilized Holma gyttja and Löftabro clay with a cement content of 100 kg/m³ and specimens cured at 8 °C. The cement type used by Åhnberg [10] is a type of Portland cement commonly used in Sweden. Figure 3 shows the comparison of the normalized strength with results found in the literature.

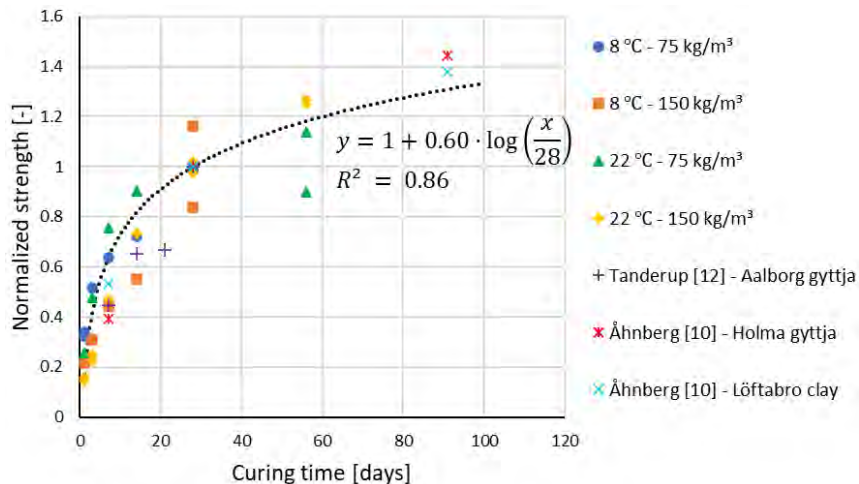


Figure 92. Normalized 28-day strength vs. curing time - comparison with the literature

The figure shows that when normalizing the strength of cement stabilized soil with the 28-day strength, the strength development follows the same trend relatively well regardless of cement content, soil type, curing temperature, and test type.

Secant modulus

The same overall trends observed in the development of the undrained shear strength are also observed for the secant modulus, E_{50} , which is determined as the slope between the origin and the point on the deviator stress vs. axial strain curve, where 50 % of the maximum deviator stress is mobilized. In Figure 4, the secant modulus is plotted against the undrained shear strength

and fitted with a linear regression passing through the origin. Figure 4 shows a good proportional correlation between the secant modulus and the undrained shear strength independent of curing temperature and cement content with a slight tendency of increasing E_{50}/c_u ratio at higher values of the undrained shear strength. This correlation applies to the tested gyttja and cannot necessarily be expanded to other soil and test types.

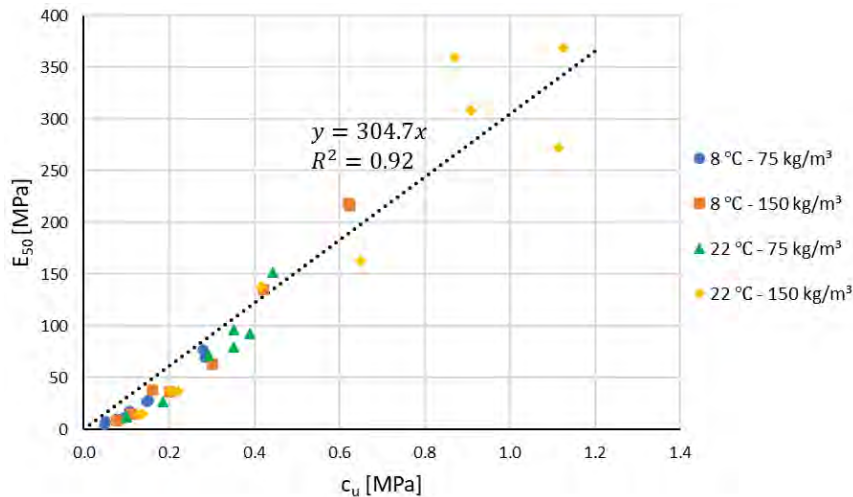


Figure 93. Secant modulus vs. undrained shear strength - UU triaxial tests

RESULTS AND ANALYSIS OF THE CIU TRIAXIAL TESTS

For all the performed CIU triaxial tests, a rather interesting stress-strain behavior is observed that was not present in the UU triaxial tests. After reaching an initial peak in the deviator stress at around 1-1.3 % axial strain for the performed tests, the deviator stress decreases shortly, whereafter it increases again and is relatively constant until the end of the test. A cement content of 150 kg/m³ has been used for the CIU triaxial tests. Figure 5 shows the deviator stress and excess pore water pressure plotted against the axial strain for the specimen that has been cured for 14 days and sheared after isotropic compression to an effective stress of 60 kPa.

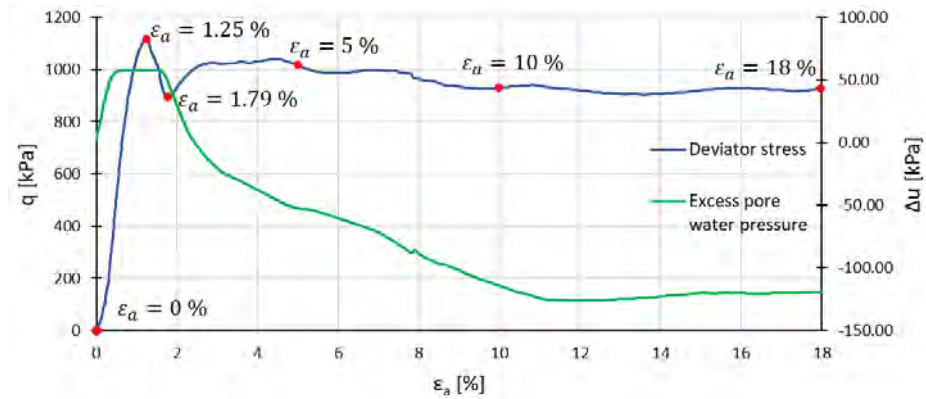


Figure 5. Deviator stress and excess pore water pressure vs. axial strain - 14 days of curing, isotropically consolidated to 60 kPa in effective stress, and a cement content of 150 kg/m³ - CIU triaxial test

Initially, a buildup of excess pore water pressure is observed followed by a plateau around the peak deviator stress where there is no change in Δu . After the peak, a negative excess pore water pressure is developed indicating a tendency to dilate. To analyze the failure mode, still images are pictured in Figure 6 for axial strains corresponding to the axial strains highlighted in Figure 5.

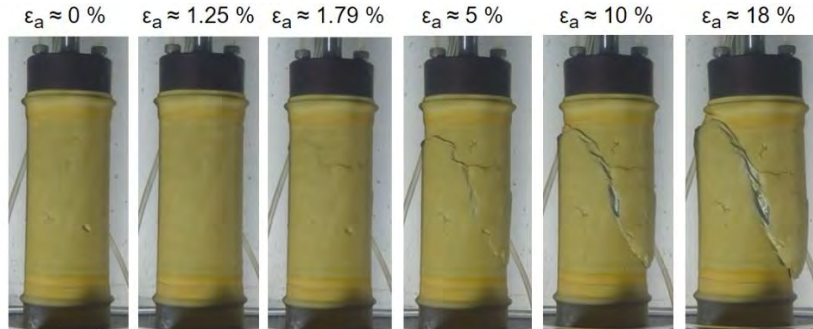


Figure 94. Stills from shearing phase - 14 days of curing, isotropically consolidated to 60 kPa in effective stress, and a cement content of 150 kg/m³ - CIU triaxial test

Figure 6 shows no visible failure mechanism at 1.25 % axial strain corresponding to the peak deviatoric stress. At 1.79 % axial strain, the first signs of a visible failure mechanism are observed. This failure mechanism develops into a single shear plane at larger strains. As proposed by Lee et al. [13], the mobilized strength and stiffness of the specimen up until the peak is governed by cementitious materials that bond the soil particles together. After the peak, a gradual breakdown of the internal structure will then take place where the cementitious bonds between the soil particles are broken down. When a clearly defined shear plane is formed, the shearing of the specimen is

governed by the friction between the two slipping halves of the specimen as opposed to the cementitious bonds.

Effective strength parameters

The effective strength parameters are determined by plotting the stress paths for specimens at the same curing time together. A straight line is then fitted to the data. Figure 7 below shows the stress paths for the three tests cured for 28 days and the fitted straight line. As the stress path prior to peak failure state has a slope of approximately 3, which corresponds to an effective friction angle of 90° and zero effective cohesion, it can be concluded that the tension cut-off has been reached. According to Åhnberg [10], it is likely for specimens consolidated at a low confining stress to reach the tension cut-off.

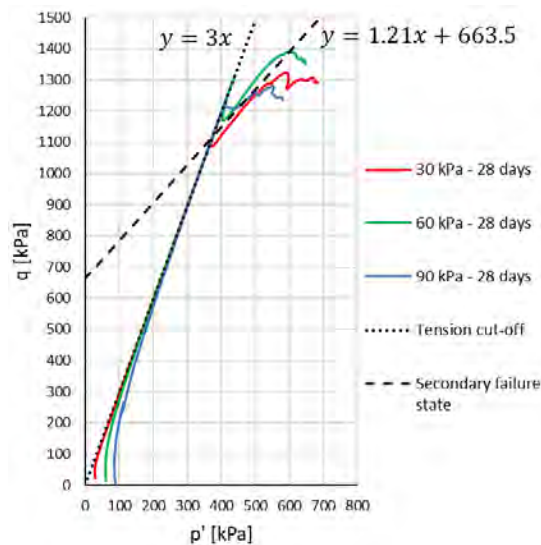


Figure 95. Deviator stress vs. effective mean stress - 28 days of curing, and a cement content of 150 kg/m^3 - CIU triaxial tests

Table 3 shows the effective strength parameters determined for the secondary failure state at large strains past the initial peak.

Table 3. Effective strength parameters - cement stabilized gyttja (150 kg/m^3)

Curing times [days]	$\phi'_{\text{secondary}} [^\circ]$	$c'_{\text{secondary}} [\text{kPa}]$
7	31.9	117
14	33.2	221
28	30.2	319

For the secondary failure state, the effective cohesion increases with the curing time, but the friction angle does not change much and does not seem to be affected by the curing time. This indicates proportionality between the undrained shear strength and the effective cohesion. Significantly higher undrained shear strengths are observed in the UU triaxial tests compared to

the CIU triaxial tests. The likely reason for this could be suction induced by unsaturated conditions in the UU triaxial tests.

DISCUSSION: LABORATORY VS. IN-SITU

The strength development of cement stabilized soil is largely influenced by the mixing procedure and the homogeneity obtained for the stabilized soil. Generally, the strength increases with homogenization, as the cement and the cementitious gels produced in the hydration are more evenly distributed in the soil [14]. Cement stabilized specimens mixed in the laboratory are usually more homogenous than what is achievable in the field [15]. This results in the strengths obtained in the laboratory being higher than what is normally obtained in the field. When comparing laboratory mixing with mixing in the field, it is important to keep in mind that the homogeneity achieved in the field can vary a lot depending on the soil type, the equipment used, and the overall in-situ mixing procedure [3]. Even for laboratory testing, the strength obtained can vary based on the exact mixing, compaction, and curing procedure [5]. For these reasons, it would be necessary to compare the tests on laboratory specimens with tests on in-situ mixed soil to predict the in-situ strength and stiffness. The curing temperature of cement stabilized columns in the field will also vary depending on the air temperature, depth below ground level, and the heat generated from the chemical reactions. When cement stabilizing in-situ, a significant increase in the temperature is observed for cement stabilized columns, which increases the rate of the chemical reactions. For cement stabilized specimens prepared in the laboratory, however, the effect of heat generation is negligible [10].

CONCLUSION

The results from the performed triaxial tests show that cement stabilized specimens with a cement content of 150 kg/m^3 obtain a significantly higher undrained shear strength and stiffness than specimens with a cement content of 75 kg/m^3 . For both cement contents, the shear strength increases with an increase in the curing time and the correlation is described well by the logarithmic relationship proposed by Mitchell [6]. Compared to the natural gyttja, a significant increase in the undrained shear strength is observed for both cement contents. Furthermore, it is observed that specimens cured at 22°C obtain higher shear strengths than specimens cured at 8°C . An investigation of maturity using the equivalent 20°C curing time indicates that specimens cured at 8°C will reach the same shear strength as specimens cured at 22°C as they mature. The low temperature retards the chemical reactions and makes them occur at a slower rate. The tendencies found for the strength and stiffness development match well with the findings from existing literature. Stress paths from the CIU triaxial tests indicate that the initial peak failure line corresponds to the tension cut-off. The fact that the specimens do

not fail when reaching the tension cut-off indicates that the material has a tensile strength due to the cementitious bonds. The secondary failure state shows effective friction angles of 30-33° that do not change significantly with the curing time, and an effective cohesion that increases with the curing time similar to results from the literature [10], [6].

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STRENGTH REDUCTION FACTORS ON LIME-CEMENT TREATED RIBS DUE TO RANDOM SPATIAL VARIABILITY AND CONSTRUCTION DEVIATION

Yutao Pan¹, Nils Martin Brandt¹, Rui Tao¹

KEYWORDS

cement treated rib; spatial variability; construction deviation; numerical modeling.

ABSTRACT

Lime-cement columns have been widely used as underground struts in deep excavations due to its effectiveness and flexibility. In practical design, a very low design value is usually applied regardless of the average strength of the column, although a much higher average strength from coring samples is available. This is mainly due to great uncertainties in properties of lime-cement treated soils. There are mainly two sources of uncertainties, namely random fluctuation of property, and construction deviation. The former is a result of uneven distribution of binder (lime-cement), in-situ water content and minerals. The latter is due to random tilting in column axes. When both uncertainties are combined, the improved ribs may become defective due to existence of random gaps and spatial variations.

In this study, the impact of construction deviation on the global strength of lime-cement treated ribs will be quantitatively evaluated using numerical methods. Random finite element analysis is used to replicate the random construction deviation. Then the global strength is evaluated based on the statistical results of random finite element method.

INTRODUCTION

Lime-cement treated ground has been widely used in many parts of the world to stabilize the ground [1]. The aim of this method is to increase the stiffness and strength of in-situ soil, reduce permeability and remove liquefaction potentials [2]. Common installation methods include jet-grouting and deep mixing.

Specifically, jet-grouting involves in-situ jetting of high-momentum binders (usually in form of wet cement/lime slurry or dry cement/lime powders), formulating the final mixture in a columnar structure. The final mixture of the binder and in-situ soils will then have hydration and pozzolanic reaction,

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gaining higher strength and stiffness over time. These columnar structures can be configured into different formations, such as walls, grids, ribs and slabs [3-5], to fulfil different functions. In Nordic practice, the deep mixed lime-cement columns are usually installed in rows of two or more columns (hereafter called ribs) in the to-be-excavated (passive) side of the retaining wall before excavation. The rib works as an in-situ strut to reduce the displacement and increase the resistance in passive zone together with the surrounding soils.

In such cases, the columns are usually idealized as perfectly vertical and cylindrical components. Designs are also done by assuming a lower bound undrained shear strength of around 150-300 kPa, despite the average strength of the cored samples are usually in the range of 800 kPa to several thousand kPa. The main reason of such conservative design is the great uncertainties in the properties of the ground. Although quality control works are usually done for individual columns, e.g. KPS, FOPS, coring. These measures can only show the uncertainties at the point level. The impact of such uncertainties on global performance of the improved ground structure member such as the deep mixed and jet-grouted ribs is largely untouched.

In reality, there are two major sources of uncertainties, i.e. random property fluctuations [6-7], and random construction deviations [4-5, 8]. The former is a result of the uneven distribution of mix ratio (soil solid, water content and stabilizer) [7, 9]. This can be attributed to insufficient mixing within a column (intra-column variation) and original spatial variation of in-situ mineralogy and water content (inter-column variation). This type of variation has been extensively studied in statistical works because the corresponding data retrieval technics are mature and usually required in the quality control phase [6,10]. The latter are usually less considered, although its impact may be more significant than the former [3]. In reality, the construction deviations may lead to continuous gaps in the treated structure [11], and that may greatly reduce the global stiffness and strength, making them less effective in resisting displacement of retaining wall. Pan et al. (2019) [11] quantified the impact of both random fluctuation and construction deviation on the global behavior of a laterally loaded cement treated slab, with a length of 21m, width of 15m and a thickness of 2m. This represents Singapore's practice, i.e. the whole excavated ground is fully treated. However, these uncertainties are not yet thoroughly considered for Nordic style lime-cement treated ribs, which are much slender and possibly more vulnerable to construction error.

In this study, random finite element method (RFEM) is implemented to consider the effect of construction deviation on the global performance of cement treated ribs.

CONSTRUCTION DEVIATION

The construction deviation of a jet-grouted column arises from the random inclination of column axis and the random variation of column diameter over depth [4,5,7], as shown in Figure 1. This random inclination can be divided into two random variables, that is, azimuth α and inclination angle β . The random variation of azimuth is due to the random orientation of the jet-grouting machine shaft and usually has no bias. In case where multiple shaft jet-grouting machine is used, the shafts in the same mixing operation should have strong correlation, because these shafts are oriented at the same direction. In Nordic context, single shaft jet-grouting is more common than multi-shaft jet-grouting. This makes the inclination angle differ from one mixing operation to the other, and the range of the inclination angle depends on the action standard and craftsmanship. Table 1 summarizes action standards in various parts of the world. In addition, the variation of column diameter stems from the vertical variation of soil property [12,13]. A random variable with a marginal normal distribution is used to simulate the diametric variation. The mean value is the target mean diameter of the columns, and the quality is controlled by the coefficient of variation (C.O.V). Previous studies show the C.O.V of jet-grouting's diameter is in the range of 0.05~0.2.

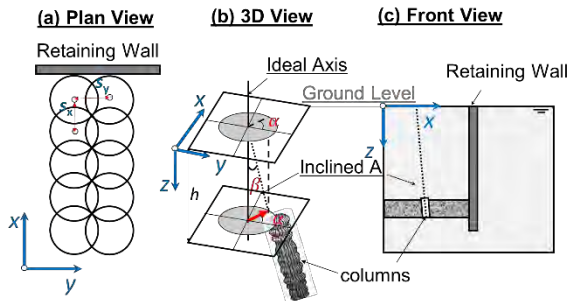


Figure 1 Illustration of construction deviation.

Table 1 Verticality requirements in different standards

Source	Type of technique used	Maximum Deviation from Verticality
ASCE Jet Grouting Guideline (2009)	Jet-grouting	1/100
Christopher and Jasperse (1989)	Deep mixing	1/100
Singapore Standard (2003)	Jet-grouting	1/75
Amos et al. (2008)	Drilled concrete piles	1/200**

Intuitively, this inclination leads to positioning error (deviation distance from the column axis and the ideal center) of the columns, which accumulates over depth. In cases where an action standard of 1/100 is used and the deep mixed ground is 20 m deep, the maximum allowable positioning error is around 0.2 m. When two adjacent columns tilt in opposite directions, the distance between two column centers increases by 0.4 m, which is higher than the preset overlap (0.2-0.3 m) between two adjacent columns. In such a case, untreated gaps will appear among the columns. These untreated gaps are characterized by much lower strength and stiffness than the treated zones, despite some random spillovers of stabilizers.

In this study, the construction deviation is characterized by three random variables, i.e. azimuth, inclination angle and C.O.V of diameter. The azimuth is a uniform distribution between $[0, 2\pi]$, whereas the inclination angle follows a uniform distribution between zero and the maximum allowable inclination, 1/100. An upper bound value of 0.2 is used for diametric C.O.V. Random fluctuation of the property field of treated ground is not considered in this study because the scale of fluctuation of treated column is usually much smaller than the diameter, and significant spatial averaging effect will limit its impact on the global performance.

NUMERICAL MODEL

Deep mixed columns with a mean column diameter of 1.0 m are used. A rectangular layout is used, as shown in Figure 1a. The transversal (s_y) and longitudinal (s_x) overlaps are 0.3m and 0.2m respectively. The treated zone consists of 2 rows of columns. The thickness of the treated zone is limited to 2 m due to lack of computational power, though the actual thickness are usually 5 to 10 meters.

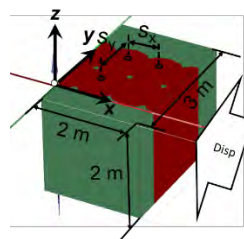


Figure 2 Defective treated ribs with construction deviation (yellow zone means treated, blue zone means untreated).

The FEM calculation zone has a dimension of 3m*2m*2m in x- (longitudinal), y- direction (transversal) and z- (vertical) directions. This covers a rib with 4 by 2 columns (Figure 2). It should be noted that an actual rib could have more than 20 columns. 4 columns are used here due to lack of computational capacity. Given that the ribs are subjected to similar loading conditions, the side boundaries are normally fixed. The bottom boundary is

also normally fixed, while the top boundary is free. A longitudinal displacement is exerted in the x -max boundary to ramp up the longitudinal load. Only longitudinal load is considered because the rib is usually basically longitudinally loaded when the wall moves inward. It can be expected that the retaining wall may exert an uneven displacement on the x -max boundary, and the other side is actually in contact with untreated soils. However, this study is only scoped to the longitudinal capacity of a defective lime-cement treated rib for simplicity. Further study is required for more realistic simulations.

Plaxis 3D ultimate is used for the simulation. A python script is generated to enable randomness simulation. Unlike conventional RFEM analyses which use user-defined material and random field generation at integration point level, this study uses generation of random volumes. Given the random azimuth and inclination angle of a columns, the column axis is determined. By generating cylinders with random diameter along the inclined axes, one can simulate the “calabash” shape of jet-grouted columns. These column volumes are then intersected with the calculation zone using a Boolean operation, and this distinguishes treated zone from untreated zone. One needs to set different models and parameters for both treated and untreated zones. Many advanced models were developed for both treated and untreated soils. However, in this study, an elastic-plastic model with Tresca criterion is used, as it corresponds well with the unconfined compressive test results which are extensively collected. An undrained shear strength of 1000 kPa and 100 kPa are used to represent treated and untreated soils, respectively. The 100 kPa is used to consider possible spillovers of binder [3]. The Elastic moduli is set to be 280 times undrained shear strength, following Lee et al. (2004) [15]. Table 2 summarizes the design configurations of ribs, namely depth of the column top and number of rows. The former is usually the same as the excavation level. The latter is an important parameter for column installations. Lower number of rows saves investment but leads to higher uncertainties. Cases No. 1, 2 and 3 explore the impact of depth, whereas Cases No. 2,4 and 5 aims at quantifying the effect of row numbers.

Table 2 Parametric study

Case No.	Depth	Number of rows	Max. Inclination Angle	C.O.V of diameter	Normalized Global Peak Strength (Q)		
					Average	C.O.V.	5% fractile
1	5	2	0.01	0.2	0.92	0.07	0.82
2*	10	2	0.01	0.2	0.88	0.08	0.77
3	20	2	0.01	0.2	0.75	0.16	0.56
4	10	3	0.01	0.2	0.92	0.06	0.82
5	10	4	0.01	0.2	0.93	0.06	0.83

*reference case to be compared with other case studies

Given the reference configuration, a typical realization of lime-cement-treated rib is shown in Figure 3a. As can be observed, significant gaps are observed between the two highlighted columns.

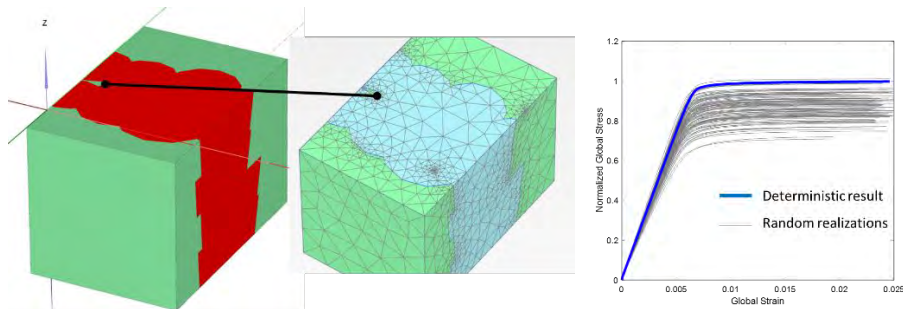


Figure 3 (a) a typical random realization and mesh; (b) Normalized global stress strain curves.

IMPACT OF RANDOM CONSTRUCTION DEVIATION ON GLOBAL PERFORMANCE

Deterministic analysis

The deterministic analysis is performed with zero inclination angle. In such case, the columns are perfectly overlapped. The normalized global stress-strain curve is plotted as the thick blue curve in Figure 3. The global stress is defined as the reaction force in x-direction divided by the cross-sectional area in the x-max boundary, whereas the global strain is the x-displacement of the x-max boundary divided by the longitudinal length of the rib. The global stress of the treated rib is normalized by the average strength of the rib, which is the weighted average undrained shear strength. This value is used because the global performance of a cement-treated rib is usually estimated using such weighted average value. In this case, the weighted average unconfined compressive strength (two times undrained shear strength) is calculated as $(1.7 \text{ m} \cdot 2000 \text{ kPa} + 1.3 \text{ m} \cdot 200 \text{ kPa}) / 3 \text{ m} = 1167 \text{ kPa}$. The normalized global stress (Q) reaches its peak at approximately 1.0 in deterministic case, indicating that the weighted average unconfined compressive strength is a good indicator of the global performance of the longitudinal capacity. This is rational because the rib is surrounded by much softer clays in transversal direction (y-direction), which can be easily displaced when the failure mechanism is formed. This would render the treated rib in an approximately unconfined case.

Random realizations

100 Monte Carlo simulations are done to evaluate for each case in Table 2. The random realizations are plotted as grey curves in Figure 3b. Most of the calculated 100 random realizations have lower peak strengths and stiffness than the deterministic counterpart. When the maximum inclination angle is set 1/100, the average normalized peak strength is around 0.88 and coefficient of variation is around 0.08. This average value is higher than the normalized global peak strength (0.62) of a wide cement treated slab [11], despite the

latter has a more significant local averaging effect. This is because the rib is not long enough, in cases there are more columns along the longitudinal directions. Higher possibility of untreated gaps is expected which leads to lower normalized global strength and higher Coefficient of variation. In addition, this simulation is based on Tresca criteria which does not account of strain softening.

Parametric study (Table 2) shows a clear reduction of average Q with increasing depth. With more rows in a rib, the uncertainty is reduced and a higher average and 5% fractile can be obtained. The 5% fractile can reach as low as 0.56 of the mean value when the depth reaches 20 m. This amounts an undrained shear strength of 560 kPa, which is still much higher than most practical design guidelines. However, given the limited number of columns simulated in this study, it is highly possible that a lower peak strength could be reached somewhere in a longer rib.

CONCLUSIONS AND LIMITATIONS

In this study, random finite element analysis on cement treated rib is used to quantify the impact of pure construction deviation on the global strength a cement treated rib. The result shows that the construction deviation makes the ribs defective and reduces the global strength to around 50% of its average strength. However, the current study is only limited to a small-scale numerical model on limited case studies. Further study is required to have a more comprehensive parametric study with more realistic dimensions of the ribs and strain-softening effect. Both factors would lead to a lower normalized global strength.

ACKNOWLEDGEMENT

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SURCHARGED EMBANKMENTS OVER PEAT IN NORTH-WEST IRELAND: PRIMARY CONSOLIDATION BEHAVIOUR

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KEYWORDS

Primary consolidation, instrumentation, peat, surcharging

ABSTRACT

This paper presents selected data relating to the primary consolidation of blanket peat under surcharged embankments on a road improvement scheme in Co. Donegal, Ireland, completed in 2022. The back-calculated compression index was found to be compatible with values for Scandinavian peats and in keeping with a well-known empirical prediction at equivalent moisture contents. Coefficients of consolidation, derived using Asaoka final primary consolidation settlements as the starting point, exceeded those inferred from piezometer data, but both values significantly exceeded oedometer-derived values at equivalent effective stress levels. Carlsten's (1988) method produced a reasonable estimate of time to 90% consolidation once used within the intended loading range. A particular focus of the paper is on the corrections/assumptions necessary to interpret primary consolidation parameters in large strain scenarios.

INTRODUCTION

Peat's low undrained shear strength, high compressibility and long-term creep behaviour render construction in peatlands extremely challenging [1,2]. As a result, peat has traditionally been excavated and replaced with more competent material, especially on large road schemes. However, given the prevalence of peat in certain countries, e.g. 33.5% of the land area in Finland, 18% in Canada, 17.2% in Ireland and Sweden [3] and the important contribution that intact peatland ecosystems make to carbon sequestration [4],

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the excavation of large volumes of peat is nowadays considered to be unsustainable.

A recent road improvement project in a blanket peat area in north-west Ireland, in which the excavate-and-replace approach was not feasible due to ecological constraints, afforded the opportunity to consider surcharging as a means of limiting post-construction settlements arising from the construction of a multi-stage embankment. Surcharging is a well-established ground improvement practice in soft inorganic soils, but there is limited experience of its use in peat soils internationally. Transport Infrastructure Ireland [5] does not currently permit surcharging in soils with high organic contents such as peat but facilitated it on this project on an exceptional basis. A general overview of the scheme is presented elsewhere [6]. In this paper, primary consolidation data, derived from an instrument cluster at one cross section along the road alignment, are presented. These data are interpreted in the context of experiences in other Irish and Scandinavian peats, and Carlsten's [7] primary consolidation prediction method. The paper has a particular focus on the challenges associated with interpreting primary consolidation data in large strain scenarios, drawing on the experiences of Arulrajah *et al.* [8] who monitored a trial embankment on marine clay in Singapore.

PROJECT LOCATION AND SCOPE

The project involved the redevelopment of a 4.5 km section of the N56 national secondary route, north of Glenties, Co. Donegal, in northwest Ireland (Fig. 1).

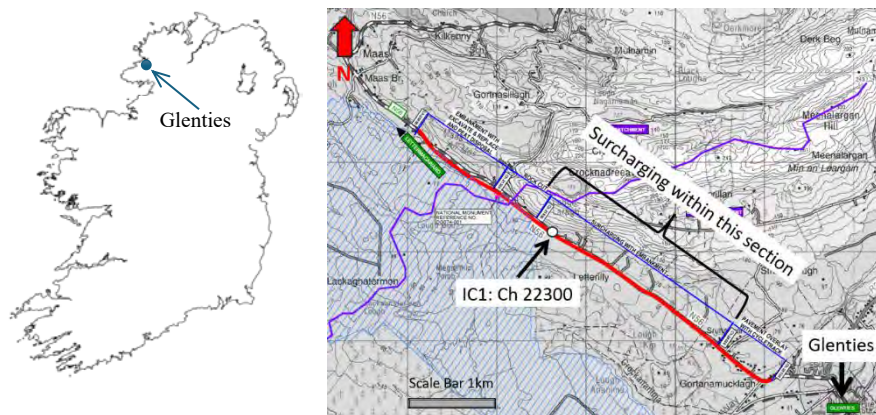


Fig. 1. (a) Glenties location; (b) plan of surcharging zones, IC1 location

A multi-stage surcharged embankment (Fig. 2a) was the preferred solution along ≈ 1.25 km of the road. In order to inform the embankment loading hold periods and assess embankment stability and peat settlements, six instrument

clusters (ICs) were established along the road alignment. Each IC incorporated a foundation settlement plate (FSP), a subsurface profile gauge (SSPG) a vibrating wire piezometer installed near mid-depth within the peat, and an inclinometer pair, one on each side of the road (Fig. 2b). The data presented in this paper relates to IC1 at Chainage 22300 along the scheme (Fig. 1b).

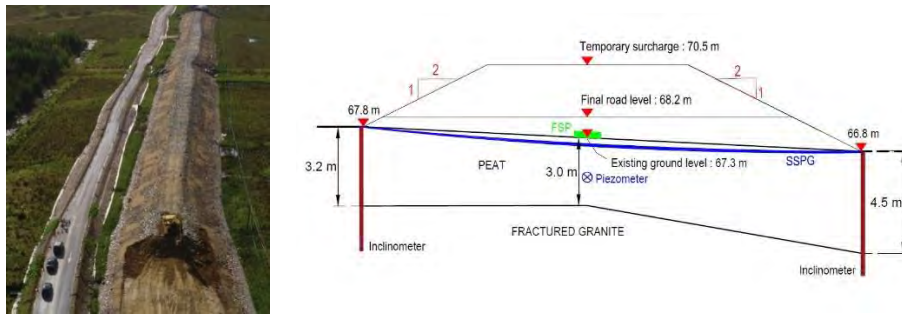


Fig. 2. (a) Surcharged embankment on N56; (b) Cross section schematic at IC1, including levels, peat thicknesses and instrumentation.

SITE CHARACTERISATION (AT IC1)

The following individual assessments of peat thickness were made at the centreline of the embankment at IC1: 3.5 m (gouge auger), 2.7 m (Russian sampler), 2.8 m (CPTu) and 3.3 m (CPT ball); an average peat thickness at the centreline of 3.0 m was adopted for subsequent interpretation (Fig. 2b). Peat thicknesses of 4.5 m and 3.2 m were inferred from inclinometers at the west and east sides respectively. The Russian sampler log indicated the existence of fractured granite beneath the peat, while the CPTu ball identified a thin sand layer 100 mm thick, both suggesting the potential for double drainage.

According to the Von Post [9] classification system, the peat at IC1 was categorised as H4-H7, with the degree of humification generally increasing with depth (Fig. 3a). It consisted of both fine and coarse fibres, with timber noted in the lower half. A broad spectrum of moisture contents (w) in the range 695-1400% was observed (average water content $\approx 980\%$), without an apparent trend with depth (Fig. 3b). CPT ball end resistance (q_{ball}) and pore water pressure measured at the shoulder position (u_2) are shown in Fig. 3c.

Three different methods were employed to determine the undrained shear strength (c_u) of the peat. A single direct simple shear (DSS) test from a specimen obtained from a block sample near IC1, consolidated to a normal stress of 7 kPa, gave $c_u = 6$ kPa. In addition, c_u was estimated from a correlation with shear wave velocity (V_s) (also plotted in Fig. 3b) and w , developed by Trafford and Long [10] from Irish, Scottish and Dutch peat sites (Eqn. 1):

$$c_u = 55.8 \left(\frac{V_s}{w} \right)^{0.683} \quad (1)$$

This approach yielded c_u values in the range 3-5 kPa. A separate assessment of c_u was made from CPT ball end resistances adopting $N_{t,ball} = 15$ [6], yielding values typically between 4-7 kPa.

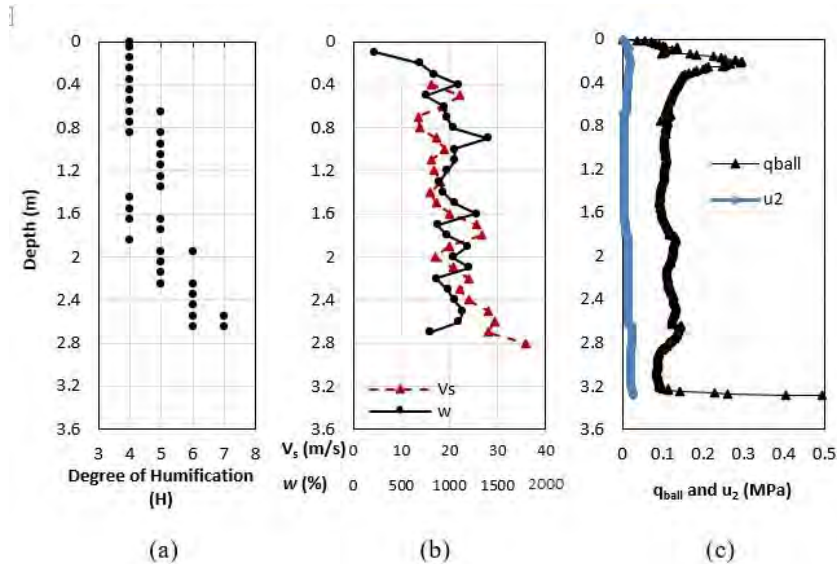


Fig. 3. Variation of (a) degree of humification, (b) water content (w) and shear wave velocity (V_s) and (c) CPT ball data with depth within peat

Six maintained load oedometer tests were performed on 80 mm diameter, 19 mm high peat specimens (from multiple locations) obtained from serrated-edge, thin-walled tubes. Sample depths ranged from 0.5-2.4 m and natural moisture contents from 520% to 1136%. The 24 h load stages were 2.5, 5, 7.5, 10, 20, 40, 80, 120 and 80 kPa. Two tests showed evidence of a yield pressure in the peat of between 5 and 10 kPa. Coefficients of consolidation (c_v) typically varied between 2-5 m²/yr at initial load stages, reducing to 0.2-0.6 m²/yr above 40 kPa.

STAGED EMBANKMENT CONSTRUCTION

The embankment at the location of IC1 was built in eight stages, each 0.5-1.0 m in height with a minimum hold period of 2 weeks, corresponding to an average filling rate of ≈ 0.14 m/week [6]. A minimum hold period of 90 days at maximum fill height was specified in order to achieve a degree of dissipation (U_v) of at least 90%. At IC1, the maximum fill height of 4.6 m

(including the fill displaced below the original ground level due to settlement) was maintained for 99 days prior to surcharge release to 2.2 m (Fig. 2b).

Fig. 4 illustrates the evolution of vertical total stress with time at IC1 due to embankment loading, calculated (for consistency) at the current level of the piezometer (initially 1.75 m below original ground level). In keeping with recommendations by Arulrajah *et al.* [8], this vertical total stress was corrected to allow for submergence of a significant portion of the embankment fill beneath the groundwater table (which was ≈ 0.6 m deep at IC1). The corrected total stresses are based on a measured fill dry density (γ_d) of 20.8 kN/m^3 (determined by *in situ* nuclear density meter), with the saturated density calculated using a specific gravity of 2.6 for the (largely) sandstone fill used.

SETTLEMENT MONITORING

General

The peat settlement arising from staged embankment loading, derived from both the FSP and SSPG (at the centre of the embankment), is also presented in Fig. 4. During the early stages, the initially rapid rate of settlement was observed to diminish significantly over the hold period. From Stage 4 onwards, the settlement-time trend for each stage became more uniform, which may be attributed to the cumulative effects of consolidation from previous load increments, and to the significant reduction in c_v with increasing vertical effective stress levels characteristic of peat [11,12]. The FSP and SSPG settlements (at the centre of the embankment in the latter case) of 1.39 m and 1.25 m respectively correspond reasonably well given the different measurement approaches involved (the SSPG tube was embedded in compacted gravel). Immediately prior to surcharge release, these settlements corresponded to vertical strains of $\approx 46\%$ and $\approx 41\%$ respectively. Some rebound in both settlement readings is observed upon release of the surcharge.

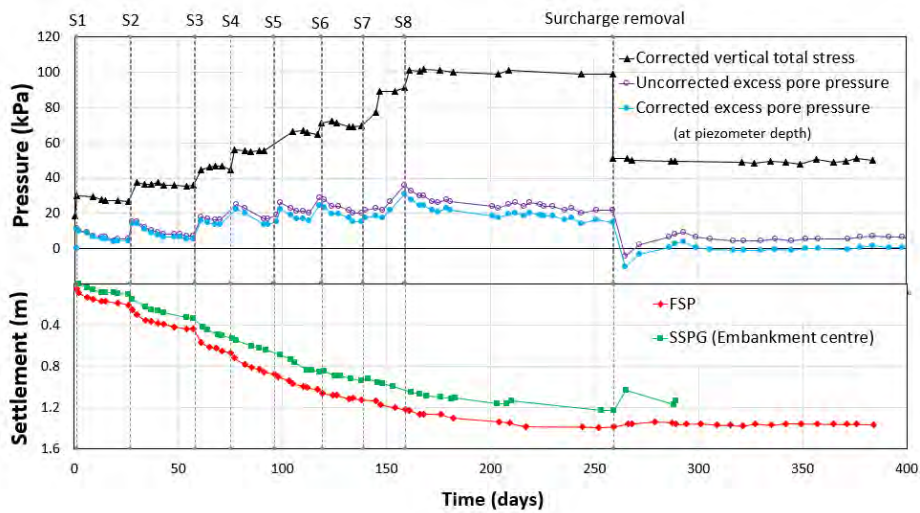


Fig. 4. Corrected total stress, uncorrected and corrected excess pore water pressure and FSP and SSPG (central) settlements due to multi-stage embankment loading

By way of comparison, Rodgers [13] stated that strains of 40-50% (assumed by the authors to exclude creep) are typical in normally consolidated (NC) Irish peats with moisture contents of 1000-1500% subjected to 2 m high embankments (40 kPa applied pressure). At IC1, equivalent strains required much higher applied pressures. Carlsten's [12] method for estimating relative compression (strain) from water content and applied loading, the basic version of which assumes peat to be normally consolidated, predicts $\approx 55\%$ strain after Stage 8. The method accommodates light overconsolidation by correcting the applied stress by subtracting the difference between the yield pressure and *in situ* vertical effective stress. Even with a correction as high as 10 kPa, the effect is merely 2-3%, so the method still overpredicts the measured strains.

The transverse surface settlement profile was assessed using the SSPG data. The profiles shown in Fig. 5 reflect the settlement at the end of each loading stage. The profile is relatively symmetrical, which is somewhat surprising given that gouge augers and inclinometers indicated a greater thickness of peat to the west of the embankment (Fig. 2b). The new road alignment did not encroach upon the previous alignment at this location.

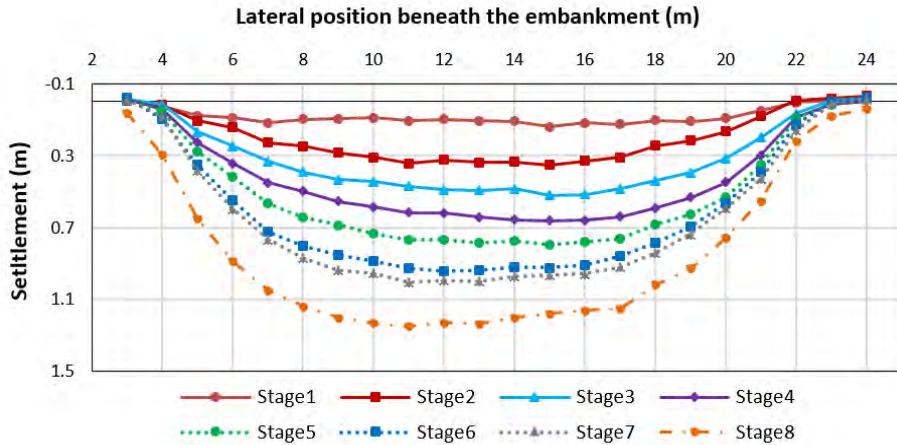


Fig. 5. SSPG settlement profiles at the end of stages 1-8 at IC1

Asaoka method and coefficient of consolidation

Given the greater frequency of the FSP data than the SSPG data, the Asaoka [14] method was applied to the former to estimate the final primary consolidation settlement at IC1. The method is applicable to situations where the load is constant and for which $U_v > 60\%$. Only stages 2 and 8 met the latter criterion (Fig. 4). Also, because the Asaoka method requires a consistent data frequency and the data were captured at irregular intervals, the settlement dataset was updated, using a linear interpolation approach, to achieve equal time intervals of ≈ 1 week. Arulrajah *et al.* [8] studied the effect of time interval on Asaoka predictions, and recommended a maximum interval which corresponded to 3% of the time to achieve 90% consolidation (t_{90}) within the marine clay. For IC1, the measured $t_{90} = 96$ days for stage 8 loading suggests a minimum recommended frequency of 3 days. The error in using 1 week intervals herein is not believed to be significant, based on their findings.

The Asaoka plots for stages 2 and 8 are shown in Fig. 6, with predicted final primary consolidation settlements of ≈ 0.454 m and ≈ 1.43 m respectively, inferred from the points of intersection of the two lines. The actual settlement at the end of stage 2 of ≈ 0.44 m and the settlement immediately before surcharge release of ≈ 1.39 m in Fig. 6, suggest that $U_v \approx 94\%$ and 97% for stages 2 and 8, respectively. An important inference from Arulrajah *et al.* [8] is that the settlement datasets reflecting high degrees of consolidation, as is the case here, will lead to more accurate final predicted settlements.

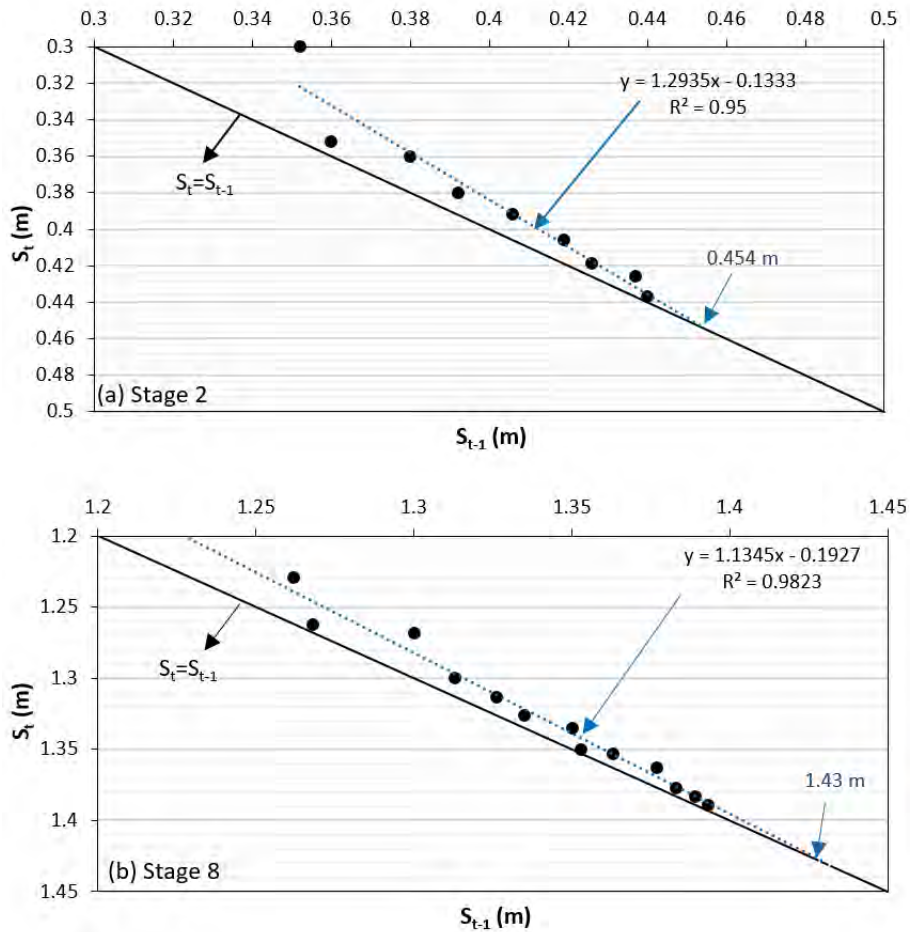


Fig. 6. Asaoka primary settlement plots for (a) Stage 2 and (b) Stage 8.

The entire embankment construction at IC1 took almost 163 days from the start of loading to the time at which its maximum fill level was achieved in stage 8. Using the Asaoka predicted final settlement of ≈ 1.43 m, a global c_v value (corresponding to a vertical effective stress increase from 7 to 82 kPa) was calculated by assuming that the full embankment height was attained in an equivalent single stage at the midpoint (81 days after commencement) of the actual loading period. Back-calculated values of c_v were determined based on (i) the value $U_v = 97\%$ inferred from settlement data, and (ii) the interpolated times to achieve $U_v = 90\%$, considering the difficulty in calculating the time factor T_v value reliably for U_v values approaching 100%.

Assuming double drainage of the peat, the drainage path length (d) for the entire embankment loading was calculated in two ways. The first method adopted an average layer thickness, i.e. $d = [3 - (1.39/2)]/2 = 1.15$ m.

Acknowledging that the load was assumed to be applied at 81 days, the second method used an average of the settlements at the midpoint of loading (0.78 m) and the end of the primary settlement (1.39 m), resulting in: $d = (3 - 1.08)/2 = 0.96$ m. Corresponding c_v values based on $U_v = 90\%$ are 4.3 and 2.9 m^2/year , respectively. Values based on $U_v = 97\%$ are 3.8 m^2/year and 2.6 m^2/year respectively.

The same calculation was conducted to determine the c_v value for stage 2. The corresponding vertical effective stress range (13-19 kPa) exceeds the peat's yield stress (σ'_{vy}) of ≈ 10 kPa estimated using $\sigma'_{vy} = 150/e_0$ [1] (with e_0 estimated from the average water content $\approx 980\%$ and $G_s = 1.5$ [15] assuming saturated conditions) and measured in two of the oedometer tests. Two U_v values (90% and 94%) and two drainage paths were considered based on the aforementioned logic. The c_v values were calculated as 30.8 m^2/yr and 33.2 m^2/yr ($d = 1.34$ m), and 23.5 m^2/yr and 25.3 m^2/yr ($d = 1.39$ m), for $U_v = 90\%$ and $U_v = 94\%$ respectively. These higher back-calculated c_v values reflect peat properties during an earlier stage in the loading history, when effective stresses were lower.

Carlsten's [12] method produced a reasonable estimate of t_{90} for stage 2 (25 days, versus 27 days observed in the field), but significantly underestimates t_{90} for the entire embankment (50 days, compared to 96 days in the field). The latter discrepancy may be the result of an attempt to extrapolate the method to applied pressures well in excess of the intended upper limit (50 kPa).

Compression index

Equation (2), used to back-calculate the compression index (C_c), was conservatively applied from Stage 3 to Stage 8 to guarantee that the settlement (ΔH) pertained to the NC state only.

$$\frac{\Delta H}{H_0} = \frac{C_c}{1+e_0} \log \left(\frac{\sigma'_{vs}}{\sigma'_{v2}} \right) \quad (2)$$

where σ'_{v2} is the effective stress at the end of Stage 2, σ'_{vs} is the maximum effective stress at the end of primary consolidation (as inferred from the Asaoka method). H_0 was taken as the layer thickness at the end of stage 2 ($3 - 0.44 \approx 2.6$ m). This yielded a C_c value of 9.1, tallying well with the estimate $C_c = w/100$ proposed by Mesri and Ajlouni [15] for fibrous peat. By way of comparison, Carlsten [7] and Long *et al.* [16] report value in the range 6-8 and 6-9 for Swedish and Norwegian peats respectively, for $w \approx 1000\%$.

PORE PRESSURES AND COEFFICIENT OF CONSOLIDATION

The piezometer was installed at a depth of 1.75 m within the peat layer, slightly below the assumed mid-depth, directly beneath the foundation settlement plate. In interpreting pore pressures, it was necessary to allow for a reducing instrument position over time as it settled in tandem with the peat, as

highlighted by Arulrajah *et al.* [8]. In the absence of this adjustment, the interpreted excess pore pressures would exceed the actual values, leading to an underestimation of U_v . Given the relatively uniform peat consistency with depth (uniform profiles of w and V_s in Fig. 2b), and a very modest decay with depth (over the 3.0 m peat thickness) of imposed total stress from the embankment (calculated using Boussinesq theory), its level was corrected by $\approx 42\%$ (i.e. $[3 - 1.75] \text{ m} / 3 \text{ m}$) of the surface settlement of the peat at any time. The excess pore pressure data, corrected on this basis, are included in Fig. 4.

The corrected excess pore water pressure exhibited a sharp initial increase during the earliest part of each load increment, followed by a gradual decline until the end of the loading stage, associated with continuous soil compression. After removing the maximum surcharge, a temporary negative corrected excess pore water pressure was measured for about 3 weeks, related to the swelling of peat, before reverting to zero.

The number of data points and/or the degree of dissipation was insufficient to make separate estimates of c_v for each of the stages; with settlement-log time graphs generally linear. However, a global estimate of c_v was made from the corrected excess pore pressures, assuming as before that the maximum load was applied in a single increment at the midpoint of staged construction. The calculation of U_v using this approach required an estimate of the excess pore water pressure that would have arisen had the load been applied in a single increment. To this end, the increases in excess corrected pore pressure (Δu) corresponding to the increases in corrected total stress ($\Delta \sigma$) are compared, at the outset of each stage, in Fig. 7. The data for stage 7 were difficult to interpret and were excluded. Both values are in reasonably good agreement; Δu falls slightly below $\Delta \sigma$ for five of the stages but this may be a result of a lag between filling and the first pore pressure measurement, in which case the peak Δu is likely to have been missed. On the basis of Fig. 7, it was deemed reasonable to assume that the excess corrected pore pressure increment corresponding to a single stage corresponds approximately to the corrected total stress increment of 90 kPa (i.e. assuming Skempton's pore pressure coefficient $A \approx 1$).

Arulrajah *et al.* [8] also highlighted the need to adjust U_v corresponding to an individual piezometer depth to reflect the mean level of dissipation over the entire layer. A standard double-drainage isochrone, based on Terzaghi's theory of primary consolidation, was used to effect the adjustment. Again considering the two possible values of d , the average U_v for the whole peat layer was calculated as 88%, with corresponding $c_v \approx 1.52 \text{ m}^2/\text{year}$ ($d=0.96 \text{ m}$) and $2.2 \text{ m}^2/\text{year}$ ($d=1.15 \text{ m}$). These values are lower than the Asaoka-derived values, an outcome also reported by Arulrajah *et al.* [8], who attributed this to the stress-strain non-linearity of the soil. It is clear that there are many more assumptions/corrections involved in the derivation of c_v from pore pressures,

rendering the Asoaka approach preferable where the data requirements are met.

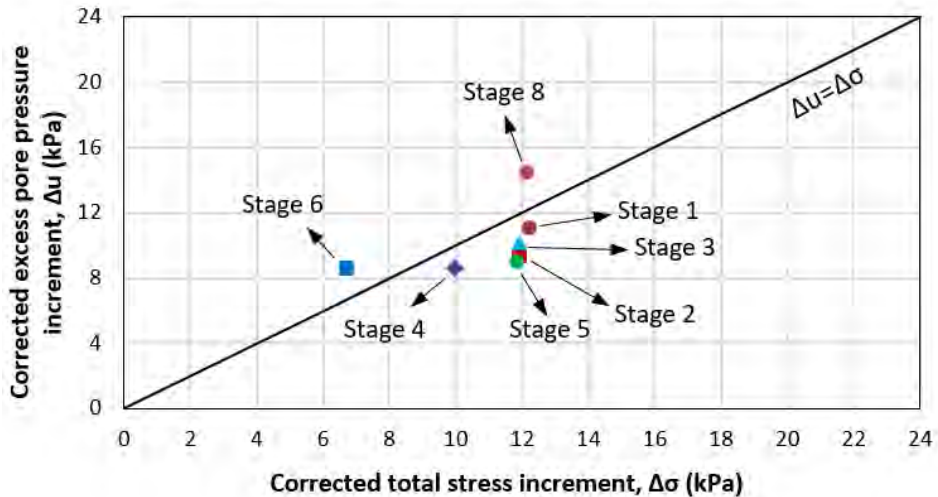


Figure 7. Variation of pore water pressure changes corresponding to the corrected total stress at IC1

Field values of c_v (derived from either the Asaoka method or pore pressures) significantly exceed the oedometer values. The field c_v value for stage 2 of 23.5-33.2 m²/year exceeds the oedometer range 2-5 m²/year by an order of magnitude at low stresses, while the values at higher stresses, ($c_v = 1.52 - 4.3$ m²/year, depending on interpretation) also exceed the oedometer values of 0.2 - 0.6 m²/year. This indicates that the oedometer values may significantly overestimate the duration for a given extent of peat consolidation in the field.

CONCLUSIONS

Primary consolidation parameters for a blanket bog peat have been back-figured from settlements and pore pressures measured at one location during the construction of a multi-staged embankment in north-west Ireland. The following conclusions have been drawn (for the IC1 location):

- Peat strains in excess of 40% were recorded as a result of a vertical effective stress increase of 82 kPa; these were lower than predicted by the Carlsten [12] method.
- The back-calculated compression index corresponds well with that measured for Norwegian and Swedish peats at corresponding *in situ* water contents, and is also predicted well by the well-known Mesri and Ajlouni [15] correlation.

- Asoaka-derived c_v values exceed those derived from piezometer data, in keeping a comparison of both methodologies at a marine clay site in Singapore [8]. However, the number of assumptions necessary to derive c_v in the latter case, as highlighted in this paper, is believed to render it an inferior approach. A key uncertainty in relation to the position of a piezometer within a layer subject to large strains could be removed by installing a companion extensometer. The Carlsten [12] method appears to predict t_{90} reasonably well for pressures below its 50 kPa limit of applicability, but may underpredict t_{90} at higher pressures.
- Field values of c_v exceed laboratory values significantly at equivalent stress levels. As a result, designers using laboratory c_v values may recommend longer staged embankment hold periods than are actually necessary, unless performance-monitoring instrumentation is used. In future, piezocone or piezoball dissipation tests may serve as a better means of assessing excess pore pressure dissipation rates.

Primary consolidation parameters for all six ICs will be presented in a future paper. In addition, the potential of surcharging to reduce long-term creep settlements is being assessed by the University of Galway, through long-term monitoring of the FSPs and SSPGs at the N56 site, in conjunction with numerical modelling and a programme of oedometer testing on peat, both modelling the surcharging process.

ACKNOWLEDGEMENT

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TANGENVIKA JERNBANEBRU, TEST PILING OF LARGE DIAMETER END BEARING PILES

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KEYWORDS

Large diameter end bearing piles, Test piling, Sustainable foundations, Composite steel/concrete piles, Fatigue.

ABSTRACT

Tangenvika jernbanebru (Railway Bridge) is to be completed in 2027, and will be Norway's longest railway bridge, 1026 meters. It is part of an upgrading of the railway line from Oslo to Hamar, reducing the travel time to under 1 hour, designing for double track and 250 km per hour, maximum speed.

It is a concrete bridge and consist of sixteen axes, where each span is 70 meters, but end spans are less, 60 and 56 m. The foundation of the bridge in the lake, will mainly rely on large diameter end bearing piles.

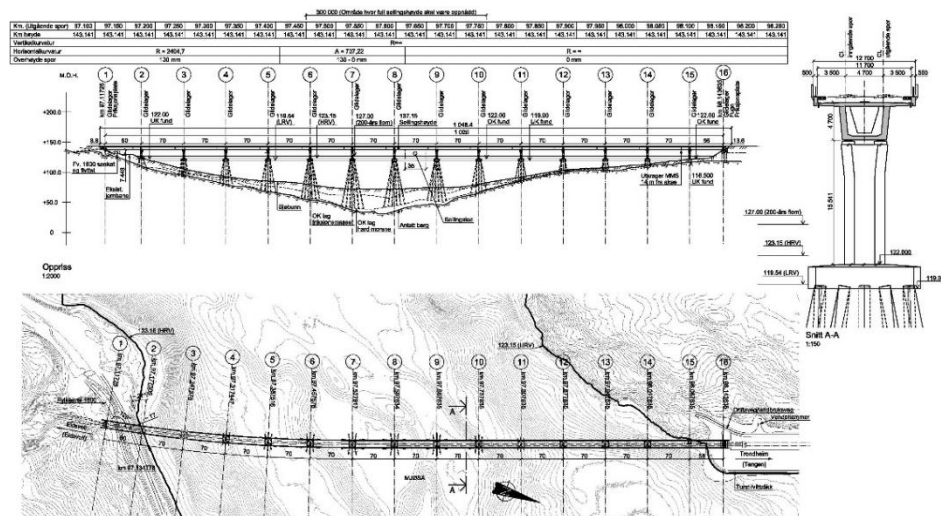


Figure 96 Tangenvika railway bridge, elevation, plan and section

Driven steel piles with diameter 1430 mm will make the foundations at 5 axes in the middle, where the sediments over bedrock are thickest and the lake is

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deepest. Drilled steel piles of diameter 1016 mm will make foundations at 7 axes.

The piles will be concrete filled and are designed as composite columns according to Eurocode EN 1994-2.

Based on the soil investigations, there were significant uncertainties related to the pile driving process and bearing capacity for the driven piles. Due to the inclusion of the steel casing in the structural capacity, quite detailed information about pile driving was necessary for fatigue assessments.

Test piling for the driven piles were conducted in November 2023. This proved the drivability to be better than anticipated. The number of blows combined with stresses in the steel casings, estimated with PDA/CAPWAP, were acceptable in regards of fatigue.

Results further indicated that end-bearing capacity estimated directly with PDA/CAPWAP underestimated the bearing capacity by a factor of at least 4. This is believed to be related to the limited mobilization of end bearing capacity, due to small displacement (relative to the pile diameter) per blow of the hammer.

The test piles could be driven deeper than first anticipated and proved it possible to achieve a higher calculated bearing capacity. The pile foundations were optimised, reducing number of piles from 12 to 10 piles per axis in most axes, giving a reduction of ten piles in total. This equals close to 600 m reduced piles length, including sacrificial test piles, corresponding to approximately 2000-ton CO₂-equivalents.

This paper will in more detail present experiences from the test piling.

INTRODUCTION

Implenia Norge has taken on a design-build contract for Bane Nor, and Norconsult is Implenia's partner for designing the bridge, hereunder the piles.

Tangenvika jernbanebru will span across a bay in Mjøsa, Norway's largest lake. The railway will deviate from its previous route in the landscape, allowing for double tracks and curves that facilitate higher speeds.

The superstructure of the bridge will be made of prestressed concrete. The substructure is founded directly on bedrock in axis 1 and 15. In axis 2 and 16 so-called steel core piles, "stålkjernepel"¹, are used.

Axes 3-14 are designed with free-standing pile groups with large diameter end bearing piles. The large diameter piles in these axes are concreted steel pipe

¹ "Stålkjernepel" is a widely used pile type in Norway, with a steel pile in the middle carrying the load, and a casing filled with concrete for protection against corrosion. The concrete is included in the buckling capacity.

casings, designed as composite steel and concrete piles. The steel casings are designed to manage 10 mm corrosion over the service life of 100 years.

Drilled large diameter piles

In axis 3-4 and 10-14 the casings with diameter 1016 mm (28 mm and 40 mm thickness) are drilled to bedrock with RC-drilling (Reversed Circulation), mainly due to limited soil cover and low soil stability at some locations. The installation of drilled piles started in September 2023. For the drilled piles ring crowns and RC drilling system is used.

Driven piles

In axis 5-9 pile casings with diameter 1430 mm (34 mm thickness) will be installed by driving down to firm moraine or bedrock. The production installation of driven piles will begin in summer 2024. Test piling was conducted late in the autumn in 2023.

METHOD FOR EXECUTION OF TEST PILING

Pile testing was carried out with the aim of obtaining more information and basis related to:

- Driveability and driving resistance, especially through layers with uncertain and possibly high strength.
- Geotechnical bearing capacity, what can be demonstrated by dynamic impact testing (PDA/CAPWAP) and how this correlates with theoretically calculated static bearing capacity according to “Bearing capacity based on new empirical relationships against CPT” in chapter 4.2.1 in Pelevedningen 2019 [3].
- Occurring driving stresses, stress interval and number of cycles define how much the steel tubes are subjected to the driving process in terms of fatigue.
- To provide a basis for developing a procedure for driving, stopping criteria and chiselling of the piles included in the bridge construction.

It was decided to place test piles near to axis 8 and 9. These test piles are horizontal piles and will not be used as a part of the permanent bridge. The piles were placed as near as possible to preliminary ground investigation boreholes.

The two test pile lengths were to be a minimum of appr. 80 meters near axis 8 and appr. 85 meters near axis 9. These lengths include the pile tip.

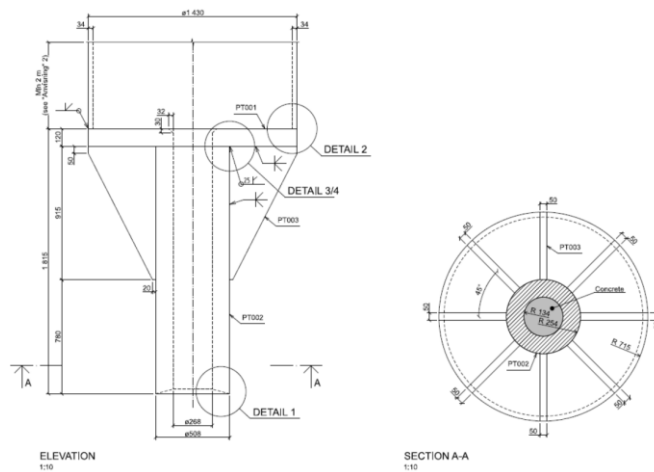


Figure 97 Dimensions of hollow concreted pile tip

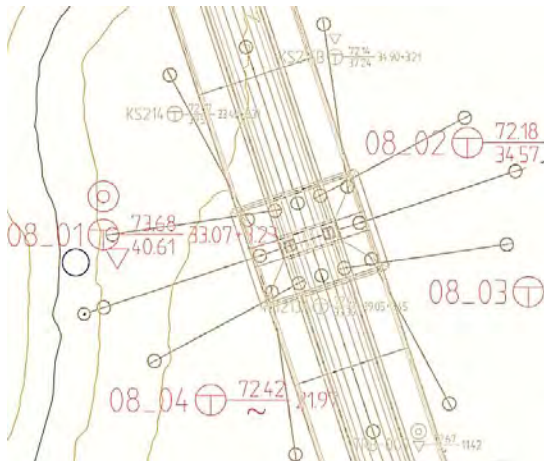


Figure 98 Test pile in axis 8 marked by the blue ring.

From the seabed, mapped with multibeam echo sounder, registered at approximately elevation +74.5, the interpreted layering can be summarized in axis 8 as:

- 0 - 7 m: Mud/clay/silt
- 7 - 26 m: Sand, with varying amounts of clay, silt, gravel, and small stones
- 26 - 34 m: Moraine
- 34 m: Bedrock - assumed to be of good quality

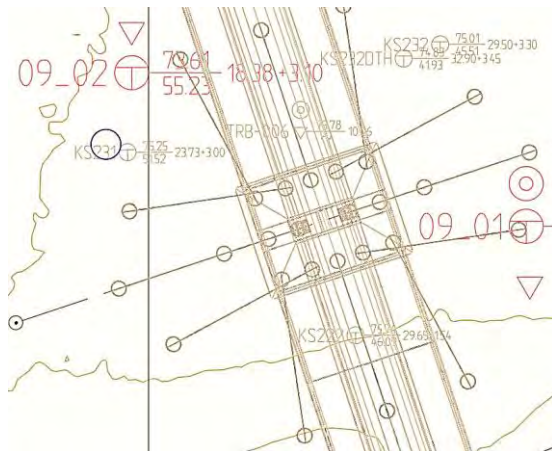


Figure 99 Test pile in axis 9 marked by the blue ring.

The ground conditions in axis 9 are briefly described, starting at the seabed at elevation +75,5 as:

- 0 - 6 m: Mud/clay/silt
- 6 - 22.5 m: Sand, with varying amounts of clay, silt, gravel, and small stones
- 22.5 - 24 m: Moraine
- 24 m: Bedrock - assumed to be of good quality

Hammer used for test piling

The S-350 Hydraulic Hammer is utilized for pile driving in the project. This hydraulic impact hammer is designed to provide high energy transfer during the pile installation process. It operates with an acceleration of 2g when set to full energy on the control panel. The hydraulic hammer's energy output is displayed on a monitor, allowing the operators to monitor and control the force applied to the pile during driving.

Pile driving procedure

The piles were driven water-filled to a minimum level with the water level in Mjøsa at the time of execution. To limit the size of tensile stresses occurring in the piles during driving, the supplied energy was gradually increased as the driving resistance increased.

GROUND INVESTIGATIONS AND RESULTS OF TEST PILING

Axis 8

Preliminary soil geotechnical investigations (borehole 08_01) is situated in a horizontal distance of only 5 m to the position of the test pile in axis 8. The depth to bedrock is assumed to be smaller in the location of the test pile compared to the borehole. The geotechnical investigations in borehole 08_01

included soil sampling and Cone Penetration Testing with pore water pressure (CPTu) executed by a sonic drilling rig on a barge.

The CPTu was performed with predrilling in between when the resistance was too large for further penetration due to risk of buckling of drill rods. The soil sampling showed that there was a clear relationship between significant amount of coarse gravel and/or cobbles in the sand layer and the need for predrilling. The size of this material was believed to cause an unrealistically high cone tip resistance (in regards of piling), due to the size relation to the cone (of size 15 cm²). In borehole 08_01 there were at times registered tip cone resistance close to 50 MPa in the soil layer mostly believed to consist of medium dense to dense sand. The high tip resistance registered at times, gave uncertainties in how much resistance to pile driving these predrilled sections would induce.

The ground investigations also included the method that in Norway is referred to as total soundings. This is a method where a rotating drill bit is pushed into the ground with recording of force needed for penetration and penetration rate. When the force needed for penetration gets high enough the standard procedure is to first increase the rotational speed, then use flushing with water and eventually introduce the use of a top hammer. The method is therefore suited for drilling both in soil and bedrock. See also [5].

The registered blows per meter and average hammer energy utilized for the test pile, is compared with the soil investigation results in borehole 08_01 in Figure 100. As earlier stated, the depth to bedrock is believed to be somewhat shorter for the test pile, so the pile is assumed to be driven with the tip very close to bedrock or most likely partially with the tip on bedrock.

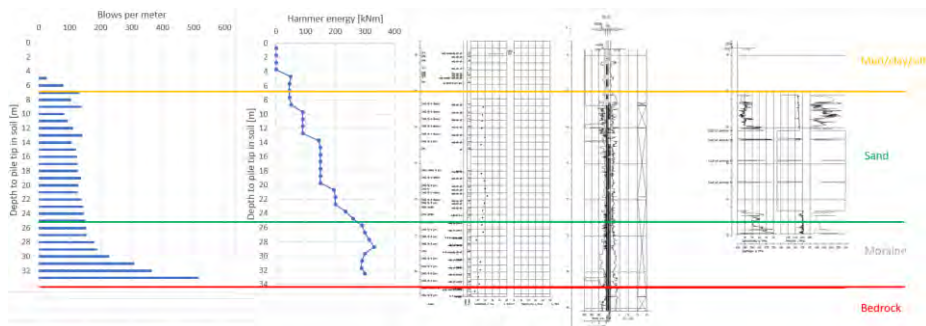


Figure 100 Blows per meter and average hammer energy compared with soil investigations in nearby borehole. Included interpretation of soil layering. Test pile axis 8

After self-penetration the test pile was instrumented for continuous PDA-measurements. The initial driving was carried out to a depth of approximately 31 m in soil before restriking and driving of the last 2 m was carried out 7 days later, to investigate the effect of time on skin friction.

Axis 9

There were preliminary ground investigations consisting of one total sounding. This is borehole KS231 situated in a horizontal distance of only 4 m from the position of the test pile in axis 9.

The registered blows per meter and average hammer energy utilized for the test pile, is compared with the total sounding performed in borehole KS231 in Figure 101. The depth to bedrock is believed to be somewhat shorter for the test pile, so the pile is assumed to be driven with the tip partially on bedrock after initial driving.

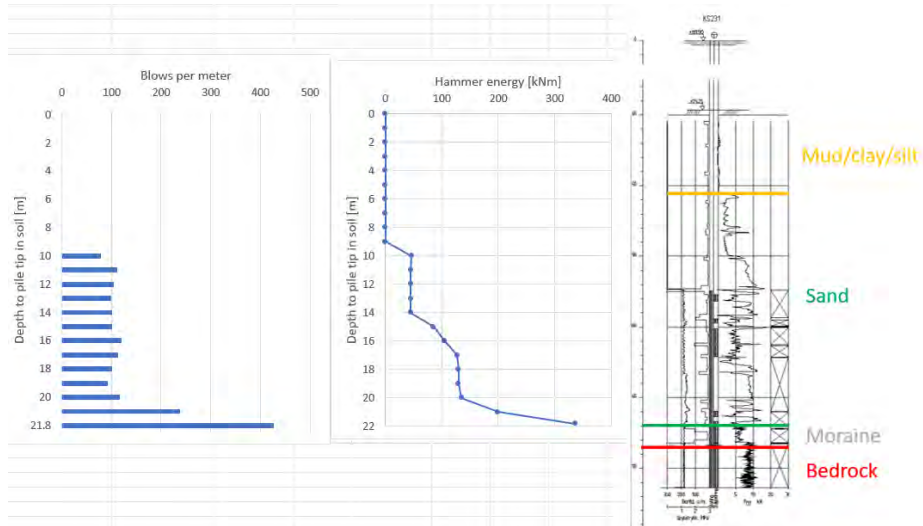


Figure 101 Blows per meter and average hammer energy compared with soil investigations in nearby borehole. Included interpretation of soil layering. Test pile axis 9

After self-penetration the test pile was instrumented using continuous PDA-measurements. After initial driving, restriking, and chiselling into bedrock were performed 2 days later. Displacement, hammer energy and accumulated number of blows versus chiselling depth are shown in Figure 102.

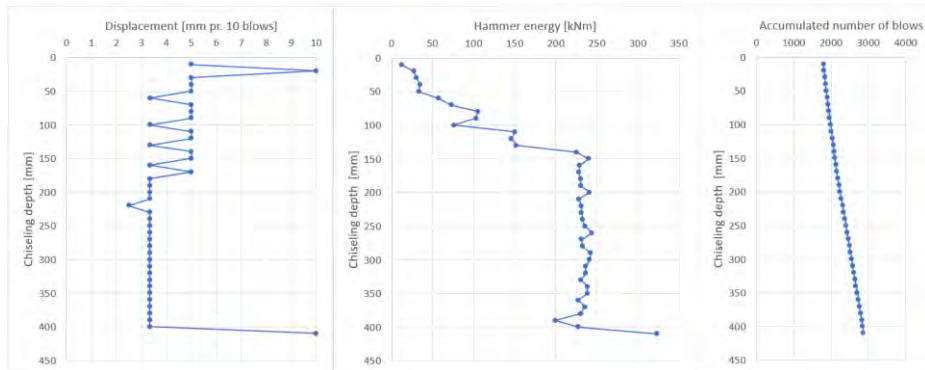


Figure 102 Displacement, hammer energy and accumulated number of blows versus chiselling depth into bedrock

INTERPRETATION OF RESULTS FROM TEST PILING

Axis 8

CAPWAP-analyses were performed on the PDA-data from single blows in various depths after the pile tip had reached what was assumed to be moraine. The estimated bearing capacity from skin friction and tip resistance is shown in Table 12.

Table 12 Estimated bearing capacity based on CAPWAP-analyses test pile axis 8

Pile length in soil [m]	Skin friction resistance [MN]	Tip resistance [MN]	Total resistance [MN]
25,5	4,5	3,9	8,4
28,5	5,9	3,2	9,1
30,5	7,6	4,6	12,2
32,5	11,1	10,6	21,7

The displacement per blow on the analysed blows are recorded to vary from approximately 2 mm to 7 mm. Due to the pile diameter being as large as 1430 mm the tip resistance is assumed to be highly underestimated according to [1] which states that “a pile tip displacement up to 10 percent of the pile diameter may be required for full mobilizations in both sand and clay soils”. Further, it is recommended in [1] that the relationship between end bearing and displacement is assumed to be as shown in Figure 103. With 2 mm / 1430 mm = 0,0014 and 7 mm / 1430 mm = 0,0049, this gives Q/Q_p varying from 0,18 to 0,33 respectively. This equals to an expected tip resistance between 3 to 5 times larger than what was mobilized and thus estimated with PDA/CAPWAP.

z/D	Q/Q_p
0.002	0.25
0.013	0.50
0.042	0.75
0.073	0.90
0.100	1.00

Figure 103 Recommended end bearing – displacement curve according to [1]

Utilizing a method presented by Kjell Karlsrud at “Geoteknikdagen 2023” [2] the actual end bearing capacity can be estimated more accurately from the CAPWAP-analyses. The CAPWAP-analyses estimate the distribution between skin friction and end bearing, but also the assumed distribution of the skin friction along the length of the pile. Unlike end bearing, skin friction is assumed to be close to fully mobilized.

The NGI99-method can be used for calculating the bearing capacity of a pile. The method is presented in [3]. Both skin friction and end bearing are determined based on tip resistance from Cone Penetration Test (CPT). When the skin friction is estimated from CAPWAP-analyses it is therefore possible to back calculate the corresponding tip resistance from CPT, $q_{c,CPT}$. This is shown in Figure 104. The best fit to measured tip resistance with CPTU in nearby borehole is achieved combining the largest skin friction estimated with CAPWAP for the separate blows. The figure further shows the importance of having data on skin friction after restriking some days after initial driving for the best estimate. The recommended time for restriking for this purpose is 7-10 days as that is the time the full-scale tests (with measurements of bearing capacity) behind NGI99 apparently is based on.

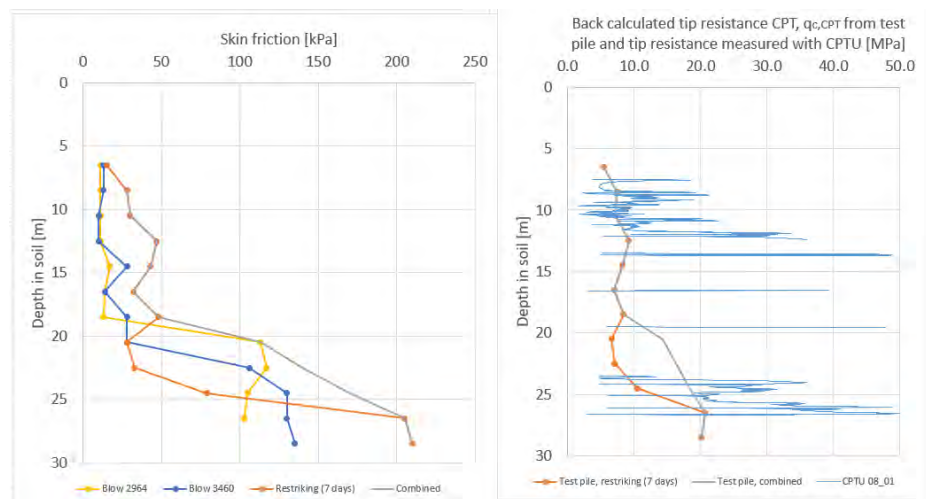


Figure 104 Distribution of skin friction estimated with CAPWAP and back calculated tip resistance $q_{c,CPT}$ compared to measured value from CPTU in nearby borehole

The back calculated $q_{c,CPT}$ can then be used to calculate the end bearing capacity based on the NGI99-method, as shown in Figure 105. In 25 m to 28 m depth, it is shown a tip resistance of 16-17 MN, which is at least 4 times larger than estimated directly with PDA/CAPWAP (Table 12). This fits surprisingly well with the mobilization curve shown in Figure 103 and the corresponding expectation of tip resistance between 3 to 5 times larger than estimated directly with PDA/CAPWAP due to limited mobilization of single blows.

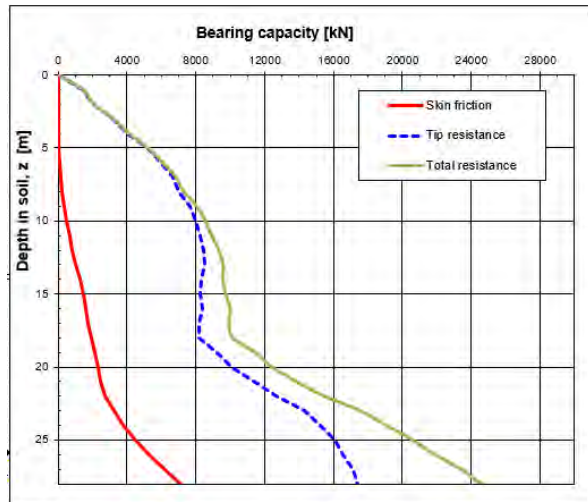


Figure 105 Calculated bearing capacity NGI99-method with back calculated $q_{c,CPT}$

With continuous PDA-measurements compression and tension stresses due to propagation of the wave from the impact of the hammer were estimated for the whole depth of pile driving. The largest compression stress was estimated at the end of pile driving with a value close to 180 MPa. The associated tension stress was approximately 40 MPa. At around 25 m depth, up to 80 MPa tension stress was estimated but the compression stress was at the same time somewhat smaller than maximum. Fatigue assessments according to [4] based on the estimated stress history on the test pile could be considered acceptable, so the steel casing could be utilized structurally after pile driving.

The test piling also proved that the drivability was significantly better than estimated with GRLWEAP before test piling. With attempts to back calculate number of blows and energy with GRLWEAP, a reasonably good fit was achieved with the use of soil resistance similar to what was calculated based on the NGI99-method and very low quake and damping values compared to what was determined with CAPWAP. The attempts at back calculating made it clear that for large diameter end bearing piles quake and damping factors determined with CAPWAP-analysis appear not relevant considering

drivability. They seem only relevant for estimating which bearing capacity it is possible to estimate directly with PDA/CAPWAP.

Axis 9

CAPWAP-analysis were performed on PDA-data from control blows after chiselling into bedrock. The total bearing capacity was estimated to approximately 27 MN.

CONCLUSIONS

- Test implementation piling significantly mitigated risk and facilitated the optimization of pile design.
- The test piles demonstrated that they could be driven deeper than initially anticipated. This allowed for a higher calculated bearing capacity, optimizing the number of driven piles in each axis. Generally, the number of piles was reduced from 12 to 10, decreasing the total number of piles by 10. This equates to a reduction of approximately 600m in pile length (including test piles), corresponding to roughly 2000-ton CO₂-equivalents.
- The skin friction estimated from CAPWAP can be used to back-calculate the cone tip resistance ($q_{c,CPT}$) with the NGI99-method. The back-calculated tip resistance aligns well with the tip resistance recorded with CPTU in the nearby borehole. In this manner, a test pile can supplement CPT/ CPTU tests for determining bearing capacity and drivability based on CPT tip resistance. This is particularly beneficial in soil layers containing gravel, cobbles, and boulders, where CPT is challenging to perform without significant predrilling. The recorded CPT tip resistance can sometimes appear unrealistically high in these soil conditions, complicating the assessment of a realistic bearing capacity and driving resistance based on CPT data. For this method, continuous PDA measurements at the depths of interest are highly recommended. Restriking (after 7-10 days) is crucial as the skin friction during driving is typically much lower than some time after, due to set-up effects.
- Interpretation of results from test piling indicate that end-bearing capacity estimated directly with PDA/CAPWAP underestimated the bearing capacity by a factor of at least 4. This is believed to be related to the limited mobilization of end bearing capacity, due to small displacement (relative to the pile diameter) per blow of the hammer. The factor will in general be smaller for smaller pile diameter, but the same method as shown in this paper may be used to estimate the actual end bearing capacity also for smaller end bearing piles.

- The test piling proved that the drivability was better than expected. For large diameter end bearing piles quake and damping factors determined with CAPWAP-analysis do not appear relevant considering drivability in GRLWEAP. Much smaller values had to be used to achieve a reasonably good fit.

ACKNOWLEDGEMENT

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THE TILLER-FLOTTEN PEAT SITE: A NATURAL PEAT SOIL DEPOSIT AIMED FOR GEOTESTING

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KEYWORDS

Peat, Geophysics, CPTU, Sampling

ABSTRACT

Better understanding of the properties of peat are necessary in the context of finding alternative construction methods that reduce the impact and degradation of bogs. In this context, a peat research site at Tiller-Flotten, Trondheim has been established. The site covers an area of 35,000 m² with a low to medium decomposed peat deposit of at least 4 m depth over marine clay. Conditions on the site are very typical of other peat deposits found in Norway. This paper presents the initial characterisation of the site including geophysical and geotechnical in-situ and laboratory testing. The aim is to establish a benchmark site for the engineering characteristics of peat that can contribute as training and teaching facilities and as ground for development to further advance the state-of-the-art in the understanding of peat soils.

INTRODUCTION

This paper builds on the work of the Norwegian Geo-Test Site program (NGTS). In the NGTS project five geo-test sites were carefully characterised using a wide range of geotechnical techniques for the purposes of benchmarking in geological and geotechnical engineering [1]. These five sites included those on soft clay, quick clay, silt, sand, and frozen soil. There has been significant recent interest in the engineering properties of peat and highly organic soils, which has encouraged adding new sites to the NGTS. This is mostly due to environmental reasons as peat is an excellent carbon sink and peatlands are home to a wide range of biodiversity. This interest is mainly motivated by the fact that the common technique of digging out and replacing the peat by granular material, due to its poor engineering properties of high compressibility and low strength for road projects for example, is no longer deemed applicable. In fact, regulatory authorities are now very

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reluctant to permit such work and it is often encouraged to find alternatives such as constructing directly on the peat. Establishing such test sites on peat, as at other test sites in the world, e.g. that at Uitdam in the Netherlands [2], sheds light on understanding the complex mechanical behaviour of peat for design and maintenance of infrastructure.

This paper presents the results of the testing carried out to develop a peat research site at Tiller-Flotten, Trondheim, Norway. Geotechnical and geophysical testing to date are summarised with a particular emphasis being placed on the difficulties encountered in investigating peat due to its natural variability. The site has already been used for several research projects including the extraction of block samples of natural peat for centrifuge testing (EU GEOLAB project CLARIFIER) [3, 4] and for investigation of the use of industrial byproducts in stabilisation of soft clay, quick clay and peat [5] (Norwegian Research Council project GOAL). The Tiller-Flotten peat site may also be seen as a potential research station for further characterization in other areas as biology, ecology, hydrology, and chemistry.

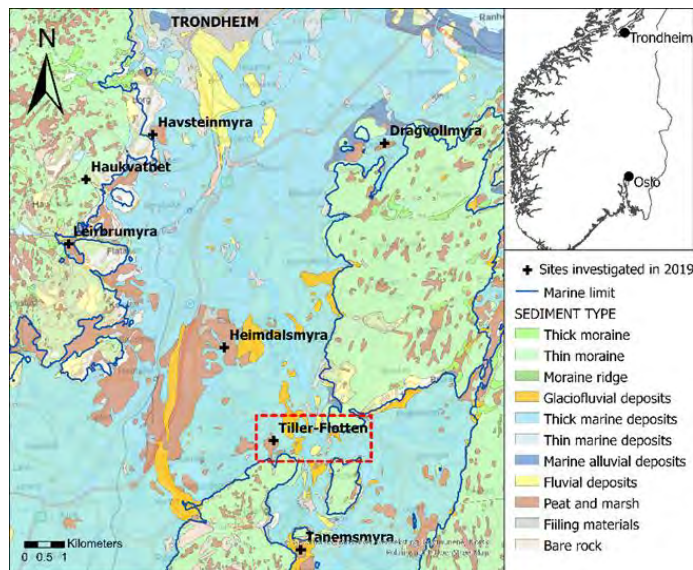


Figure 1 Location of test site, and other peat sites in the area, superimposed on Quaternary geological map. Base map from Norwegian Geological Survey (NGU) <http://geo.ngu.no/kart/losmasse/>. Figure modified from [7].

SITE LOCATION AND GEOLOGICAL BACKGROUND

The Tiller-Flotten peat site is situated approximately 14 km south of Trondheim in Norway (Figure 1). The peat research site is immediately adjacent and just south of the NGTS quick clay research site [6]. The area developed as a natural raised bog (“myr” in Norwegian) since the retreat of the glaciers some 10,000 years before present. The site is characterised by a

thrifty vegetation with species growing in very nutrient-poor bogs and bogs only receiving water from rainfall. The peat is underlain by a thick deposit of marine clays. The geological background for the neighbouring quick clay site is of deposition occurring in a saline environment of 30-40 m depth with changing conditions towards estuarine and beach conditions and that the site emerged from the sea ca. 9,500 years ago. More on the geological background of the area is described by [6].

The general stratigraphy in the area is dominated by the peat layer over sensitive clay, overlying non-sensitive clay. This is illustrated by the Electrical Resistivity Tomography (ERT) profile in Figure 2 (after [7]) where the top 1-5 m is characterised by a resistive layer ($\rho > 100 \Omega\text{m}$) (i.e. peat) and fairly constant values ranging from 30 to 40 Ωm down to about 35 m below ground level (i.e. sensitive clay) over a non-sensitive clay with resistivity less than 10 Ωm .

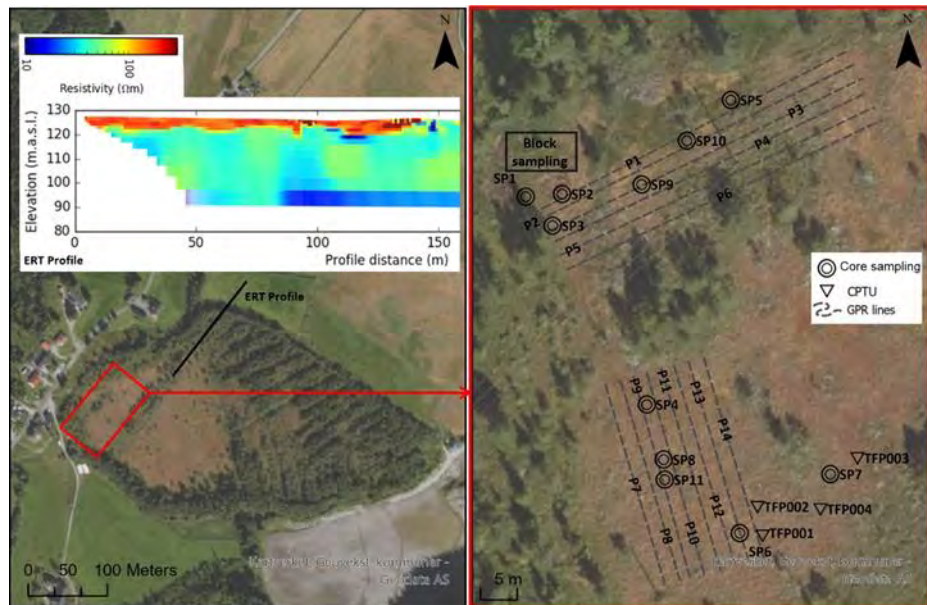


Figure 2 Site detail showing peat sampling area (quick clay area is to north), locations of block sampling, peat cores, CPTU, ERT and GPS lines. ERT profile shown in inset.

DETAILS OF INVESTIGATION ON THE SITE

The location of all tests on the site including the ground penetrating radar profiles (GPR), the block sample locations, the position of the Jowsey auger probes, the shear wave velocity (V_s) soundings and the piezocone penetration tests (CPTU) are shown on Figure 2.

Resistivity was measured using the Electrical Resistivity Tomography (ERT) technique. The final RMS (root mean square of the misfit between the data and the models) is 0.7%, which is considered good.

GPR data was obtained using a 250 MHz unit which was hauled by the operator across the study area. GPR involves the transmission of high frequency radio waves into the subsurface and recording the reflected waveforms. The radio frequency (rf) velocity through peat is taken to equal 0.035m/ns, which is very close to that of pure water (0.03m/ns) as might be expected due to the very high water contents found in many peat soils.

In situ shear wave velocity profiles were obtained using a portable downhole sonde which was developed for the purposes of taking V_s readings through a vertical peat column by [8].

Soil sampling was either by block sampling according to [9] or by using a Jowsey auger [10] which produces a 52 mm diameter “half core” of the peat over the full depth profile).

CPTU testing was by means of a lightweight 1t Pagani rig. Work was carried out during the summer of 2023 when the water table was low and the rig was able to traffic over the peat surface.

GPR RESULTS

An example of a GPR profile (Profile 1, P1, in Figure 2) is shown on Figure 3. A clear reflection of the propagating wave can be seen at the base of the peat which allows “picking” the peat base profile. The GPR derived peat depth was checked at several locations (e.g. SP2 and SP10 on Figure 3) using the Jowsey probes or shear wave velocity sonde and was proven to be very accurate.

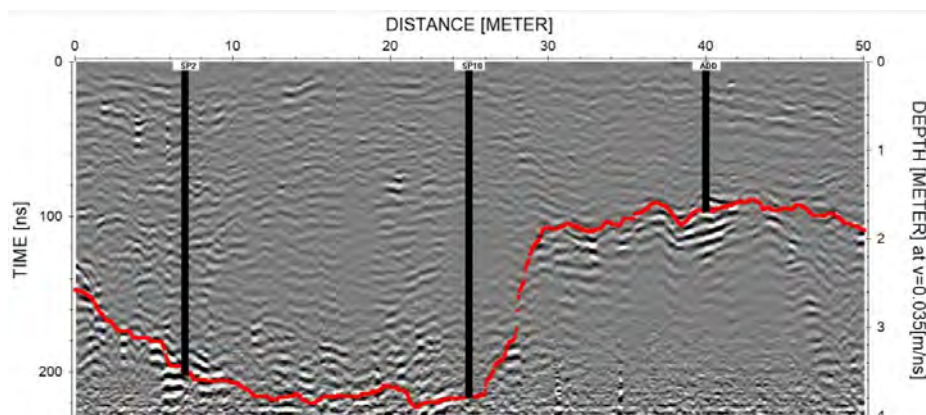


Figure 3 GPR Profile 1 showing a clear reflection of the signal at base of peat. Peat depths at SP2 and SP10 (which were proven to be 3.55 m and 3.8 m respectively) are shown as reference. An additional probe at the location indicated would be very useful.

The results of the GPR work can then be used to develop a series of contours of peat thickness as shown in Figure 4. The peat thickness is very variable across this relatively small study area and varies between some 1 m in the east of the study area to 4 m in the west. A north-south clear relatively deep channel can be observed in the western side of the study area. The underlying soils also appear to form a high ridge in the to the east.

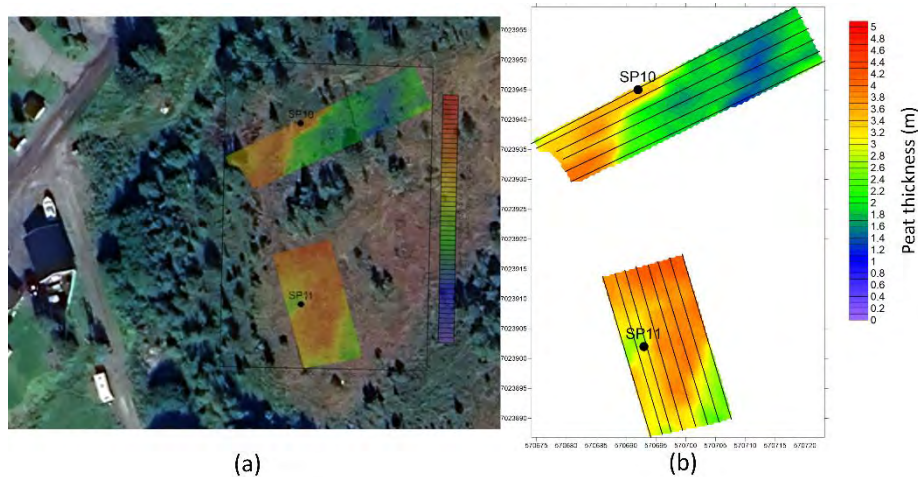


Figure 4 Peat thickness contours as derived from GPR (and verified by probing, e.g. proven peat thickness at SP10 and SP11 was 3.8 m and 2.9 m respectively)

PEAT INDEX PROPERTIES

Values of the water content (w) and the von Post [11] degree of decomposition of the peat are shown on Figure 5. The water content data, in particular, is characterised by its extremely high variability. Values vary between some 500% to over 2000% with an average of about 1150%. It is likely that the sampling processes, both the block sampling and the Jowsey auger sampling, led to some loss of water in the samples. Nevertheless, the material seem to be highly variable and thus forms a significant challenge for geotechnical design and analysis.

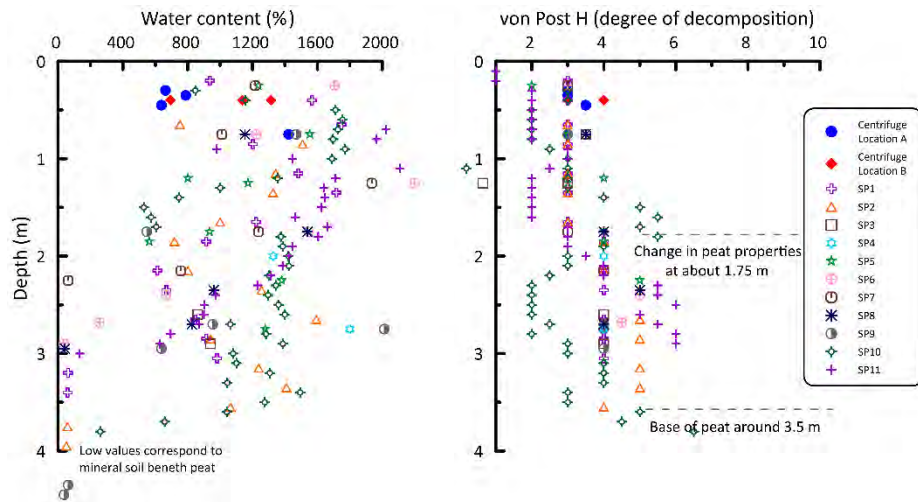


Figure 5 Peat index properties (a) water content and (b) degree of decomposition

The von Post H values (and visual observations of the peat) show a clear change in the peat properties at about 1.75 m depth. The material becomes more decomposed below this level with von Post H increasing from ca. of 3 to 4-5.

Similarly, the von Post F (fine fibre content) and R (coarse fibre content) indicate that, although the material is highly fibrous throughout the profile, there is a greater percentage of fibres over the top 1.75 m. The material has an average bulk density of about 1.01 Mg/m^3 (i.e. very close to that of water) and an average loss on ignition (LOI) of 96.5% (i.e. it is more or less completely organic).

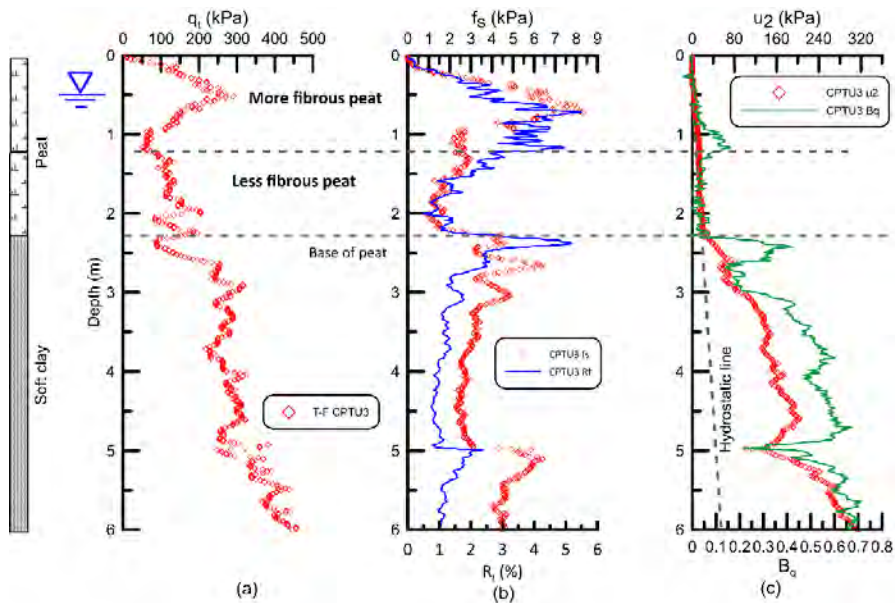


Figure 6 Typical CPTU test (CPTU3) showing all data (a) q_t , (b) f_s / R_f and (c) u_2 / B_q

CPTU TESTING

All CPTU data for one specific test (CPTU3) is shown on Figure 6. The figure gives plots of corrected CPTU cone resistance (q_t), sleeve friction (f_s), friction ratio (R_f), generated pore pressure (u_2) and pore pressure parameter [$B_q = (u_2 - u_0) / (q_t - \sigma_{v0})$; where u_0 is the in situ pore water pressure and σ_{v0} is the total overburden stress]. The test was performed to 6 m depth which means that the probe penetrated the complete peat layer and the soft clay under. The peat layer can be identified by the low cone resistance ($q_t < 200$ kPa), high friction ratio ($R_f = 100 * f_s / q_t$ up to 7%), low pore pressures (u_2) with a tendency towards negative values in the more fibrous zone (i.e. over the top 1.75 m.) and low normalised pore pressure ($B_q < 0.1$ compared to the clay layer below). These observations agree with those of others, who have shown that peat can be characterised by CPTU by low q_t , high R_f and low u_2 / B_q [12]. In addition, the q_t and R_f data clearly distinguishes between the two peat layers with the upper less decomposed layer having relatively higher q_t and higher R_f . The upper peat layer is about 1.25 m at the location of CPTU3. However, the q_t and R_f data do not clearly identify the difference between the peat and the underlying soft clay. This boundary however is defined well by the u_2 / B_q data with penetration in the peat being largely drained and showing high u_2 / B_q values in the soft clay.

On Figures 7a and 7b the focus is on the CPTU data in the peat only with data from all 4 positions shown. Generally, the q_t and f_s data are similar at all four

locations for the upper 1.25 m layer of peat. In the lower layer q_t is particularly variable, varying between values as high as 200 kPa to 20 kPa.

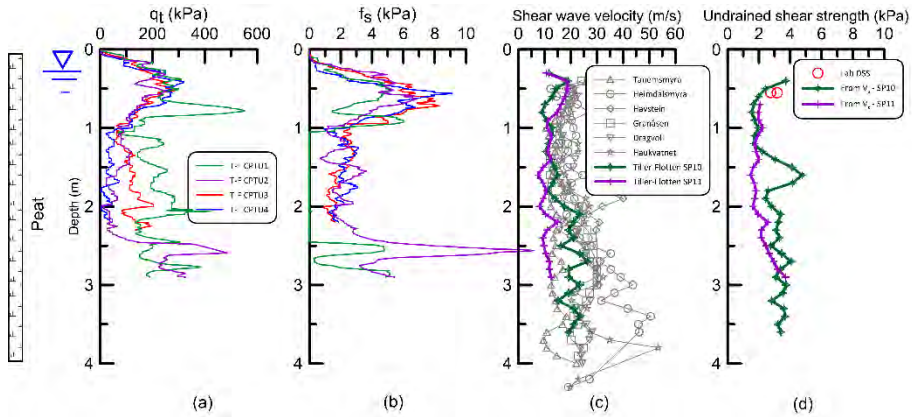


Figure 7 All CPTU tests with focus on peat and shear wave velocity profiles (a) q_t and (b) f_s , (c) shear wave velocity V_s and (d) undrained shear strength s_u .

SHEAR WAVE VELOCITY TESTING

The two shear wave velocity profiles from Tiller-Flotten are compared to those from six other Trondheim sites (see Figure 1 for site locations) on Figure 7c. Values of V_s are very low being on average 17 m/s for SP10 and 12 m/s for SP11. The Tiller-Flotten data falls to the lower V_s end of the Trondheim profiles and is most similar to data from Tanemsmyr. This is not surprising as both of these sites are effectively undisturbed natural peatlands. Many of the other sites have been altered by drainage or nearby construction works. There is no clear reduction in V_s with depth, a finding perhaps inconsistent with the CPTU2 and CPTU3 q_t profiles.

The value of undrained shear strength (s_u) derived from the V_s data is shown on Figure 7c. The formula derived by [9] was used:

$$s_u = 55.4 \left(\frac{V_s}{w} \right)^{0.68} \quad (1)$$

This formula was based on laboratory direct simple shear (DSS) tests consolidated to in situ (i.e. very low) effective stresses and V_s measurements on the same samples at equivalent stress. The profiles suggest the s_u values are very low, increasing very from about 2 kPa near the surface to 3 kPa with depth.

DIRECT SIMPLE SHEAR (DSS) TESTING

Some DSS tests were carried out on specimens derived from one of the block samples (depth 0.6 m) at the geotechnical laboratory of Deltares in Delft. The

samples were prepared by careful trimming of the block with a serrated knife and were 63 mm in diameter and 20 mm high. They were initially consolidated to vertical stresses between 5 kPa and 60 kPa (σ_{vc}') and then sheared undrained (sample height maintained constant). The undrained shear strength (s_{u-DSS}) was taken at the point where the shear strain equals 15%, which is normal practice at NGI. The results, plotted as shear stress versus shear strain and normalised shear strength s_{u-DSS}/σ_{vc}' versus σ_{vc}' , are shown on Figures 8a and 8b respectively and compared to a series of other test results on Trondheim peat.

At low stresses (i.e. less than the yield stress, which is estimated to be of the order of 10 kPa to 15 kPa) the values are very variable but as the stresses are increased beyond 10 kPa the s_{u-DSS}/σ_{vc}' values becomes constant at about 0.4. A very similar value was reported for Swedish peats [13].

The s_{u-DSS} values obtained at σ_{vc}' equal to 5 kPa (i.e. approximately equal to the in situ stress) are compared to those derived from the V_s profile on Figure 7d. It can be seen the comparison between the two sets of data is very good.

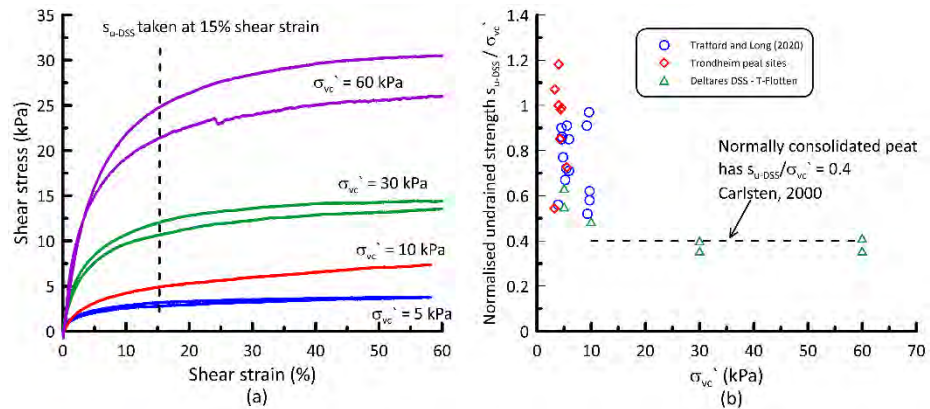


Figure 8 DSS test results (a) shear stress vs shear strain and (b) normalised undrained shear strength versus consolidation stress.

CONCLUSIONS

The Tiller-Fløtten peat research site development addresses the increasing interest in engineering related peat research due to environmental reasons. Investigations carried out at the site indicate that the successful application of geophysics in demonstrating the thickness variability of the peat layer. The CPTU results are useful in characterising peat and complemented by shear velocity measurements shed light on the strength properties of the peat. Limited “advanced” lab tests are currently available at the test site, however, the DSS results show consistency with those found elsewhere and with values derived from shear wave velocity measurements.

ACKNOWLEDGEMENT

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THREE-DIMENSIONAL EFFECTS IN TEMPORARY EXCAVATIONS IN SOFT CLAY

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KEYWORDS

Excavations, soft clay, design

ABSTRACT

Excavations must always be performed in such a way that the stability conditions are satisfying. For excavations without geometrical constraints in the vicinity a battered excavation may be sufficient. Otherwise, embedded retaining walls are required. Depending on the site conditions the retaining walls may be of cantilever type or supported at one or several levels.

In order to seek to avoid a retaining wall or - when it is needed - optimise it, it may be beneficial to account for “3D-effects”. The 3D-effects may be “natural”, e.g. an excavation with a limited horizontal extent or working equipment with a limited extent along the crest of the excavation. However, temporary 3D-effects in the construction stage may also be achieved by carefully planning the excavation works in such a way that the extent of the critical section is limited (e.g. a sequential excavation process).

This Paper summarises the results of three R&D-projects with focus on how 3D-effects can be accounted for in the design of temporary excavations in soft clay and thereby contributing to minimising the carbon footprint. The work, which is partly based on 3D finite element analyses, includes suggestions for best practice using extended 2D analyses, as well as design examples, for battered excavations and excavations supported by retaining structures.

INTRODUCTION

Temporary works, e.g. excavations, should be planned and designed in order to minimise the costs and the carbon footprint given the site-specific conditions and safety regulations.

In connection with temporary excavations these aspects may be achieved by accounting for “3D-effects”. Such effects may be the result of e.g. the geometry of the excavation, the construction sequence or the horizontal extent of heavy working equipment next to the excavation. Some examples from practice are given in Figure 1 in which a weak section (-) is surrounded by strong sections (+). Ideally, accounting for 3D-effects will result in a battered

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excavation, but when such a solution is not sufficient the amount of, e.g. the sheet pile walls and/or the loads in the bracing system can be minimised.

Historically, semi-empirical expressions and/or design charts have been developed for assessing 3D-effects in ultimate limit state design for e.g. shallow foundations (bearing capacity), slopes (stability) and retaining walls (basal heave). However, such expressions and charts are developed based on idealised situations. Accounting for 3D-effects in practice requires careful considerations. A possible approach is to perform 3D finite element analyses, but such analyses are time-consuming and are thus mainly carried out in large construction projects or in R&D-projects.

In ultimate limit state design, the 3D-effect may be quantified by a 3D factor of safety, F_{3D} , which always will be larger than the 2D factor of safety for the weak section, $F_{2D,weak}$, but always lower than the 2D factor of safety for the strong sections surrounding the weak section, $F_{2D,strong}$. The 2D safety factors for the weak and the strong sections respectively are fairly easily determined using e.g. design charts or conventional 2D software based on traditional Limit Equilibrium methods or the finite element method. However, in order to account for 3D effects more sophisticated software is needed resulting in considerably longer calculation times.

This article summarises Swedish best practice for estimating F_{3D} in temporary excavations in soft clay with an undrained shear strength in the order of 10-20 kPa and an overconsolidation ratio in the order of 1,0-1,5. The study is limited to idealising and treating soil strength as “undrained”, i.e. limited to excavations of short duration in low permeable soils. The importance of considering the effect of dissipation of (stabilising) excess pore pressures after excavation is, however, reported in [1].

Section 2 covers battered excavations while Section 3 covers excavations supported by embedded retaining structures. The suggested methods are generally based on 2D analyses of the weak and the strong sections.

3D-EFFECTS IN BATTERED EXCAVATIONS

The 3D-stability when a weak section is surrounded by infinitely strong sections have been studied by several researchers in the past. In e.g. [2] both plane ends, cf Figure 2, and curved ends have been analysed for a situation where the ground consists of a cohesive soil with constant undrained shear strength and unit weight while the only destabilising actions are the depth and the length of the excavation. If assuming plane ends $F_{3D,plane}$ can be formulated analytically. However, it is not possible to formulate an analytical expression for $F_{3D,curved}$ if assuming curved end. Based on the results reported in [2] an approximate analytical solution accounting for curved ends may be formulated as (cf. [3]):

$$F_{3D,curved} \approx F_{2D,weak} + 0,75 \cdot (F_{3D,plane} - F_{2D,weak}) \quad (\text{Eq. 1})$$

The maximum depth of the failure mechanism resulting in the lowest factor of safety accounting for 3D-effects, $F_{3D,critical}$, is always smaller than or equal to the maximum depth of the of the critical failure mechanism for the 2D factor of safety for the weak section, $F_{2D,weak,critical}$. Consequently, several 2D failure mechanisms, with varying maximum depth, must be analysed in order to find the critical 3D-mechanism using Equation 1.



Figure 1. Some typical situations where 3D-effects have been incorporated in design in order to achieve satisfying stability conditions in temporary excavations. Upper left: A battered excavation with a limited horizontal extent. Upper right: A cantilever sheet pile wall next to trafficked railway tracks. Lower left: A cantilever sheet pile wall supporting an excavation that progresses sequentially from right to left while simultaneously casting a concrete slab. Lower right: A sheet pile wall supported by struts while excavating and casting a concrete slab sequentially.

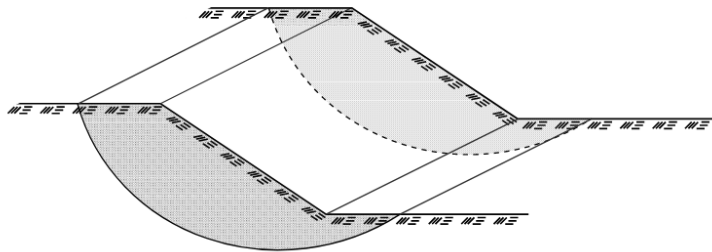


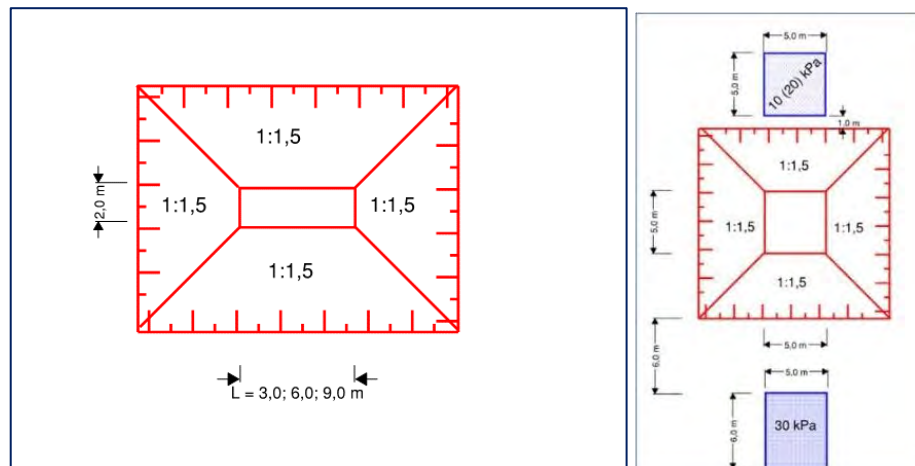
Figure 2. The 3D failure mechanism for a weak section surrounded by infinitely strong sections if assuming plane ends.

In reality, the strong sections surrounding the weak section are rarely infinitely strong. In such situations Equation 1 may overestimate F_{3D} . In order to overcome this, several scenarios, frequently occurring in practice, have been analysed. The study is briefly described below (see [3] for details).

The study focuses on four main scenarios (see Figure 3):

- A long excavation subdivided in short segments in which excavating, pipe installation and backfilling are made sequentially
- An excavation for a manhole (the length of the excavation is limited, but equal to the width of the excavation)
- An excavation for a bridge foundation (the length of the excavation is limited, but larger than the width of the excavation)
- A long excavation with heavy machinery locally located behind the crest of the excavation

Each main scenario has been further subdivided when it comes to the undrained shear strength of the soil, the geometry of the excavation and the adjacent load leading to a total of 66 scenarios being analysed. Each scenario has been studied using 3D finite element analyses (PLAXIS 3D), using the Mohr-Coulomb model, resulting in a value for F_{3D} considered to be the benchmark for that specific scenario. In a few scenarios (3 out of 66) the critical failure mechanism turns out is to be “local bearing capacity”, i.e. the critical mechanism does not involve the excavation. The 63 scenarios have also been studied using conventional limit equilibrium analyses (Slope/W) and 2D finite element analyses (PLAXIS 2D) of both the weak and the strong sections.



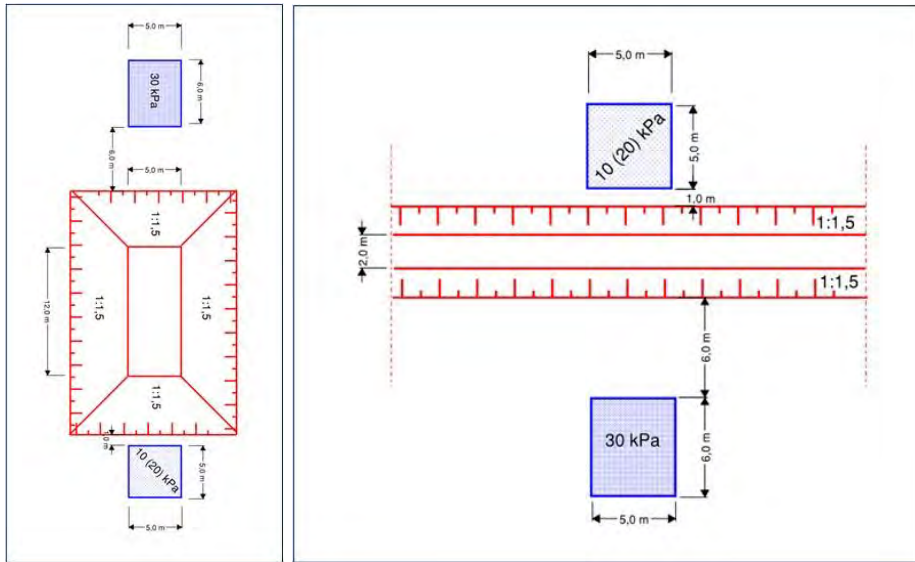


Figure 3. Upper left: A long excavation subdivided in short segments in which excavating, pipe installation and backfilling are made sequentially. Upper right: An excavation for a manhole. Lower left: An excavation for a bridge foundation. Lower right: A long excavation with heavy machinery locally located behind the crest of the excavation.

Various ways of combining $F_{2D,weak}$ and $F_{2D,strong}$ for estimating F_{3D} were formulated and compared to the benchmark (i.e. the 3D finite element analyses). The resulting suggested expression for F_{3D} is:

$$F_{3D} \approx F_{2D,weak} + \left(1 - \frac{F_{2D,weak}}{F_{2D,strong}}\right) \cdot 0,75 \cdot (F_{3D,plane\ ends} - F_{2D,weak}) \quad (\text{Eq. 2})$$

Similar to Equation 1, several 2D failure mechanisms, with varying maximum depth, must be analysed in order to find the critical 3D-mechanism using Equation 2. The resulting estimated $F_{3D,critical}$ is, with a few exceptions, within $\pm 10\%$ compared to using 3D finite element analysis, cf. Figure 4.

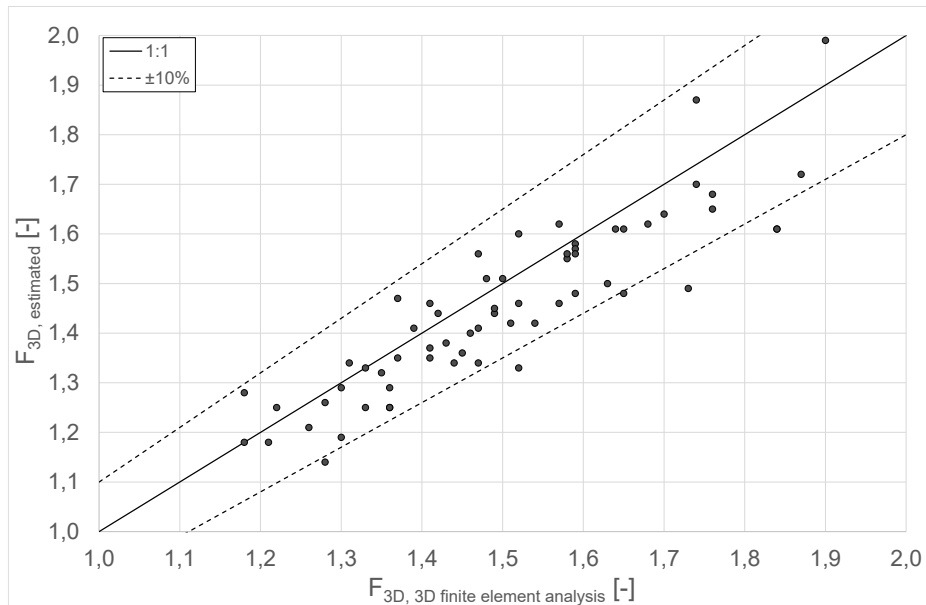


Figure 4. Comparison of F_{3D} according to 3D finite element analysis and Equation 2.

3D-EFFECTS IN EXCAVATIONS SUPPORTED BY EMBEDDED RETAINING WALLS

This section covers consideration of sequential excavation processes in the design of excavations supported by embedded retaining walls, specifically braced excavations. A complete description is given in [1], that also reports the design and monitoring results of two projects where sequential excavation was carried out. Accounting for the 3D-effect in excavations supported by a cantilever wall, which is described in more detail in [4], may be considered to be a special case of the more general cases of embedded retaining walls described in [1].

In the following paragraphs, some highlights from an idealised example involving a braced sheet pile wall covered in [1] is reported to illustrate the benefit of a sequential excavation and support process.

Figure 5 illustrates the idealised example involving a 3.5 m deep excavation in a clay characterised by an undrained shear strength c_u of 20 kPa. If the critical cross section “B” would be carried out and analysed as a 2D situation with an internal bracing system, the structural loads (considering partial factors) in the retaining system would be as in the right-hand illustration in Figure 5. Furthermore, the net earth pressure on the wall $(N_a + N_p)c_u - (\sigma_{va} - \sigma_{vp})$ considering analytical limiting (Rankine) earth pressures is shown. The net pressure is calculated based on the earth pressure coefficients on the excavated and retained side of the sheet pile wall set to $N_a = N_p = 2.57$ (i.e. according to Janbu [5] and assuming full roughness) and, in accordance with

Swedish guidelines [6], considering a water-filled crack on the retained side of the wall. The latter governs the net earth pressure above a depth of 5.5 m where the water pressure is larger than the active earth pressure.

A sequential excavation process is considered in this example to avoid an internal bracing system (rather the waler beam and the soil transfers loads between the weak cross section “B” and the strong neighboring sections “A” and “C”). The example resembles the real case “Knutpunkt Gamlestaden” reported in [1]. The simplest consideration of the 3D-effect is to consider transfer of loads between the sections in the waler beam only. Transfer of loads will, however, also take place via mobilisation of soil strength. Hence, compared to a 2D situation the active earth pressure is reduced in section “B” while it increases in sections “A” and “C”. On the contrary, the passive earth pressure is increased in section “B” while it decreases in sections “A” and “C”. The distribution of the forces in the waler beam and the earth pressure against the sheet pile wall are sketched in Figure 6.

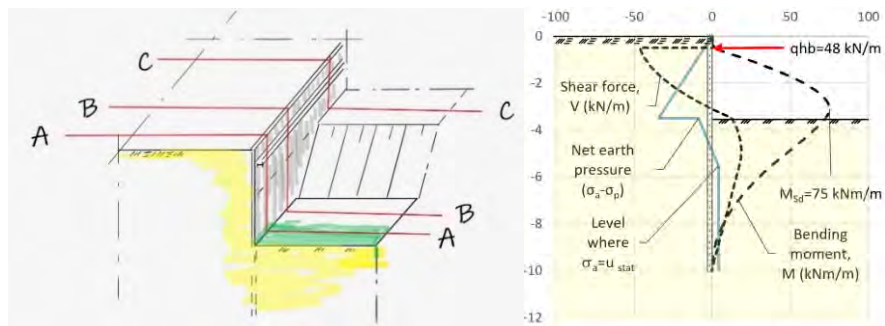


Figure 5. Left: Illustration of an idealised sequential excavation: “A” previously excavated and supported (by a concrete slab) section, “B” critical cross section, and “C” partially excavated section. Right: Structural design loads and earth pressure if analysing section “B” as a 2D situation.

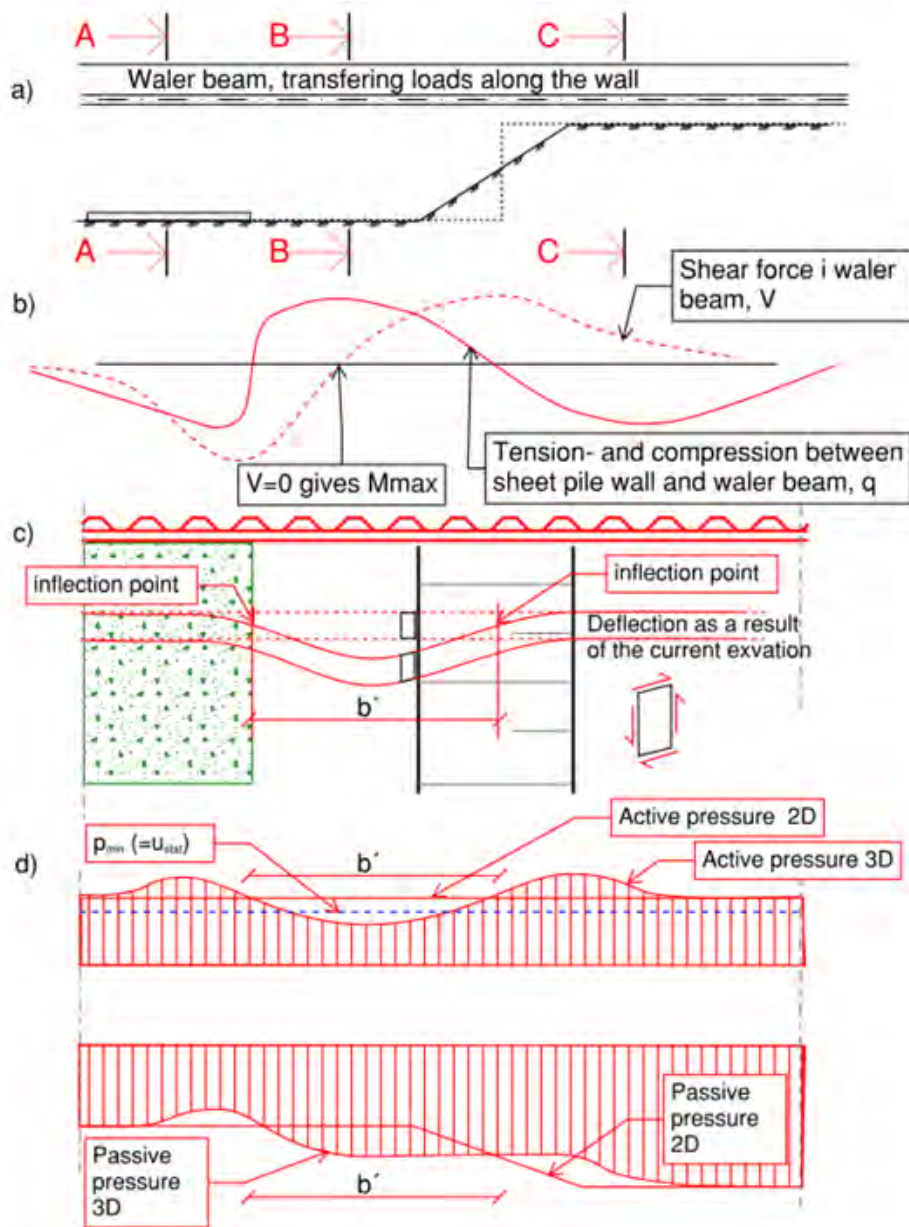


Figure 6. Sketches of the loads in waler beam and soil during sequential excavation. a) Geometrical front view corresponding to Figure 5, b) Sketch of load transfer via waler beam, c) Geometrical top view with sketch of soil deformation, d) Sketch of idealised (re)distribution of earth pressure against the retained (top) and excavated (bottom) side of the retaining wall.

Considering load transfer in the waler beam and the soil, the structural loads in the retaining system during excavation of section “B” is illustrated in Figure 7. A detailed description of the design is provided in [1]. One of the important aspects that are outlined in [1] is, e.g., that the integrated “arching effect” and load transfer in the soil in the interfaces between the weak section and the strong sections must not exceed the design soil shear strength.

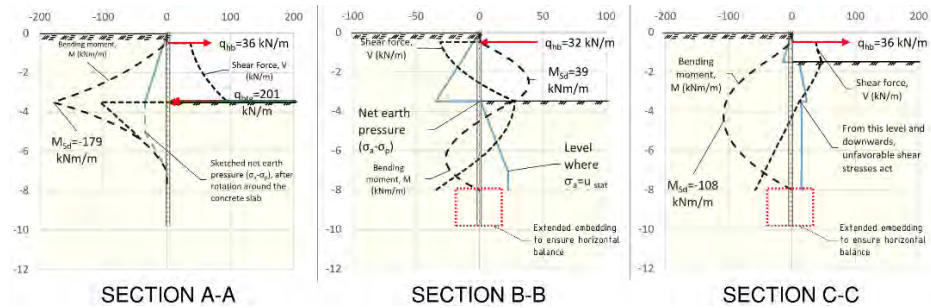


Figure 7. Structural design loads in retaining system for sections “A”, “B” and “C” considering sequential excavation (illustrating excavation in “B”).

By sound design consideration of the effect of sequential excavation and support process, internal bracing can be avoided and still conventional sheet piles (due to the limited bending moment) can be used in this example. This is summarised in Table 1. It would, conversely, be possible to keep the internal bracing system but then consider 3D-effects in design to reduce the design load and/or length of the retaining wall.

Table 1. Summary of the structural design loads in the idealised example (wall length 10m).

Situation	Max bending moment [kNm/m]	Waler beam [kN/m] section forces	Concrete working platform max [kN/m]
2D	75	48	-
2D after bracing removed	70	-	120
3D considering waler beam only ^{a)}	244	-58/+48/-58	242
3D considering waler beam and soil ^{b)}	179	-36/+32/-36	201

The excavation length of the weak section is limited to ^{a)} 3.7m and ^{b)} 5.0m respectively. In this example this is governed by the structural capacity of the chosen waler beam.

CONCLUSIONS

Sequential excavation processes can in some situations be a means to create “3D-effects” and avoid the need for retaining structures, minimising the amount of steel or concrete in retaining structures, or reduce the deformations in the surroundings. However, when 3D-effects are accounted for in design the complexity increases as there is a need to analyse several neighboring cross-sections and in particular to consider their intricate interaction. Accordingly, the design must be based on sound conceptual models and considerations. This paper, which is a summary of [1, 3 and 4], provides guidance and best practice for industry when excavating in soft clays. The design should, as always, be well-documented and thoroughly checked by an independent reviewer with adequate theoretical knowledge and experience. Furthermore, the design and restrictions must be transferred to clear-cut drawings. Also, a close collaboration is needed with the personnel at site so that the work is executed as intended. Finally, monitoring excavation-induced displacements should be done in order to ensure that the stability of the excavation is satisfying.

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TOWARD SUSTAINABLE LIQUEFACTION MITIGATION: MODELING THE CYCLIC MECHANICAL RESPONSE OF BIOCEMENTED SOILS

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KEYWORDS

Biocementation, Cyclic loading, Liquefaction, Constitutive Model

ABSTRACT

Earthquake-induced liquefaction remains a significant hazard to seismically prone regions. Although traditional mitigation methods, such as compaction, drainage and grouting are effective, they often have significant environmental and economic implications. Biogeotechnical methods, particularly Microbially Induced Calcite Precipitation (MICP), have emerged as a practical and environmentally conscious alternative. Experiments have shown that MICP induced calcite contents as low as 1% can sufficiently increase liquefaction resistance. Yet, there are few attempts to model this behavior, limiting the technique's adoption. This study aims at addressing this gap by introducing Hujieux-BC, a constitutive model for the cyclic behavior of biocemented soils. In this model, the physical mechanisms characterizing biocementation, inter-particle bonding and densification are incorporated independently. The effects of these mechanisms on the mechanical behavior are represented through changes in the material's elastic moduli and relative density as well as the introduction of a bonding parameter. The model performance is evaluated using a cyclic direct simple shear test from the literature that covers a range of cementation levels. The model is able to replicate the soil's behavior at both treated and untreated states using only the calibrated parameters at the untreated state and the biocementation parameters obtained through the measured calcite content and the change in shear wave velocity. Notably, the model is able to estimate the number of loading cycles required to trigger liquefaction reasonably well. This accuracy, combined with the model's straightforward calibration process can pave the way for more tailored liquefaction mitigation techniques using MICP.

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INTRODUCTION

In response to the limitation and environmental concern associated with traditional liquefaction mitigation techniques such as compaction, drainage, and grouting, there has been a growing interest in exploring more sustainable, cost-effective, and less intrusive alternatives. Among these, biogeotechnical approaches, particularly Microbially Induced Calcite Precipitation (MICP), have gained attention as promising solutions to enhance soil stability and resistance to liquefaction.

Despite the demonstrated potential of MICP in substantially improving the resistance of soils to liquefaction, as supported by experimental evidence [1], [2], [3], [4], wider application of these techniques is limited by the current lack of models that can accurately describe the cyclic behavior of biocemented soils.

To bridge this gap, this study presents the Hujeux-BC model, a novel cyclic elastoplastic constitutive model for biocemented soils. This model extends the principles of the Hujeux cyclic model [5] by incorporating the influence of biocementation. These adjustments are based on empirical data, including measurements of calcite content and change in shear wave velocity, allowing the model to capture the nuanced effects of biocementation under both monotonic and cyclic loading conditions.

One of the distinguishing features of the Hujeux-BC model is its ability to separately address the mechanical consequences of biocementation, such as densification and inter-particle bonding. This separation is crucial for accurately simulating the degradation of bonds over time, while also considering the lasting effects of densification. Such an approach helps improve our understanding of how biocemented soils respond to repeated cyclic loading over extended periods.

The following sections present the rationale behind the model formulation and the model performance in simulating an undrained cyclic direct simple shear (DSS) test on biocemented soils at varying cementation levels under different cyclic stress ratios (CSR).

THE EFFECTS OF BIOCEMENTATION

The model captures the influence of biocementation by examining the underlying physical mechanisms that shape its mechanical response. Key aspects of the mechanical behavior of biocemented soils under monotonic loads, as highlighted by several review articles [6] [7], include enhanced stiffness, increased strength with subsequent softening, and dilatancy. These effects are widely recognized to stem from two principal processes: densification and interparticle bonding [8], [9]. Densification is attributed to calcite accumulation in the pore spaces, a phenomenon readily quantifiable

through techniques like acid washing or thermo-gravimetric analysis [10]. In contrast, assessing interparticle bonding is more complex. Although detection is feasible with sophisticated methods such as scanning electron microscopy or X-ray micro-tomography [11], it remains difficult to accurately identify the “active” bonds at a large scale. Alternatively, these bonds are estimated using indirect methods such as the change in shear wave velocity (V_s) [8].

Further, the cyclic behavior of biocemented soils has been explored through various laboratory and scaled model experiments, revealing that even low cementation levels markedly enhance liquefaction resistance [1], [2], [3], [4]. These experiments demonstrate that biocementation's cyclic behavior is similar to its monotonic behavior, suggesting identical underlying physical mechanisms. Specifically, biocementation increases resistance, reduces porewater pressure buildup, increases stiffness, and postpones the onset of liquefaction.

The model translates the densification and bonding effects of biocementation into mathematical terms by incorporating specific parameters that reflect changes in the soil's physical properties. This approach not only preserves the original modeling framework's essence but also ensures coherence and originality in the presentation of the biocementation process and its implications on soil behavior.

Densification

Densification is mainly governed by the change in the void ratio which in turn induces changes in the soil's density and influences the state of the soil.

A densification parameter for the elastic behavior is introduced as:

$$\xi_{den} = \frac{\rho_{cc}(G_s + eSr) + m_{cc}(G_s - Sr)}{\rho_{cc}(G_s + eSr)} \quad (1)$$

Where G_s is the specific gravity of the soil, Sr is the degree of saturation, e is the untreated void ratio, $m_{cc} = \frac{m_c}{m_s}$ is the calcite mass content, and $\rho_{cc} = \frac{\rho_c}{\rho_s}$ is the calcite density ratio.

The effect of densification on the yield surface is introduced through the parameter ψ_{den} expressed as:

$$\psi_{den} = \exp\left(\frac{e - e^*}{\lambda}\right) \quad (2)$$

here, λ is the coefficient of compressibility obtained as the slope of the critical state line (CSL) in the $v - \ln(p')$ plane and e^* is the void ratio after biocementation

Bonding

The effect of active calcite bonds on the elastic behavior is obtained through the change in shear wave velocity and is estimated as:

$$\xi_{bon} = \frac{(V_s^2)^*}{V_s^2} = \frac{(V_s + \Delta V_s)^2}{V_s^2} \quad (3)$$

where V_s and V_s^* are the shear wave velocity before and after calcite precipitation.

The effect of bonding on the yield surface is introduced through the parameter ψ_{bon} expressed as:

$$\psi_{bon} = (\psi_{bon_0})^R \quad (4)$$

where ψ_{bon_0} is the initial degree of bonding and R is a debonding function given by:

$$R = \exp(-\omega \cdot \varepsilon_v^p) \quad (5)$$

here, ω is the debonding rate and ε_v^p is the plastic volumetric strain. This expression was inspired by the work of Koliji et al. (2010) on structured soils since it was observed that their evolution during loading closely resembles that of biocemented soils [12].

MODEL FORMULATION

The model presented in this study is based on the work of Hujeux (1985) where the detailed equations are described. The following presents the changes due to biocementation.

The change in the elastic behavior due to biocementation is expressed through the elastic cementation factor CF_E which is a combination of the densification and bonding elastic parameters described in the previous section as:

$$CF_E = \xi_{den} \cdot \xi_{bon} \quad (6)$$

The elastic moduli are then expressed as:

$$\begin{aligned} K(p') &= CF \cdot K_{ref} \left(\frac{p'}{p_{ref}} \right)^n & G(p') \\ &= CF \cdot G_{ref} \left(\frac{p'}{p_{ref}} \right)^n \end{aligned} \quad (7)$$

where K_{ref} and G_{ref} are the reference bulk and shear moduli, p_{ref} is the mean effective pressure at which the moduli were estimated, p' is the current mean effective pressure, and n is the elastic exponent.

The change in the yield surface to account for the increased strength of biocemented soils is expressed using the mean pressure at the critical state p'_{cs} which evolves to:

$$p'_{cs}^* = CF_S \cdot p'_{cs} \quad (8)$$

where $CF_S = \psi_{den} \cdot \psi_{bon}$ is the cementation factor for strength.

MODEL PERFORMANCE

The model's performance was tested against a series of undrained cyclic DSS tests conducted by Lee et al. (2022) on Ottawa sand F65 with light cementation levels [4]. The tests were conducted under a confining pressure of 100 kPa at 5 different cyclic stress ratios ($CSR = \tau_{max}/\sigma_{v_0}$), 0.1, 0.15, 0.2, 0.25, 0.3. All of the samples in the test were very lightly cemented with calcite contents below 1 %.

The model parameters were calibrated on the untreated results with the biocementation parameters estimated using the reported calcite content and change in shear wave velocity [4]. The results shown in Figure 106 show the experimentally obtained and model predicted evolution of the excess pore water at a CSR of 0.2. There is a clear alignment between the experimental and model results demonstrating the model's ability to capture the trends during the test as well as the number of cycles needed to reach an r_u value of 0.95. These results highlight the model's potential in predicting liquefaction under cyclic loading.

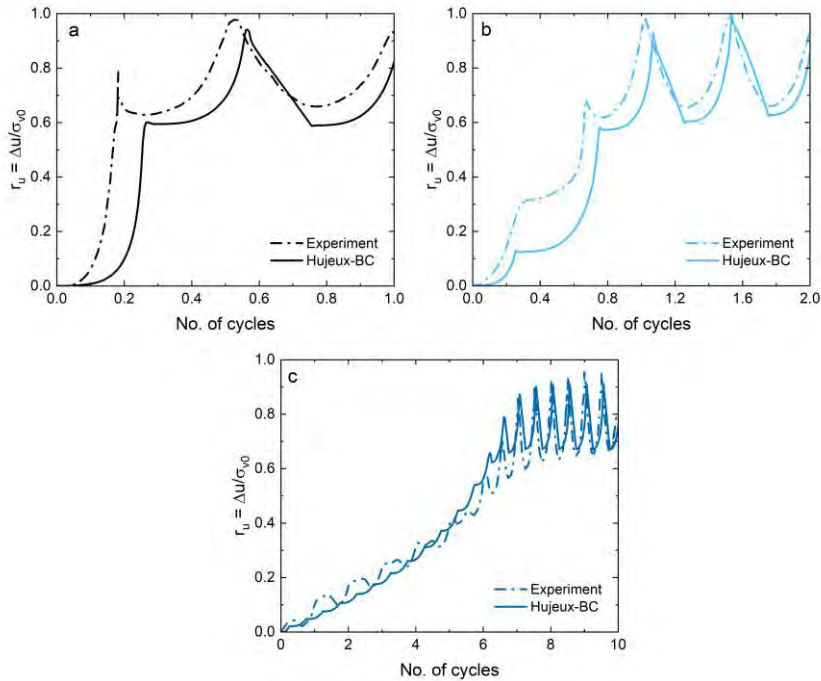


Figure 106. Comparison between experimental and simulations results for an undrained cyclic DSS on MICP-treated sands using Hujieux-BC a) untreated, b) $m_{cc} = 0.14\%$ – $\Delta V_s = 16$ m/s, c) $m_{cc} = 0.43\%$ – $\Delta V_s = 36$ m/s (experimental data from Lee et al (2022) [4])

Using the same model parameters calibrated on the tests shown in Figure 106, the model was used to estimate the number of cycles required to reach an excess porewater pressure of 0.95 for all of the CSR values. The CSR – N curves shown in Figure 107 show the experimental and model results for the untreated state as well as at two cementation levels. The model predicted curves closely match the experimental ones for high CSR values but start to deviate at lower CSR values. The mean absolute percentage error was calculated at 36% for all of three curves. This slightly large error is mainly due to the large differences predicted at the CSR of 0.1, focusing on the high CSR values, where liquefaction is more likely to occur, the calculated error for the three curves is around 14%. Given the inherent variability of cemented soils, the results offer a reasonable estimation showcasing the model's potential for evaluating MICP based mitigation techniques.

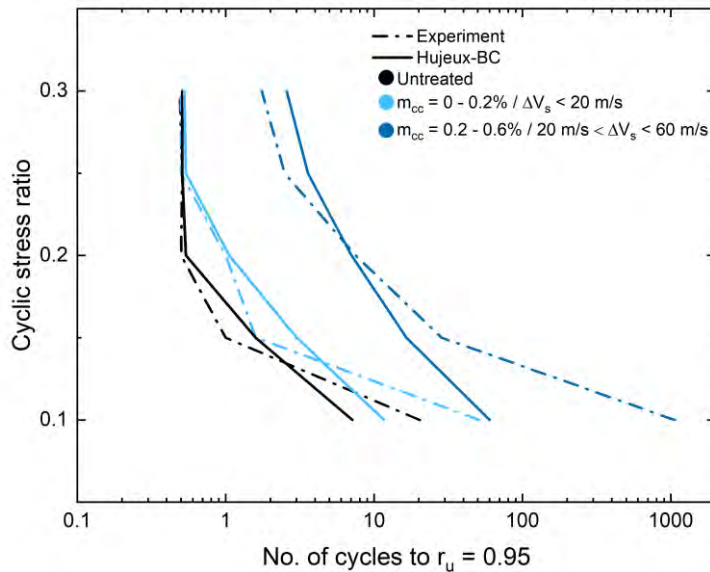


Figure 107. Comparison between simulated and experimental CSR – N curves (experimental data from Lee et al (2022) [4])

CONCLUSIONS

This study introduces the Hujeux-BC model, specifically designed for the analysis of biocemented soils, by incorporating the impacts of biocementation through the calcite content and change in shear wave velocity. The model uniquely represents densification and inter-particle bonding independently, giving it the potential to simulate the long-term soil behavior as bonds degrade. The model accurately reflected the behavior of biocemented soils under cyclic loading, successfully generating accurate CSR – N curves for soils with varying cementation levels.

Despite the inherent challenges in modeling biocemented soils, this study successfully demonstrates that beginning with the untreated, natural state of sands and including factors for densification and bonding impacts allows for an accurate representation of their cyclic behavior. Additionally, as experimental data becomes more available, model calibration can be improved leading to a more robust framework.

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UNSATURATED BEHAVIOR OF STABILIZED CLAY: ASSESSMENT OF LOW-CARBON BINDERS IN A FIELD STUDY

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KEYWORDS

Dry deep mixing, low carbon binders, SWCC, soft clay, partial saturation

ABSTRACT

The stabilization of soil with cementitious materials often induces desaturation. This study investigates the stabilization of soft clay after a one-year curing period under real field conditions. Three binders—Terra Poz, Terra Green, and Terra GTC—are tested to assess their potential as low-carbon substitutes for ordinary Portland cement (CEM I) in soft clay stabilization at Malminkenttä, an old airfield in Northeast Helsinki. Stabilization involves dry deep mixing of the low-carbon binders with saturated soft clay in separate columns.

Measurements of total suction are conducted using a chilled mirror device, establishing the soil-water characteristic curves (SWCC) under both drying and wetting paths. The impact of stabilization with different binders on air entry value, saturation behavior, and residual water content of the samples is analyzed.

Observations indicate that hydration and reactions between the clay and binders lead to a reduction in water content of the stabilized soil, resulting in persistent unsaturated behavior even after one year in a saturated soil environment.

INTRODUCTION

The dry deep mixing method (DDM) has been extensively used in Finland in the last four decades to improve the properties of very soft clays, which are found across the country. In this method, lime and cement have been used as the binder materials which can easily react with the high-water content of Finnish clays. However, due to the large carbon footprint that these materials have, continuing to use them would hinder any efforts towards a carbon neutral society. The city of Helsinki for instance, has decided to stop completely the usage of only lime and cement in column stabilization and it is now carrying out an extensive field and laboratory testing program along with

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environmental monitoring of field-mixed columns stabilized with low-carbon binders. This testing is being performed in *Malminkenttä*, an old airfield in Northeast Helsinki with clay deposits that consist of sensitive, high plasticity 5 to 20 m deep clay layers. The water content of some of the layer is up to 160%. The area has been designated as the location for a future residential plan development that requires an extensive soil improvement. If deep mixing is to be used, the total length of the required stabilized columns in Malminkenttä to improve the whole area is estimated to be 100 million meters. This would represent an enormous amount of carbon emissions if the stabilization was done with the traditional lime/cement binder.

Many of the binders used in Malminkenttä have shown to be an adequate performance for soft soil stabilization [1]. In this study, the soil-water characteristic curves (SWCC) of stabilized field-mixed with three different low-carbon binders used in Malminkenttä is obtained and compared against one another. The samples were extracted from field columns recovered after 1 year of curing in Malminkenttä and belong to the most upper part of the column, between 1 m to 2 m depth. Specimens from the columns showed a persistent unsaturated behavior even after one year within the saturated soil environment. This behavior has implications in the way columns are analyzed and simulated as explained in Bozkurt et al. [2], where numerical simulations of undrained extension triaxial tests resulted more accurate when the stabilized soil was simulated as partly saturated. Thus, a preliminary study of the partly saturated conditions of these columns is performed as a starting point to understand the effects of partial saturation on strength and conductivity of field columns.

MATERIALS AND METHODS

Clay deposits

The specimens tested in this study belong to columns that were initially constructed with Malminkenttä soft clay, a fully saturated and normally consolidated postglacial clay with high plasticity and low shear strength. The water content at the depth the samples in this study belong to (1-2m) is between 90 to 120%, with a shear strength between 7 to 15 kPa and a sensitivity value of around 30. The groundwater level has been measured from five groundwater pipes at the location of the original columns. According to measurements of the pipes, the pressure level is about 0-1 m above the ground (+15 - +16 MASL). Thus, the columns are surrounded by fully saturated soil.

Field-mixed specimens

In Malminkenttä, deep mixing with dry binder was used to build columns in different test areas. In this study, the stabilized material was extracted from stabilized columns that were recovered with the help of a steel pipe to a depth

of approx. 3 m and lifted with an excavator's bucket. Then, steel tubes of 51 mm were used to extract cylindrical specimens by pressing the tubes against the central part of the lifted columns along the longitudinal axis of the columns in the section that belongs to 1 to 2 meters depth. Before starting the measurements, the specimens were extruded from the tubes and their initial water content was measured to determine their initial degree of saturation. Then, the specimens were placed in a triaxial cell and subjected to high back pressure of around 400 kPa for several days with periodic measurement of B-values. Once a B-value higher than 0.95 was measured, the specimens were deemed as saturated and removed from the triaxial cell to start the suction measurements.

The specimens used for the determination of the SWCCs consist of three different binders: (1) a gypsum-based binder (Nordkalk Terra GTC) and two lime kiln dust (LKD)-based binder (Nordkalk Terra Green and Nordkalk TerraPoz). The emissions from the production of these binders are significantly lower than emissions from purely lime/cement-based binders. The components of these binders and their ratios are presented in Table 1, along with the carbon emission from their production.

Table 1. Binders used in the study.

Binder	Components	Percentage (%)	Emissions (kgCO ₂ e/t)
Terra GTC	gypsum	33	186
	slaked lime	33	
	CEMIII/A	33	
Terra POZ	CaO	33	563
	LKD	33	
	CEMIII/A	33	
Terra Green	LKD	50	237
	CEMIII/A	50	

Suction measurements and analysis

For the suction measurements in this study dewpoint potentiometer (WP4) was used. This method is an indirect measurement which determines water potential by balancing the liquid-phase water of the sample with the vapor-phase water in the enclosed space above the sample. The correlation between the total suction of the sample and the vapor pressure of the air in the sealed block chamber is described by Kelvin's function in Equation 1 [3].

$$\Psi = \frac{RT}{M} x \ln \frac{P}{P_0} \quad (1)$$

where R is the universal gas constant, M is molecular mass of water, T is the temperature of the sample, P is the vapor pressure of air and P₀ is the saturation vapor pressure at the sample temperature.

The saturated samples were placed in small metal cups to measure their mass and suction at 100% degree of saturation. The samples were then left to air-dry, with regular suction measurements taken during the drying process. This cycle was repeated until the suction measurements stabilized, indicating the samples were fully dried. A desiccator with drying silica gel packets was used to ensure the samples reached the driest condition possible. After drying, the stabilized samples were placed in humidity-controlled rooms with varying relative humidity levels of 45, 65 and 100% for at least 24 hours each, to reach equilibrium. Full saturation, with suctions close to zero, was achieved by spraying water on the samples. The samples were then dried in an oven, and the gravimetric water content and degree of saturation at different stages of the drying and wetting paths were calculated. The experimental data was fitted by using the model proposed by Van Genuchten [4] to evaluate the Soil-Water Characteristic Curves (SWCCs). The model equation is given as Equation 2:

$$S_r = \left[\frac{1}{1 + (\alpha h)^n} \right]^m, \quad (m = 1 - 1/n) \quad (2)$$

where S_r represents the degree of saturation, h denotes suction and is a positive value, and α , n and m are three parameters that are determined through fitting the model to experimental data.

RESULTS AND DISCUSSIONS

The Soil-Water Characteristic Curve (SWCC) defines the relationship between suction and either water content or degree of saturation in soil as it transitions between the saturated and unsaturated phases. The SWCC is widely used to predict the permeability, shear strength, and deformation of unsaturated soils [5,6,7]. In this work the total suction measurements of stabilized soft clay samples after one year of field curing are plotted against their degree of saturation. The laboratory data and the fitted curves are depicted in Figure 1. The shape and characteristics of SWCCs are distinctly influenced by the type of the binder used in stabilization. Notably, the degree of saturation corresponding to suction in the wetting path is lower than that in the drying path, though this effect is minimal at the suction corresponding to full saturation (S_{rs}) in the case of the Terra Green binder. This hysteresis behavior, observed in both drying and wetting paths, has been documented in previous studies for the cement-stabilized clay samples in Liu et al. [7] and for fly ash-stabilized silt and clay samples in Wen et al. [8] study on.

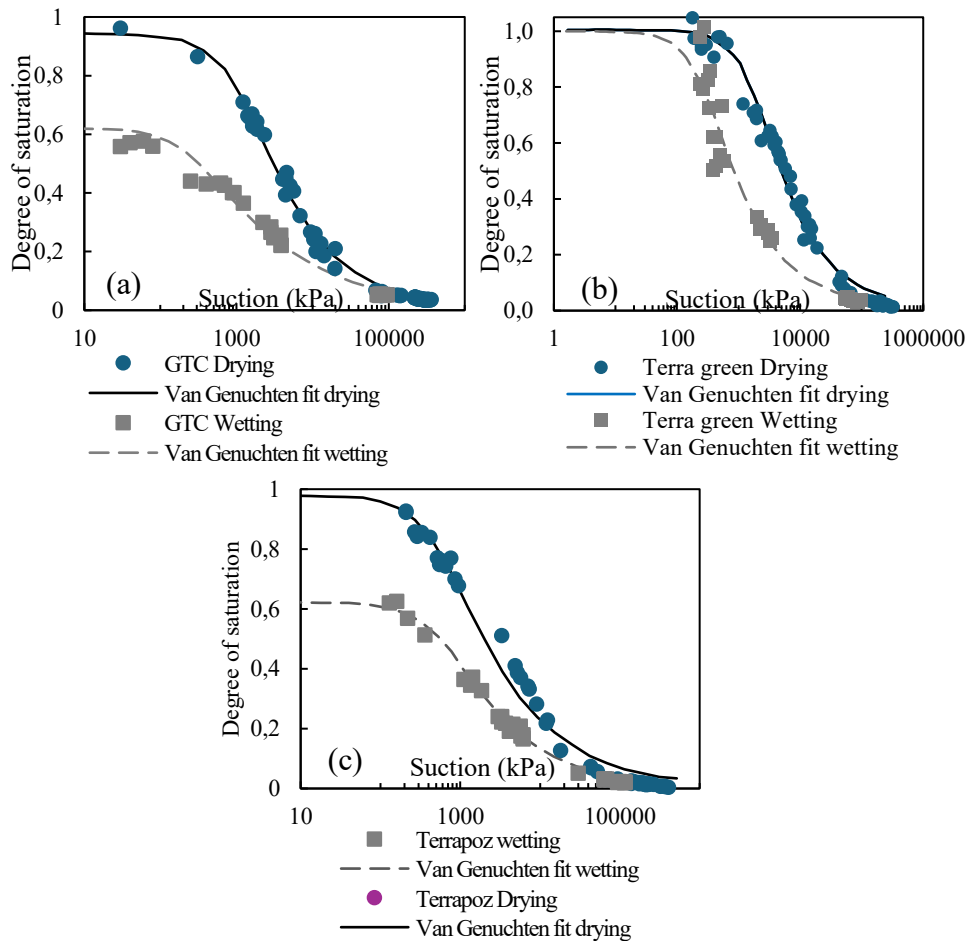


Figure 1 Drying and wetting SWCCs of stabilized soft clay after one year (a) with Terra GTC binder, (b) with Terra Green binder, and (c) with Terra Poz binder.

In samples containing Terra GTC and Terra Poz binders the hysteresis behavior is more pronounced in capillary saturation and transition zones. However, in samples containing Terra Green, the lag is visible only in the transition zone. The difference between the drying and wetting paths is not as apparent in the residual zone, regardless the type of samples. The observed behavior can be related to factors such as the ink-bottle effect, where the heterogeneous pore structure, radius of pore channels, and whether the pores are draining or wetting, affect the water retention capacity of the soil. Additionally, the angle of contact between the soil grains and/or cemented products, possible trapped air, and the shrinkage or swelling of soil during drying and wetting phases contribute to the shape of hysteresis [7, 9]. Chen et

al. [10] explains that as the initial void ratio decreases, the ink-bottle effect weakens, leading to a reduction in soil hysteresis potential and a contraction of the SWCC hysteresis loop. Similarly, when the pore size decreases under a fixed void ratio, the ink-bottle effect fades, resulting in a shorter distance between the drying and wetting curves as the suction increases.

In Table 2, the properties of the soil water characteristic curves based on the Van Genuchten equation are presented. From the values of the coefficient of determination (R^2) it can be presumed that this model can be fitted relatively well to the experimental data of stabilized samples in both drying and wetting phases. The air entry value (AEV), indicating the point where the soil desaturation begins, is highest in sample containing Terra Green during the drying phase, followed by Terra GTC and Terra Poz.

On the other hand, the degree of saturation in the residual suction area, residual saturation (S_{r_i}), displays values approaching zero regardless of the type of the binder. It's noteworthy to mention that achieving the dry conditions is highly unlikely not only due to the climate of the region, but due to the fact that the stabilized columns are situated below the groundwater level.

The process of stabilization aims to establish interparticle bonding, which fills up the pores through hydration and pozzolanic reaction products, consequently reducing pore size and increasing the AEV parameter. The elevated AEV values observed in Terra Green and Terra GTC samples suggest better stabilization with these binders, consistent with the strength analysis findings reported in the research conducted by Ramírez and Korkiala-Tanttu [1].

The initial in-situ water contents of the samples after one year of curing in Malminkenttä were 77.3%, 89%, and 68.7% for Terra Green, Terra GTC, and Terra Poz bearing samples, corresponding to a degree of saturation of 91.7%, 90%, and 90.5%, respectively. With the application of back pressure in a triaxial cell (400 kPa, for several days) to achieve full saturation and by examining the SWCC curves, it is evident that the conditions of all the samples fall within the transition zone, beyond the AEV point. This likely indicates that the hydration process of the added binders, which led to the desaturation of the soil, is not yet fully completed in the tested area, and the hydraulic properties, permeability, settlement, and strength characteristics of the soil are still developing toward their saturated final state.

Table 2. The parameters of SWCC fit by Van Genuchten [4] equation.

	Drying path					
	S_{rs}	S_{rr}	α	n	AEV (kPa)	R^2
Terra GTC	0.94	1.0e-10	0.0008	1.59	500	0.996
Terra Green	1.12	3.7e-21	0.0004	1.69	650	0.984
Terra Poz	0.92	1.0e-18	0.0016	1.52	230	0.987
	Wetting path					
	S_{rs}	S_{rr}	α	n	AEV (kPa)	R^2
Terra GTC	0.56	6.2e-16	0.0020	1.44	117	0.988
Terra Green	1.01	0.003	0.0034	1.59	105	0.985
Terra Poz	0.62	1.0e-16	0.0018	1.56	290	0.988

CONCLUSIONS

The behavior of binder-stabilized clay was analyzed through the study of the soil-water characteristic curve (SWCC) of the extruded samples from the field. SWCCs were established for stabilized soil under both drying and wetting paths, enabling the assessment of the impact of the different binders used on unsaturated properties of the samples.

The findings facilitate establishing a correlation between the strength properties and hydraulic conductivity of the stabilized soil under unsaturated conditions post-stabilization process. Furthermore, a prediction could be made regarding the time required to achieve the minimum strength properties at full saturation using the soil water retention capacity and unsaturated hydraulic conductivity.

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USE OF LECA-LIGHTWEIGHT AGGREGATE IN RAILWAY STRUCTURES

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KEYWORDS

lightweight aggregate, railway embankment design, cyclic loading, numerical modelling, ground improvement

ABSTRACT

Railway embankment design is crucial for passenger and operator satisfaction, focusing on comfort, safety, and timely operation. This paper examines Leca LWA's suitability in railway structures, determining optimal cover depth and location, and assessing its performance under high-cycle loads. The evaluation may lead to a potential use of Leca LWA in railway embankments in the future.

Simulation results show that placing Leca LWA below a 300-mm extra subballast layer improves bearing capacity and stability. The study also finds that Leca LWA can withstand cyclic loading regardless of cover depth. Recommendations include an 1150-mm cover depth, comprising a 550-mm ballast layer, a 300-mm subballast layer, and a 300-mm extra subballast layer, above Leca LWA, based on a maximum embankment height of 2.5 m and a maximum axle load of 25 tons, as studied. Further investigation is needed to assess the impact of increased height or load. The findings, after further validations, may aid in creating practical guidelines for using Leca LWA in railway embankments.

INTRODUCTION

From a geotechnical perspective, railway embankment design in cold regions like the Nordic and Baltic countries must prioritize bearing capacity, stability, settlement, and frost heave prevention for safety and efficiency.

In Nordic countries, railway tracks often cross soft clay and peat areas. Such soils may have insufficient strength, as well as may settle too much to serve as a basis for a railway embankment. Ground improvement techniques like, for example, the replacement of weak soil with lightweight aggregates (LWA)

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can enhance stability, particularly when combined with methods like preloading and column stabilization. While models exist for static loads, limited attention has been given to embankments under cyclic loading typical of railways, which can impact settlement and passenger comfort.

This study aims to assess Leca® lightweight aggregates (Leca LWA) in railway embankments, focusing on optimal cover depth and performance under high-cycle train loads. Through analysis and simulation, the study will enhance the understanding of LWA behaviour in railway conditions, validated by the Plaxis Soft Soil Creep (SSC) model. The scope will include static and moving traffic loads, with consideration of a maximum railway speed of 160 km/h.

RAILWAY EMBANKMENT

Railway embankments support tracks and endure train loads, typically consisting of ballast, subballast, and subgrade layers. A frost protection layer may also be required in Nordic countries, as frost action can affect performance, especially if the subgrade is frost susceptible. To prevent this, a non-frost susceptible frost protection layer is necessary. (Li et al., 2015; Nurmikolu & Silvas, 2013)

Leca lightweight aggregate

Leca LWA is a ceramic material produced at high temperatures, expanding clay to five times its original volume. Its hard exterior and porous interior make it versatile for engineering applications, offering low bulk density, water permeability, insulation, strength, durability, and recyclability.

With properties akin to medium-dense sand but significantly lighter, Leca LWA effectively reduces earth pressure while maintaining strength. In railway embankment structures, it enhances stability, reduces settlement, improves drainage, and prevents frost damage in the subgrade.



Figure 1 Leca® LWA physical properties (Leca website).

Studied railway embankment profile

The studied profile, as shown in Figure 2, is taken from RATO 3, referring to it as a technical regulation for railway tracks. The dimensions of cross sections are presented in Table 1.

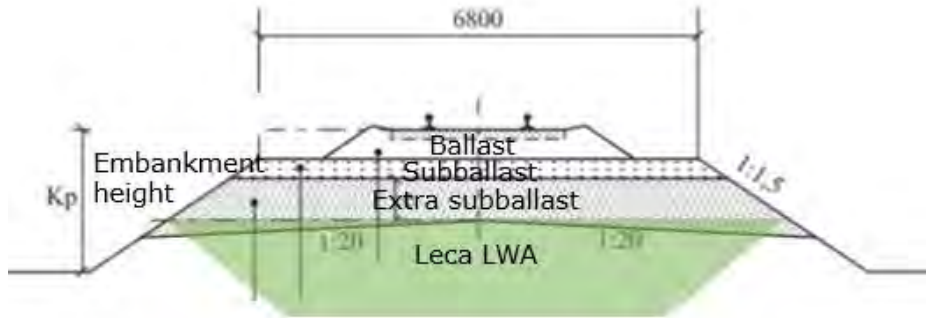


Figure 2 Typical railway cross section from Finland (Finnish Transport Agency).

Table 1. Studied dimensions of the cross section.

Components	Dimensions
Width of sleepers (single track)	2.6 m
Embankment height	1.5 m / 2.5 m
Ballast layer	0.55 m
Subballast layer	0.3 m
Extra subballast layer	0 m – 1 m (varied)
Leca LWA layer	1 m / 1.5 m / 2 m / 3 m

METHODOLOGY

Plaxis is commonly used for advanced geotechnical engineering modeling. The SSC-model in Plaxis computes creep deformation over time, suitable for simulating cyclic loading by considering increasing plastic strain. Leca LWA conducted cyclic compression tests, correlating deformation over cycles. The high-cycle accumulation model correlates PLAXIS deformation over time with Leca's cycle-based deformation. This approach is applied to validate the HCA model with Leca laboratory data.

The lab results of cyclic compression tests on 2 LWA samples were replicated by the High cycle accumulation (HCA) model, as shown in Figure 3. The HCA model was established by Wichtmann et al. (2005) and can predict strain or stress accumulation under cyclic loading with numerous cycles and small strain amplitudes ($\varepsilon^{ampl} < 10^{-3}$). It finds applications in railways, water gates, and offshore wind turbine foundations. Developed from cyclic triaxial drained tests on sand specimens, the model considers factors like

strain amplitude, preloading history, average stress, density, and grain size distribution, influencing strain accumulation direction and intensity (Wichtmann et al., 2005).

Wichtmann et al., (2009) proposed the following equation to describe the effect of different functions on the intensity of accumulation:

$$\dot{\varepsilon}^{acc} = f_{ampl} \dot{f}_N f_e f_p f_Y f_{\pi}$$

,where f_{ampl} =strain amplitude, f_N =cyclic preloading, f_e =average void ratio, f_p = average mean stress, f_Y = average stress ratio, f_{π} = polarization change.

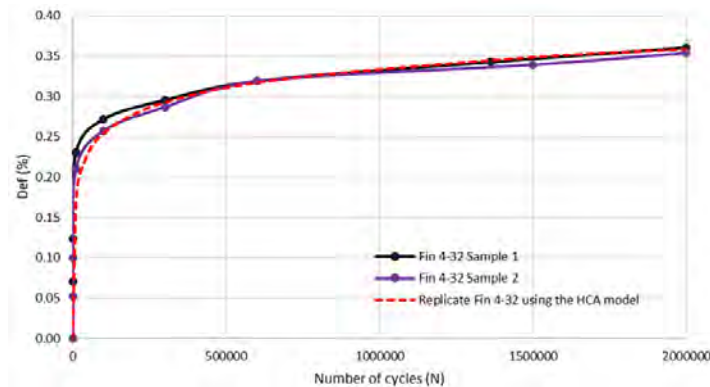


Figure 3 Replication of Fin 4-32 samples under cyclic compression tests by using the HCA model.

A Plaxis model, as shown in Figure 4, has been created to extract values such as strain amplitude and stresses because the laboratory test conditions differ from those on the construction site. With adjustments made to the amplitude and stresses in the HCA model, the correlation between the number of cycles (HCA model) and time (Plaxis) is presented in Figure 5.

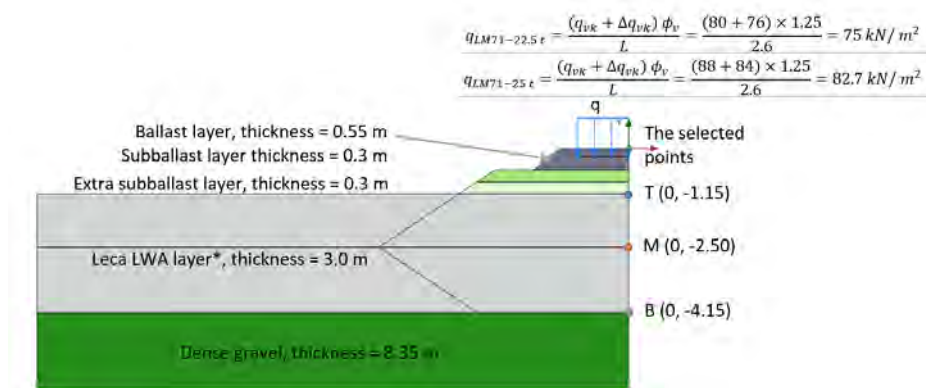


Figure 4 Plaxis model for cyclic loading analysis *This approach of modelling Leca LWA (without proper excavations) must ensure the sufficient embankment stability.

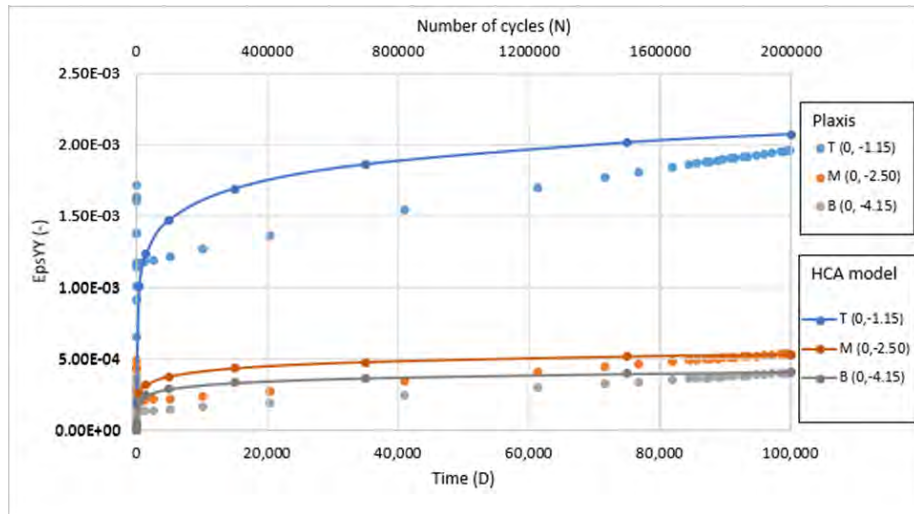


Figure 5 Vertical strain correlation between number of cycles (N) and time (T)

Nurmikolu (2004) reported that 3 million cycles can be approximated to represent 150 million tons of railway track load when using two adjacent 250 kN axle loads as one cycle. Therefore, the number of cycles per year for the selected route can be estimated using the same approximation. For example, the gross tonnage per year is 17.2 million from Lahti to Kouvola (Väylävirasto, 2019). Thus, the estimated number of cycles per track per decade is 1.7 million.

ANALYSIS AND RESULT DISCUSSION

All parameters used for calculations and models can be found in the first reference, which is the master's thesis of the main author.

Bearing capacity

To verify sufficient bearing capacity at the top of subballast layers when using Leca LWA with different cover depths, this study employs the Odemark method for calculation. A minimum bearing capacity of 180 MPa must be achieved when conducting the plate loading test (InfraRYL, 2020). The adopted Odemark E modulus for subballast is 300 MPa (InfraRYL, 2020), and for Leca LWA, it is 50 MPa (Pahkakangas et al., 2020). The results suggest that a 300-mm thick extra subballast layer is optimal because the minimum required bearing capacity at the top of the subballast layer can be achieved regardless of the subgrade bearing capacity.

Embankment stability

The cross-sectional dimensions vary (see Table 1). A Python script runs 52 analyses in Plaxis. The HS soil model is used for ballast, subballast, extra subballast, and Leca layers, with parameters sourced from Kalliainen &

Kolisoja (2017), Watn et al. (2004), and Høva et al. (2009). Parameters for subgrades are derived from the Murro test embankment (Koskinen et al., 2002), representing typical Finnish soil conditions with a thin dry crust over soft clay.

The results indicate a clear improvement in the factor of safety against slope failure as the thickness of the extra subballast layer increases from 0 to 0.3 m. However, there is no clear indication from the results whether an additional subballast layer further enhances embankment stability when its thickness exceeds 0.3 m. This finding supports the same optimal location, beneath a 300-mm extra subballast layer, for Leca LWA as determined from bearing capacity calculations using the Odemark method.

Displacement in Leca LWA layers

The displacement occurs in Leca LWA layers including the plastic displacement induced by cyclic loading and the elastic displacement caused by static train load.

As it is verified using 0.3-m thick subballast layer has been proven a positive impact in terms of bearing capacity and embankment stability.

As mentioned in the last chapter, the gross tonnage per year is 17.2 million from Lahti to Kouvola (Väylävirasto, 2019). Thus, the estimated number of cycles per track per decade is 1.7 million. under the axle load of 25 tons, this corresponds to 2.0 mm displacement at T, 1.4 mm at M, and 0.9 mm at P (positions of T, M, and B refer to Figure 4) when the thickness of the extra subballast layer is 0.3 m.

Nevertheless, these values are relatively small and can be almost neglected, especially for lower classifications of railway substructures. In other words, Leca LWA materials can satisfactorily withstand cyclic loading caused by moving trains, regardless of the cover depth.

CONCLUSION

This paper shows that placing Leca LWA below a 300-mm extra subballast layer significantly boosts railway embankment stability. However, limited improvement is seen with extra subballast layers over 300 mm. Leca LWA withstands cyclic loading regardless of cover depth, but these findings are specific to certain embankment setups.

Displacement from cyclic loading is analyzed using the Plaxis SSC model, validating the HCA model by Wichtmann et al. (2005). While helpful for estimating plastic strain under high-cycle train loads, there are limitations. For instance, Plaxis simulations show a linear relation, differing from the logarithmic one in the HCA model, leading to initial plastic strain

underestimation. Also, differences between sand and Leca LWA pose test condition challenges.

This study evaluates Leca LWA for bearing capacity, embankment stability, and cyclic loading, guiding geotechnical designers. It also highlights simulation model feasibility for assessing plastic strain from high-cycle loads.

Future studies could explore higher cycle numbers in compression experiments for understanding long-term displacements from cyclic loading. Verifying assumptions about pre-consolidation pressure through test embankments with Leca LWA layers would be valuable. Additionally, combining Leca LWA with other ground improvement methods could offer cost-effective railway embankment designs.

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USING WOOD-BASED BY-PRODUCTS FOR IMPROVEMENT OF QUICK CLAYS

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KEYWORDS

Quick clays, bioashes, LCA, ground improvement, laboratory analyses

ABSTRACT

To improve the engineering performance of quick clays, the dry deep soil mixing method is applied using binders (lime, cement) which production is associated to large CO₂-emissions. Therefore, there is a strong interest in finding alternative binders accompanied by integral analyses including the production, transport, installation, and performance of such materials during ground improvement works. This study focuses on a large laboratory study of mixing wood-based by-products, also called bioashes, in two forms: bottom-ash and fly-ash, each one combined with cement, into quick clay to investigate the strength and deformation properties of the improved quick clay, as well as the leaching of contaminants for its environmental characterization. The laboratory study is complemented with a Life Cycle Analysis (LCA) on the use of bioashes for quick clay stabilisation. These findings reveal that bioashes may provide sustainable alternatives to lime or cement in soil stabilisation of quick clays.

INTRODUCTION

Quick clays are clay types characterized by a collapsible grain structure and a very low remoulded shear strength. They are mainly found in Norway, Sweden, Canada, and Russia, and can cause large and destructive landslides. To improve the engineering performance of such clays, several ground improvement methods are usually applied. One of them is the in-situ deep soil mixing with binders such lime, cement, or a combination of these. However, cement and quick lime production are associated to large CO₂-emissions, and therefore, there is a strong interest in finding alternative binders accompanied by integral analyses including the production, transport, installation, and performance of such materials during ground improvement works.

The possibility of adding by-products in combination with cement or/and lime to modify the engineering properties of clays has been studied before (Wu et

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al., 2021; Åhnberg, 2006) and some specific research has been focused on the use of wood-based ashes or bioashes (FHWA, 2003; Vegdirektoratet, 2007; Bujulu, 2008; Deb and Pal, 2012; Rangaswamy, 2016; Islam et al., 2018; Cheng et al., 2018; Hov et al. 2023). Still research is needed on specific by-products and its application to quick clay stabilisation .

Recently Paniagua et al. (2023) presented proof-of-concept results when mixing four types of wood-based ashes and cement with quick clay. It was shown that the strength and stiffness of the original material increased, and that the materials leached elements in quantities below or close to the leaching limits, after the European regulation, for being disposed on inert landfills. Paniagua et al. (2023) study focused on comparing the performance of the different wood-based by-products and therefore, just one dosage amount and one binder ratio were used.

The present study takes further Paniagua et al. (2023) conclusions and performs a larger laboratory study using wood-based by-products in two forms: bottom-ash and fly-ash, each one combined with cement, Multicem® or cement and lime into quick clay. Different binder ratios and dosages were combined to investigate the strength and deformation properties of the improved quick clay with time. In this paper, the focus is given to the strength development with time for the stabilized material. The study also includes a Life Cycle Assessment (LCA) on the use of bioashes for quick clay stabilisation .

MATERIALS AND METHODS

Tiller-Flotten quick clay

The Tiller-Flotten quick clay is a low plastic marine clay with a clay content (fraction $< 2 \mu\text{m}$) of $\sim 50\text{--}75\%$ (L'Heureux et al., 2019). At the depth where samples were taken (i.e., 7–19 m), the water content of the clay is 40–50% and the unit weight is $\sim 18 \text{ kN/m}^3$. The pore water has a salt content (NaCl) of $\sim 2 \text{ g/l}$. The clay fraction is dominated by biotite (26%) and illite/muscovite (23%), followed by chlorite (16%), plagioclase (13%), amphibole (7%), quartz (7%) and potassium feldspar (4–6%). Unconfined compression tests on specimens taken from block samples give an undrained active shear strength of 50–70 kPa and an elasticity modulus (E50) of 2,5 MPa.

Wood-based by-products

The wood-based by-products (bioashes) come from the combustion centre of Bergene Holm at Brandval, Norway. The bioashes were tested in two forms: fly ash (FA) and bottom ash (BA). They come from the combustion of wood at 1000–1200°C (i.e., boiler temperature) with a type of fuel composed by approx. 40% dry wood chips and approx. 60% bark. XRF and XRD analyses

on the ashes show the main presence ($> 45\%$) of calcium oxide (CaO) in the form of portlandite ($\text{Ca}(\text{OH})_2$) for BA and calcite (CaCO_3) for FA.

Other binders: cement, lime and Multicem®

The wood-based binders were mixed with either cement (CEM), Multicem (MC), or cement and lime (B80), in different proportions. The cement used was a Portland limestone cement (CEM II/A-LL 42.5 R) produced in Skövde, Sweden. The Multicem (MC) used combined cement and Cement Kiln Dust (CKD), which is a by-product from cement production, in a 50/50 proportion. The lime (B80) used contained 80% quick lime and 20% lime kiln dust and it is commercialized as Terraplus B80 (B80).

Sample preparation & testing program

The laboratory analysis included geotechnical and environmental tests performed at NTNUs Geotechnical Laboratory. Before mixing, both ashes were dried in an oven set to 50°C for 24 hours to remove any excess water content. They were sieved to a grain size $< 250\ \mu\text{m}$. Particles between $250\ \mu\text{m}$ and $2\ \text{mm}$ were grinded to powder in a mortar before being sieved again to sizes $< 250\ \mu\text{m}$.

The wood-based ashes were combined with the other binders in different ratios and with different binder contents. The binder content was varied around $60\ \text{kg/m}^3$ since this recently is a typical amount used in quick clay stabilisation (Hov et al. 2022). Table 1 presents an overview of the laboratory program followed for preparation of specimens with either BA or FA (named A as a general form). Three samples were made in each combination type.

Table 1 Laboratory program with curing time in [days]

Binder type	Binder ratio [%]	Total binder content [kg/m^3]		
		40	60	80
CEM	100	28	---	---
CEM/A	50/50	7,14,28	7,14,28	7,14,28
	35/65	28	---	7,14,28
	25/75	7,14,28	7,14,28	7,14,28
	15/85	7*,14*,28	7,14,28	7*,14*,28
MC/A	50/50	7,14,28	7,14,28	7,14,28
CEM/B80/A	25/25/50	7,14,28	7,14,28	7,14,28

*Just BA samples

The stabilized soil samples had a diameter of 50 mm and a height of 100 mm and were prepared according to the procedure outlined in NGF (2012). The procedure consists of mixing the remoulded natural clay and the dry binder mixture for 2–4 min and then dynamically compacting the specimens in layers of around 20 mm within plastic moulds. The specimens were cured at 20°C . The samples were then tested in unconfined compression (UC) tests after the curing times described in Table 1. Measurements of water content and density

for all stabilized samples cured for 28 days were done prior UC testing. The pH after 28 days of curing was also measured in some selected samples.

Life Cycle Assessment (LCA)

An environmental evaluation on the utilization of bioashes was carried out by an LCA. This methodology considers the life cycle of the product under analysis, the environmental impact, and challenges of the associated processes to the product under study and quantifies such environmental burdens to make comparisons between the systems under analysis (Bjørn et al. 2018). A complete description of the performed LCA analysis is presented in Jafarbiglookarami (2021). Here, a summary of this analysis is given.

To define the scope of the study, the whole process was divided into three sub-systems. The first two systems explain the processes leading to the production of ashes and cement. The third system describes the in-situ processes for applying deep mixing stabilisation over one cubic meter (1 m^3) of stabilized soil. A comparative and attributional type of LCA is done and it is meant to be a cradle-to-site (Song et al. 2020). The use, maintenance and disposal of the material are not included in the boundaries of the study. The transport sector between the installation site and the ash/cement production plants are also omitted in the studied boundaries since this can vary according to the project location. The service life for the mixed soil is also not included in the analysis as the operational lifetime may vary to a great extent, and no information is available in this context regarding the wood ash mixture.

The Life Cycle Inventory (LCI) includes:

- Binders LCI: the ashes are considered a by-product and therefore the environmental loads can be ignored since they are transferred to another sector. The cement is the only source of environmental burdens from the binders. According to the Environmental Product Declaration for the cement used this study emits 620 g CO₂-equivalent.
- Deep mixing LCI: Fig. 1 presents the system boundaries defined for evaluating the environment footprint of the deep mixing method which includes mixing of binders, penetration of the mixing tool and injection of the binders while retracting the mixing tool. The mixing machine will require diesel to operate and it is influenced by the operation time of the machine and the ratio of binders to be injected in the soil (since this is influencing the mixing energy). In the analysis, it is assumed that the binders are delivered mixed. The diesel and cement are grouped as background processes (raw resources) and the mixing, penetration and injection are group as foreground processes.

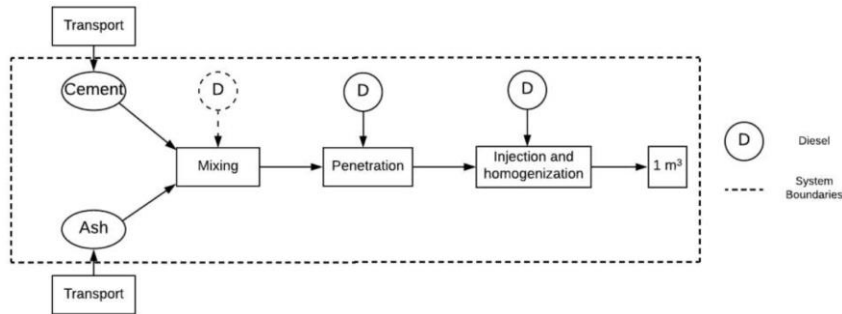


Figure 1. Proposed LCI for deep mixing. The process starts with mixing the binders in dry form, then penetration of the mixing tool, followed by injection (pumping) and homogenisation of the soil-binder mixture. The diesel needed for mixing the binder before injection in the soil is shown with a dashed line since it is neglected for simplicity. Taken from Jafarbiglookarami (2021).

The Life Cycle Impact Assessment (LCIA) framework used is the ReCiPe2016 (Huijbregts et al., 2017) which consists of different impact categories in human health, ecosystem, and resource extraction. More information is given in Jafarbiglookarami (2021). The selected impact categories include the ones listed in Table 2. Table 3 presents a summary of the assumptions made for LCA requirements.

Table 2 Selected impact categories for this study (Jafarbiglookarami, 2021).

Impact Category	Abbreviation	Units
Global warming	GWP100*	kg CO ₂ -eq
Terrestrial acidification	TAP100	kg SO ₂ -eq
Photochemical ozone formation (total)	POFP100	kg NO _x -eq
Ozone depletion	ODP100	kg CFC11-eq
Freshwater eutrophication	FEP100	kg P-eq
Terrestrial ecotoxicity	TEP100	kg 1,4-DB
Human Toxicity (total)	HTP100	kg 1,4-DB

*100 for the Hierarchist value choice

Table 3 Assumptions made for LCA requirements (Jafarbiglookarami, 2021).

Diameter and Length (m)	0.8 m & 2 m, respectively
Time required for penetrating 2 meters (up and down)	0.8 min
Time required for injection and homogenization of 2 meters (i.e., compressor running time)	0.8 min
Penetration diesel consumption	30 L per hour
Compressor diesel consumption	5 L per hour

RESULTS AND DISCUSSION

Water content and density

The water content after 28 days of curing (Fig. 2) depend on the water content of the original clay. Since the clays were sampled from different depths and

show different water contents in its natural state, it is convenient to express the results in terms of water content loss. The results show an increase in water content loss with an increase in binder content, no matter the ash type used. This is an expected observation since more soil water is used for binder hydration when more binder is added. On the other hand, when the ratio of ash to cement is higher (i.e., more ash is added respect to cement), the water content loss reduces. For the case of the ratio ash to MC or CEM/B80 no other differences in water content loss are observed.

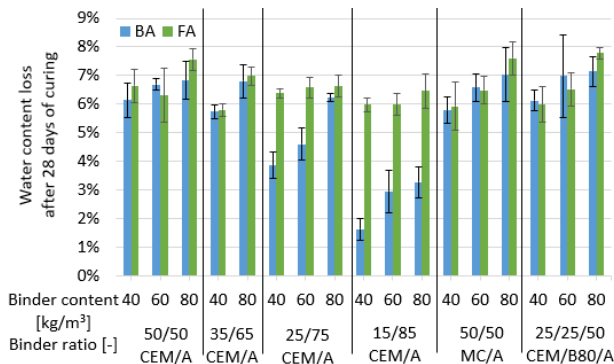


Figure 2. Water content loss after 28 days of curing.

The density of the specimens cured for 28 days is presented in Fig. 3. The density of the natural clay is also shown (grey shaded area). The measured values of the stabilized clay are slightly lower than the in-situ values as also presented by Paniagua et al. (2019).

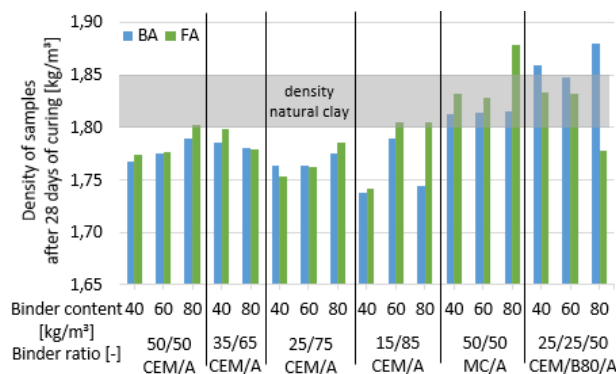


Figure 3. Density of samples after 28 days of curing.

pH results

Fig. 4 presents the pH values measured in some selected samples cured by 28 days and in the materials (clay, cement, and ashes) prior to being mixed (i.e. in a single state). The results show that samples mixed with CEM/BA binder

have higher pH compared to CEM/FA specimens. Also, the CEM/A stabilized specimens show higher pH than the MC/A and CEM/B80/A specimens. The high pH of BA contributes to increasing the pH in the clay mixture for the pozzolanic reactions to occur. As discussed by Sargent (2015), an aqueous media with a pH > 10.5 is usually necessary for pozzolanic reactions. In this sense, a higher strength gain for the samples mixed with BA is expected.

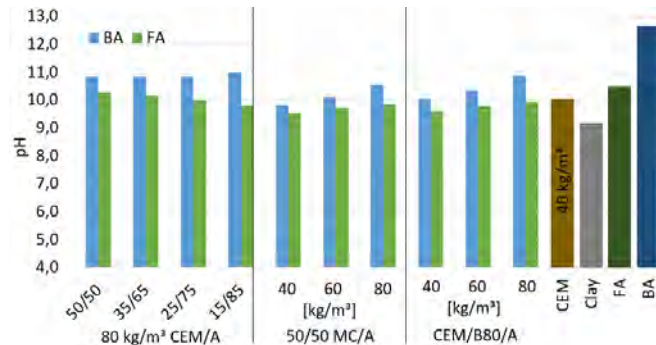


Figure 4. pH of stabilized samples at 28 days of curing

Strength results

Fig. 5 and Fig. 6 present the results of the maximum shear strength from UC tests measured at 28 days of curing and its variation with the amount of binder added to the mixtures. The results are presented in a form where the content of BA or FA, CEM, MC and B80 are separated to facilitate the discussion on the influence of adding the wood-based ashes for quick clay stabilisation.

Taking as a reference the sample stabilized just with 40 kg/m³ of CEM, one can observe that adding the same amount of BA gives an increase in strength. The strength also increases, but not as much as in the previous case, if the CEM content is then reduced to half and B80 is added instead in the same proportion as the CEM, keeping the ratio of CEM/B80 and BA equal.

Reducing the CEM content by approx. 25-30% from the 40 kg/m³ added to the reference sample and adding either BA in the same proportion of CEM (for a total binder content of 60 kg/m³) or in a ratio CEM/BA of 35/65 (for a total binder content of 80 kg/m³) gives stabilized clay samples with similar strength values and slightly below the strength reached for the sample stabilized with just CEM. A combination of 80 kg/m³ of MC/BA in a 50/50 ratio gives also similar strength results (one should remember that MC has 50% CEM in its composition, which then is also a reduction of the CEM used compared to the 40 kg/m³ sample).

Even though some of the results in Fig. 5 show a lower strength gain compared to the just CEM stabilized samples, the strength measured on these specimens are above the strength in UC test for the natural clay and might be enough depending on the required strength for the application of stabilized

quick clay in practice. Looking at Fig. 6, the results indicate that the FA are not performing as well as the BA. The measured strength when combining CEM/FA in 50/50 proportion is slightly over the measured strength when just CEM was added. The incorporation of FA results in a relatively poor performance dependent on the cement content and the total binder content.

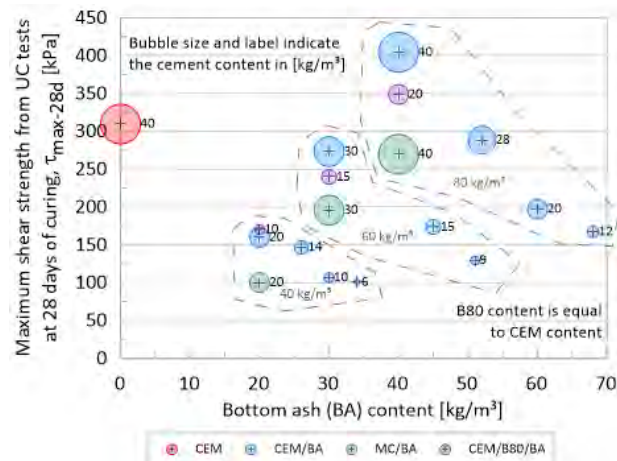


Figure 5. Variation of strength of stabilized clay samples, after 28 days of curing, with the content of BA, CEM and other binders.

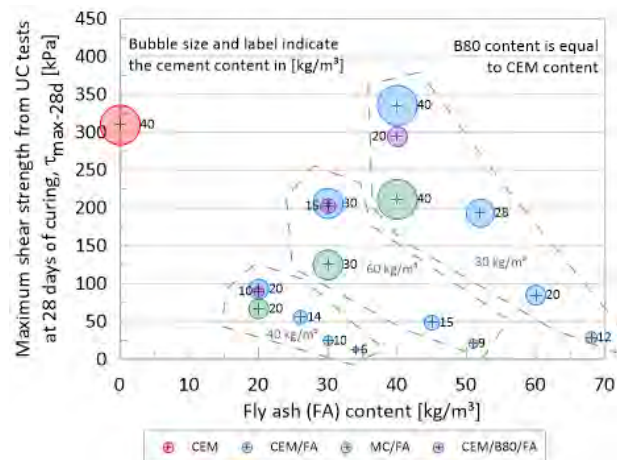


Figure 6. Variation of strength of stabilized clay samples, after 28 days of curing, with the content of FA, CEM and other binders.

The XRF and XRD analyses of ashes (Jafarbiglookarami, 2021) described that calcium oxide (CaO) is the dominating composition for both types of ashes in the form of Portlandite for BA and Calcite for FA. The BA contained higher silicium oxide (SiO_2) and aluminium oxide (Al_2O_3) (ca. 3%) higher than FA. This indicate that BA may have better pozzolanic properties than FA, however, the effect is very limited for both types of ashes. On the other

hand, the flocculation phenomena can be enhanced by ions with higher electron charges, the data presented by Jafarbiglookarami (2021) indicate a higher concentration of phosphor, manganese, magnesium, and potassium in BA, which may indicate better exchange properties for BA. These results indicate that the FA have a more limited reactivity than BA and therefore can explain the lower strength values measured when the clay was stabilised with FA and CEM. Still, the initial cementing reactions are mainly related to the cement as expected and when BA was also added, BA might contribute to the pozzolanic reactions.

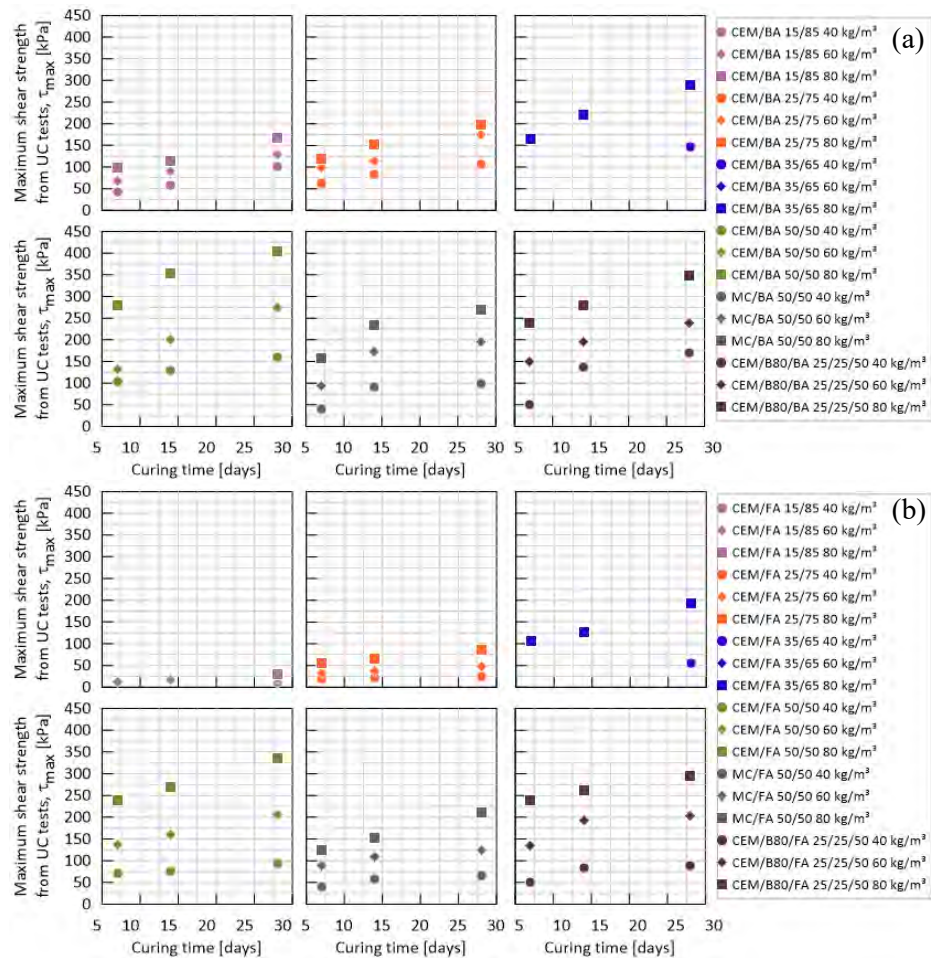


Figure 7. Variation of strength with curing time for different specimens stabilised with CEM and (a) BA or (b) FA in the proportions indicated.

Fig. 7 shows the strength increase in time for the stabilized clay samples, with the content of a) BA or b) FA, CEM and other binders. As expected, there is an increase in strength with curing time, however, this increase is faster for specimens stabilised with BA and CEM, compared to specimens stabilized

with FA and CEM. The results also indicate the strong influence of the cement proportion when added in combination with FA: cement contents lower than 25% respect to the FA content give a limited strength gain with time and values below 100 kPa after 28 days of curing.

LCA results

The first assessment of the impact indicators was focused in the in-situ processes without considering the transport to make the results of this study no-case dependant. Then, the system under study just consumes cement and diesel and the results obtained indicated that the environmental impact indicators are proportional to the cement content, which then emphasises the importance in its reduction for deep mixing application. The authors strongly suggest revising Jafarbiglookarami (2021) for further details in this analysis.

Then, considering the low cement combinations into further analysis, the contribution of transport section becomes unneglectable due to its consumption of diesel. Therefore, the system boundaries were extended further from the ones presented in Fig. 1 and the results obtained from the LCA are presented in Fig. 8. It is observed that transport impact increases with distance.

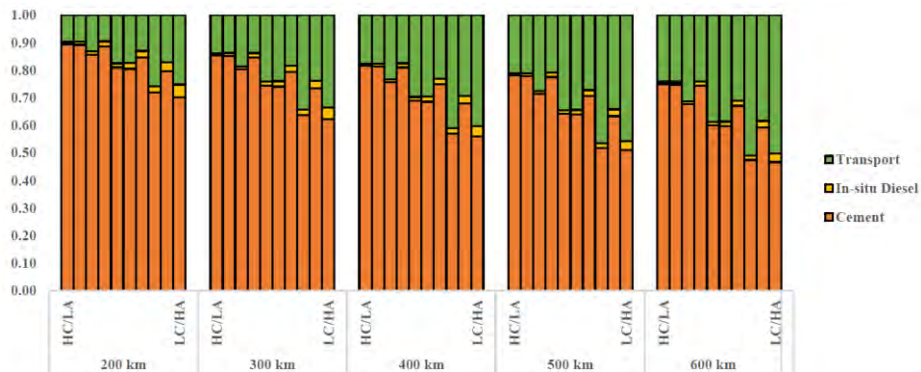


Figure 8. GWP contribution of transport, diesel and cement for the different distances between the ash source and project location (200 km to 600 km). HC denotes high cement content, LC denotes low cement content, HA denotes high ash content, and LA denotes low ash content. Taken from Jafarbiglookarami (2021).

Finally, to identify the binder combination giving the best performance in mechanical and sustainable terms, a minimum strength of 75 kPa was selected. With this classification, all combinations including BA and some of the combinations including FA were further utilised in the analysis (transport was not included in this analysis). Figure 9 presents the ratio between GWP100 and the maximum shear strength from UC tests at 28 days of curing per m of column installed (a column of 25 m was assumed in the analysis) vs the ash and cement content in selected mixtures. Then, the optimum combination in the case of mixtures including FA is the 80 kg/m³ in a 50/50

proportion of FA since it is giving the lowest amount of kg-CO₂ per kPa of strength and per m of column installed. In the case of the mixtures including BA, the combinations showing the lowest GWP impact are associated to strengths near the 100 kPa and include a relatively large proportion of BA respect to CEM. However, if higher strengths are required, the optimal binder content is calculated for 60 kg/m³ of CEM and BA in a proportion of 25/75. It should be mentioned that similar trends were observed for the other impact categories presented in Table 2, and these are detailed in Jafarbiglookarami (2021).

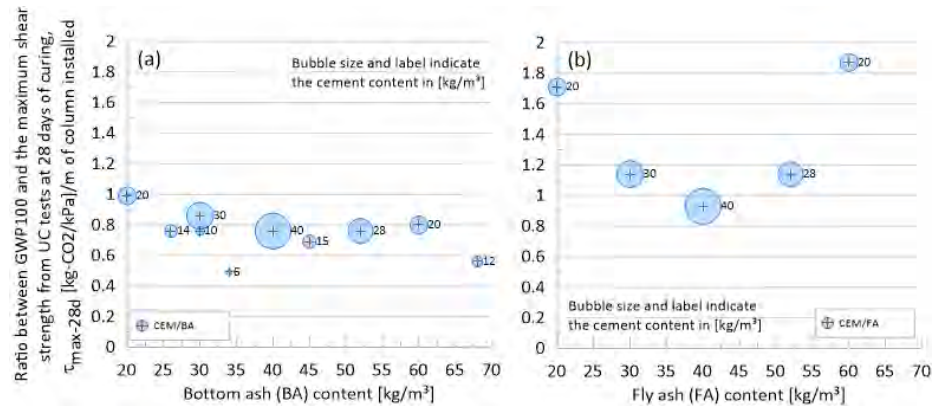


Figure 9. Ratio between GWP100 and the maximum shear strength from UC tests at 28 days of curing, per m of column installed vs the CEM and (a) BA and (b) FA content.

CONCLUSIONS

After combining bioashes with cement into quick clay, the strength and stiffness after 28 days of curing increased up to 5 times the values of the undisturbed clay. The increase of strength and stiffness with time was also measured, getting the largest strength gain for the samples improved with bottom ashes. Both types of ashes showed to be rich in calcium oxide and alkaline structures which indicated the positive contribution to the rate of chemical reactions to occur during quick clay stabilisation. The better reactivity for improving the strength was observed for the bottom ashes, however, due to the low particle size of the fly ashes, these could act as a non-reactive filler along with bottom ash, this requires further study.

The Life Cycle Analysis (LCA) on the use of bioashes for quick clay stabilisation revealed that the environmental costs relative to the obtained geomechanical response might sit in an acceptable range for bottom ashes and compares to traditional lime cement mixtures. The results indicate that the mixtures including BA and CEM performed better than the ones combining FA and CEM. However, in order to upscale the application of such wood based by-products; several barriers need to be overcome in particular regarding their physical characteristics that make them compatible to the

current technology for deep dry mixing. In addition, the production of such materials in terms of amount and quality needs documentation and a further discussion: there is an inherent variability in the ashes due to the yearly variability of the raw materials. The quality documentation should be aborded from the mechanical effect (in terms of strength increase) and the environmental impact related to for example leaching of contaminants.

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UTILIZING CRUSHED CONCRETE AGGREGATE IN THE CONSTRUCTION OF ROADS AND STREETS IN ACCORDANCE WITH FINNISH NEW GUIDELINES AND REGULATION

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KEYWORDS

crushed concrete aggregate, C&DW, substrates, guidance, regulation

ABSTRACT

Construction and demolition waste (C&DW) are one of the major waste types produced by modern society. To decrease the use of non-renewable natural resources as well as environmental effects of earthworks, natural aggregate materials can be replaced with recycled materials such as recycled crushed concrete aggregate. Since early 1990's crushed / recycled concrete aggregate (CCA / RCA) has been successfully used [1] in Finnish road constructions.

The aim of the article is to introduce the new guidelines [2,3,4,5,6,7] and the End-of-Waste regulation [8] for RCA published in Finland in years 2022-2023, as well as their impact on the production and utilization of concrete aggregate in the construction of roads and streets. New guidelines are including the guidance of using RCA in urban areas (street, parks, trenches, and technical networks) as well as new practices and guidelines using RCA in roads. The article also describes the new opportunities to utilize RCA introduced by End-of-Waste regulation. The new opportunities are e.g., RCA usage in substrates. New guidelines and regulations have standardized the utilization of concrete aggregate in civil engineering.

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INTRODUCTION

Construction and demolition waste (C&DW) constitutes a major waste category by modern society. In earthworks, reducing the reliance on non-renewable natural resources and mitigating the environmental impacts can be achieved by substituting natural aggregate materials with recycled alternatives, such as recycled crushed concrete aggregate. Since the early 1990s, crushed or recycled concrete aggregate (CCA / RCA) has been successfully utilized [1] in Finnish road construction.

REGULATIONS

In Finland, the most common ways to utilize RCA are Government Decree on the Recovery of Certain Wastes in Earth Construction 843/2017 [9], Environmental Permit or Government Decree on Criteria for Assessing the End of Classifying Crushed Concrete as Waste 466/2022 [8] (Table 1). Utilization of RCA is significantly impacted by the location of the site from an environmental perspective. For example, the proximity to groundwater areas or other sensitive nature sites may limit the use of RCA. In this context, it is necessary to evaluate which process is appropriate for the purpose of utilization.

Table 1. List of main regulation and decrees for utilization of RCA in Finland.

Type	Description
Government Decree on the Recovery of Certain Wastes in Earth Construction 843/2017 (in Finnish: " <i>Valtioneuvoston asetus eräiden jätteiden hyödyntämisestä maarakentamisessa</i> ")	The purpose of the decree is to promote utilization of wastes by defining the conditions under which the use of wastes specified in the decree for construction purposes does not require an environmental permit according to the Environmental Protection Act (527/2014).
Environmental permit under Environmental Protection Act 527/2014 (in Finnish: " <i>Ympäristönsuojelulaki</i> ")	An environmental permit must be applied for activities that may cause environmental pollution or the risk of pollution. The Environmental Protection Act defines professional or institutional waste treatment as an activity requiring a permit.
Government Decree on Criteria for Assessing the End of Classifying Crushed Concrete as Waste 466/2022 (in Finnish: " <i>Valtioneuvoston asetus betonimurskeen jätteeksi luokittelun päättymisen arviointiperusteista 466/2022</i> ")	The objective of the decree is to establish nationally applicable criteria for ending the waste classification of RCA, so that these aggregates can be introduced to the market as aggregate products and fertilizer products.

Government Decree on the Recovery of Certain Wastes in Earth Construction defines the criteria that, when met, allows the use of RCA in earth construction with notification of registration. Under the decree, the use of RCA is permitted in road and field structures, as well as in the foundation

structures of industrial and storage buildings. The Centres for Economic Development, Transport and the Environment (ELY Centre) processes the registration notification. Also, a final report describing the amount of RCA used in the site, in which structures, and coordinates of the structure is submitted to ELY Centre upon completion of the construction [9].

If the intended use, structure, or properties of the RCA do not fall within the scope of Government Decree on the Recovery of Certain Wastes in Earth Construction, utilization may be possible with an environmental permit. Regional State Administrative Agency (AVI) processes the environmental permit if the amount of RCA to be utilized exceeds 50,000 tonnes per year, or the municipal environmental protection authority if the amount is less than 50,000 tonnes. Environmentally permitted sites usually require post-monitoring and documentation. For example, monitoring of pH in groundwater and surface waters nearby the utilization site.

The national End-of-Waste (EoW) decree for RCA in Finland came into effect on September 1st, 2022. The decree defines assessment criteria for when RCA ceases to be considered as waste. The decree is applied to manufacturers who have an environmental permit for waste crushing operations under the Environmental Protection Act. Adoption of the decree is voluntary for RCA producers. In addition, the EoW decree does not override existing legislation. EoW RCA is treated as any other aggregate materials, and its use in construction does not require a permit from authorities. The user must still consider the characteristics of RCA. For example, aluminium and polyester cannot be used in RCA structures due to its high pH compared to natural aggregates.

RCA utilized in accordance with End-of-Waste decree does not require such monitoring or documentation as in Government Decree on the Recovery of Certain Wastes in Earth Construction and environmentally permitted sites. However, clients may require contractors to provide information about the structures separately (for example, city of Helsinki requires documentation provided in accordance with guidelines for documenting underground and underwater structures).

EoW decree enables new opportunities to utilize RCA, such as gabion structures, thicker structural layers such as embankments (Government Decree on the Recovery of Certain Wastes in Earth Construction has 1.5 m layer thickness limit), landscaping (no covering requirement of RCA) and substrate production (raw material source of RCA must be unused concrete products from concrete industry), and the use of RCA as aggregate in concrete production. At the beginning of 2024, there are nine EoW RCA manufacturing locations in Uusimaa region.

Besides utilizing RCA by Government Decree on the Recovery of Certain Wastes in Earth Construction, Environmental Permit or EoW decree, there are

a few exception occasions for the permitting. If the use of RCA is minor, utilization may be possible with a permit granted by the municipal environmental protection authority. Environmental regulation does not define minor waste usage, but the upper limit is often considered to be between 100 and 1000 tonnes, depending on the quality of the waste.

In addition, utilization of RCA may also be possible with an experimental operation permit, which is exempt from the need for an environmental permit. The use of an experimental procedure requires that it concerns a new activity, or, for example, waste processed in a new way, which functionality is desired to be tested.

QUALITY AND PRODUCTION

RCA must be CE-marked for use in earth construction when a harmonized standard exists for the specific use. The CE marking of RCA is possible for aggregate with a particle size of less than 90 mm.

The ownership of RCA and a change in ownership have implications for the CE marking obligation of the material; if the owner of the material changes before the material is utilized in a construction project, it must be CE marked. The marking is not required when the RCA is not placed on the market, but its owner operates or commissions both the production of the RCA and its installation in the structure.

Regardless of the concrete waste crushing location and the CE marking (whether present or not), the process's quality control must be conducted in accordance with the SFS 5884 standard [2] (Table 2). The final product must meet the requirements of InfraRYL (Finnish quality requirements in infrastructure construction) and the product standards relevant to its use (e.g., SFS-EN 13242 Aggregates for unbound and hydraulically bound materials for use in civil engineering work and road construction) as well as requirements of environmental regulations.

Previously, four quality classes were used for crushed concrete: RCA I-IV. With the introduction of the End-of-Waste regulation for RCA, the classification has been changed so that the product's waste status is directly visible from the class. Classes RCA I and RCA II have been divided into two: RCA Ia & RCA Ib and RCA IIa & RCA IIb. The “a” classes are used for RCA that ceases to be considered as waste. RCA classes Ib, IIb, III and IV will continue to be considered as waste. For RCA class I, the raw material source can only be concrete waste originating from unused concrete products from the concrete industry. For RCA classes II-IV, the raw material sources can include concrete waste from concrete industry as well as concrete waste originating from construction, renovation, or demolition sites.

Assessment and Verification of Constancy of Performance (AVCP) class differs between RCA which has waste status (RCA classes Ib, IIb, III and IV) and EoW RCA (RCA classes Ia and IIa). For waste status RCA, AVCP class is 4 which means the quality control of the material is done by the manufacturer. Manufacturer is responsible for the performance and determination of the product. In turn, the AVCP class for End-of-Waste RCA is 2+, which requires the involvement of third parties in quality control.

Table 2. Main quality and production guideline for utilization of RCA in Finland.

Type	Description
SFS 5884:2022, Factory Production Control of crushed concrete for earth and landscape construction (in Finnish: “ <i>Betonimurskeen maa- ja viherrakennuskäytön laadunvalvontajärjestelmä</i> ”)	The standard defines how concrete waste generated in the manufacturing of building products, construction production, as well as in the repair and demolition of structures and buildings, is processed into RCA that meets the technical and environmental suitability requirements set for earth and landscaping construction.

UTILIZATION AND CONSTRUCTION GUIDELINES

Numerous organizations have created guidelines for utilization of RCA. For instance, Finnish Transport Infrastructure Agency has published specific guidelines for utilizing RCA within its projects. Furthermore, municipalities have developed guidelines and protocols for using RCA in urban structures and plot construction. On the other hand, material suppliers have established detailed instructions for using their materials. Main guidelines and manuals for utilizing RCA in Finland are presented in Table 3. Guidelines are binding and manuals are binding only when the client decides so.

Table 3. List of main guidelines and manuals for utilization of RCA in Finland.

Type	Description
Helsinki, Espoo, Tampere, Turku, and Vantaa cities. 2024. Crushed concrete aggregate in municipal earth construction, Guidelines for design, construction, and maintenance (in Finnish “ <i>Betonimurske kaupunkien julkisessa maarakentamisessa</i> ”). [3]	The guide addresses utilization of RCA in urban street, park, and plot construction. It considers the special features of urban construction, such as underground structures and city-specific practices.
HSY. 2024 draft will replace the existing guide from 2014. Recycled crushed concrete – A usage guide for design, construction, and maintenance (in Finnish: <i>Betonimurske – käyttöohje suunnitteluun, rakentamiseen ja ylläpitoon</i>). [10]	The guide describes the use of RCA from the perspective of the water supply network, such as filling of pipe trenches.
UUMA. 2023. Utilization of recycled materials in network projects, manual (in	The manual presents how recycled materials, substituting for natural earth materials, can be utilized in various earth

Finnish: “ <i>Tekniset verkostot uusiomaarakenteissa</i> ”). [7]	structures associated with underground networks. It covers impacts of recycled materials on different network materials and the risk of permeation through water pipe walls.
Finnish Transport Infrastructure Agency. 2023. Recycled materials suitable for road construction (in Finnish “ <i>Väylärakenteisiin soveltuvia uusiomateriaaleja</i> ”). [11]	The annex compiles information on recycled materials which technical suitability has been assessed by May 30th, 2023. The criteria for assessing suitability are presented in the guide “Use of recycled materials in road construction”.
Finnish Transport Infrastructure Agency. 2022. Use of recycled materials in road construction (in Finnish: “ <i>Uusiomateriaalien käyttö väylärakentamisessa</i> ”). [12]	The guide introduces typical recycled materials for infrastructure projects, procedure for assessing the technical suitability of recycled materials, legislation, and the use of recycled materials.
Finnish Transport Infrastructure Agency. 2022. Technical suitability assessment of recycled materials (In Finnish: “ <i>Uusiomateriaalien teknisen soveltuvuuden arviointi</i> ”). [13]	This guide clarifies the procedures for assessing the general and project-specific technical suitability of recycled materials, as well as the roles and responsibilities of the assessment parties.
Finnish Transport Infrastructure Agency. 2022. Processing and utilizing of concrete waste in infrastructure projects (in Finnish “ <i>Betonijätteen käsittely ja käyttö väylähankkeissa</i> ”). [14]	The guide presents the process of processing concrete waste generated in infrastructure projects into technically and environmentally suitable RCA for infrastructure construction.
INFRA ry. 2021. Technical suitability and use of crushed concrete aggregate in road structures (in Finnish: “ <i>Betonimurskeiden tekninen soveltuvuus ja käyttö tierakenteissa</i> ”). [15]	The guide presents how the technical suitability of RCA has been assessed for use in projects under the Finnish Transport Infrastructure Agency and the ELY-Centres.
Rakennustieto Oy. 2023. InfraRYL (in Finnish: “ <i>Infrarakentamisen yleiset laatuvaatimukset, InfraRYL</i> ”). [16]	InfraRYL describes general quality requirements and accepted best practices in Finnish infrastructure construction.
City-specific guidelines [4] [5]	City-specific guidelines present general requirements for the use of RCA in urban construction projects.
Material supplier guidelines	Various material suppliers have published their own guidelines.

SUMMARY

In Finland, utilization of crushed / recycled concrete aggregate (CCA / RCA) is primarily influenced by its origin. It is important to know whether RCA comes from a demolition site or is a waste product of the concrete industry. In addition, it is important to know whether RCA is crushed directly on site or whether it is sent to separate production plants for processing. Regarding the

production facilities, it is necessary to consider whether they have the proper environmental permits and whether they can process the concrete waste so that it can be processed into a product equivalent to natural aggregates under the End-of-Waste regulation.

In Finland, the most common ways to utilize RCA are Government Decree on the Recovery of Certain Wastes in Earth Construction, Environmental Permit or End-of-Waste decree (Table 4). Moreover, RCA can be used in accordance with municipal environmental protection authority's minor waste utilization procedure or with an experimental operation permit.

Table 4. Processing and utilization of RCA (simplified).

	RCA with waste status	End-of-Waste RCA
Processing ⁽¹⁾	Internal quality control	Third-party quality control
Regulation	Government Decree on the Recovery of Certain Wastes in Earth Construction, Environmental Permit	RCA End-of-Waste Decree
Construction	InfraRYL, Client specific guidelines and manuals, Material supplier guidelines	
Post-construction supervision	ELY Centres, Municipal environmental protection authority, Regional State Administrative Agency (AVI)	No supervision regarding construction of RCA structures which means the responsibility of client, constructor, and designer is emphasized.

¹⁾ *Assessment and verification of constancy of performance (AVCP) class 4 is sufficient for RCA with waste status which means internal quality control by the manufacturer is adequate. The AVCP class for End-of-Waste RCA is 2+, which requires the involvement of third parties in quality control.*

Technical requirements of the site impact on use of RCA. For instance, the load and environmental conditions must be suitable for the use of RCA, and the chemical properties of RCA must be compatible with the equipment and devices presents in the structures. When RCA is used in construction, quality assurance often occurs through InfraRYL (Finnish quality requirements in infrastructure construction). InfraRYL outlines the general quality requirements for RCA and provides guidance on conducting quality control measurements of built structure.

Various organizations have developed guidelines and manuals for the utilization of RCA. For example, Finnish Transport Infrastructure Agency has published its own guidelines for using RCA in its operations. Additionally, cities have released guidelines for utilizing RCA in urban structures and plots. Material supplies, in turn, create detailed instructions for using their own materials.

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WHERE AND HOW TO EXECUTE SUSTAINABLE SOIL IMPROVEMENT UTILIZING VIBRO REPLACEMENT (STONE COLUMNS) IN THE NORDICS

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KEYWORDS

Alternative soil improvement, stone columns, vibro replacement, environment, sustainable solution, local material and knowledge

ABSTRACT

In parallel to the increase of population in Nordic region, which is projected to be more than 10% toward 2030, urbanization shows a rising trend especially around the capital cities of Denmark, Sweden and Norway. As the statistics show that building and construction industry globally accounts for 37% of energy and process-related CO₂ emissions, significant efforts are required to transform these activities into a green industry by providing circular and sustainable solutions to achieve UN Sustainable Development Goals.

Based on the above it is expected from geotechnical engineers to look into to check constantly where and how the CO₂-footprint can be reduced by keeping the projects practical and safe. Therefore, it is obvious that commonly used ground improvement methods in Scandinavia, which are based on cement and lime material, need to be – at least partially - replaced with alternative techniques or alternative binders.

This leads to the investigation on application of more sustainable ground improvement methods like vibro replacement / stone columns. In this case no binder is needed at all and even the re-use of construction materials such as excavated/blasted rock is possible. This leads to a CO₂-reduction of 80 – 90% compared to the traditional methods. This paper aims to present the research done for the investigation of how and where vibro replacement can be used considering the typical local soil conditions. Some examples will be shown to underline the practicality of the approach considering typical KPIs based on time, costs and CO₂-impact.

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INTRODUCTION

In Sweden and Norway, geotechnical considerations play a crucial role in construction and infrastructure development due to the varied nature of soils across the regions. Both countries feature diverse soil types, ranging from soft clay soils, loose sands and gravels to strong bedrock, often influenced by glacial deposits and mountainous terrains. In urban areas, where development is dense, the composition and stability of soils become paramount factors in engineering projects. Soil improvement techniques are frequently employed to enhance the load-bearing capacity and stability of foundations, particularly in areas prone to settlement or landslides.

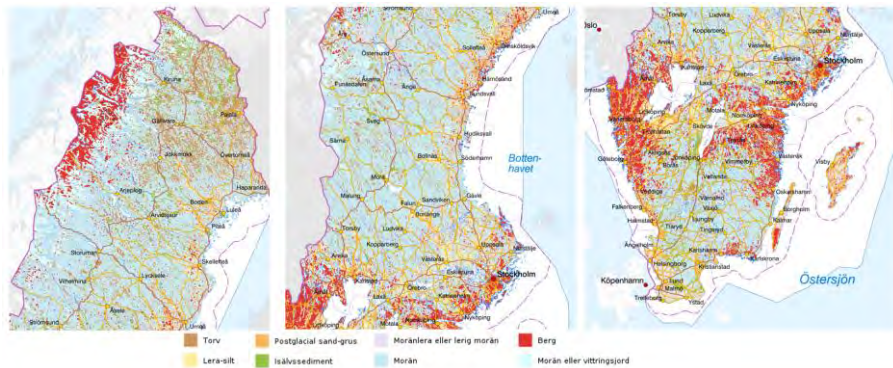


Figure 1 Soil types in Sweden (north, middle, south respectively-Source: SGU Website).

Ground improvement techniques in the Nordic countries contains a range of methods utilized to address the challenges posed by soft clays, peat, permafrost, and other adverse soil conditions. These methods include mainly dry deep soil mixing to increase load-bearing capacity, preloading with surcharges and prefabricated vertical drains to accelerate consolidation of compressible soils as well as soil replacement with engineered (light) fill materials. These techniques are essential for ensuring the stability and serviceability of infrastructure projects in the Nordic region, where challenging soil conditions are prevalent.

VIBRO REPLACEMENT AS A GROUND IMPROVEMENT METHOD

Vibro replacement is a globally used soil improvement technique. It involves using a bottom feed vibrator to penetrate fine soils like clay and/or silt, enlarging the voids created with crushed rock or gravel and densifying the installed material with the vibrator to create stone columns including a certain soil replacement of the soil during the construction process. These stone columns enhance soil behavior by increasing strength, stiffness, and permeability. While successful applications of vibro replacement have been documented in various countries, limited documentation exists in Nordics,

where challenging ground conditions like partially extremely soft and sensitive clays pose obstacles.

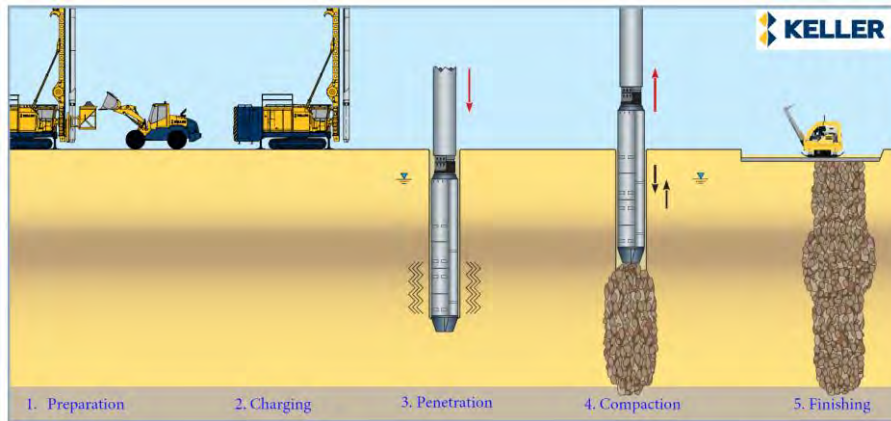


Figure 2 Process of vibro replacement-stone columns.

Stone columns offer a variety of benefits as reducing foundation settlement, increasing bearing capacity, allowing quick drainage of excess porewater, mitigating liquefaction potential, providing slope stabilization etc. In addition to stone columns, vibro concrete and vibro mortar columns are ground improvement techniques that also involve the use of vibration to improve soil. In these methods, a vibrating probe is inserted into the ground as in stone column process, which are then filled with concrete or mortar instead of gravel.

When working with sensitive and very soft soils, the application of vibro-stone columns encounters certain natural limitations. Specifically, for a first evaluation of a project, the remolded undrained cohesion of the soil shall exceed 3-5 kPa with a sensitivity value of 8-10 maximum. While pure stone columns may not be suitable for peat soils, special concrete can replace gravel within peat layers. Utilizing a bottom feed system with an adjustable frequency of the depth vibrator becomes essential in such scenarios. Moreover, the application in extremely soft soils is constrained by the absence of lateral confinement, further complicating the process. In this case geotextile or geo-grid encased columns can be a solution to overcome the bulging in the lack of lateral confinement.

Electronic devices are used with all vibro techniques to ensure and automatically record consistent high-quality work. They track various parameters like time, depth, speed, force, and current during penetration and compaction. This information can be shown graphically and printed out for each column, making it easy to monitor and document the process (see Figure 3).

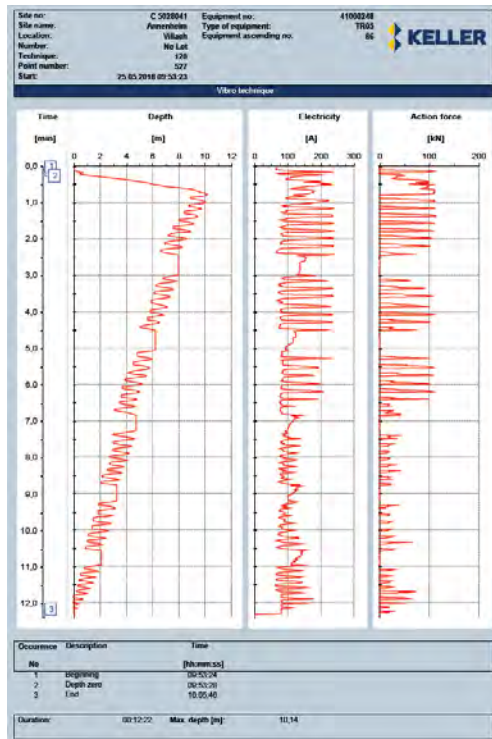


Figure 3 Typical quality control graph.

APPLICATION OF STONE COLUMNS IN VERY SOFT SOILS

The above stated constraints were confirmed by Keller's worldwide experience with vibro columns as being crucial for the safe and effective application of this solution. These projects have provided valuable insights into the soil conditions necessary to ensure the successful implementation of vibro techniques. Some of them have been provided and summarized on the table below.

Table 1. Examples of previous vibro projects with low soil shear strength parameters.

Location	Purpose	Type	Undrained shear strength
Sweden [1]	Road emb. stab.	Grouted stone column	8-14 kPa
Sweden [1]	Road emb. stab.	Vibro stone column	15-25 kPa
Austria	Bearing capacity	Vibro stone column	5-10 kPa
Germany	Road emb. stab.	Vibro concrete column	4-8 kPa
UK	Sewage works	Vibro stone column	7-10 kPa
Malaysia [2]	Road emb. stab.	Vibro stone column	5-15 kPa
Poland	Highway emb.	Vibro stone column	5 kPa

In addition to the summarized projects, it's noteworthy that Keller has further advanced its efforts by initiating a real execution trial with a research project

for stone columns in Norway, supported by RFF (Regionale Forskningsfond) Oslo. The VIBE project [3] is investigating sustainable ground improvement solutions for Oslo, with a particular focus on vibratory techniques and the incorporation of recycled raw materials. By conducting large-scale field tests at the Onsøy test field, the project aims to quantify the technical feasibility and environmental impacts of utilizing recycled materials in soil stabilization, especially in Norwegian clays. Initial results and partial findings offer a glimpse into the project's progress and its potential to revolutionize construction practices while promoting environmental sustainability.

The test field served as a platform to explore the impacts of vibrations, coupled with soil displacement, on soil parameters and pore water pressures in such challenging soil conditions. Further research is planned to investigate similar phenomena in typical fine-grained soils with slightly higher residual shear strength, aiming to broaden the understanding of these techniques in diverse soil compositions.

SUSTAINABLE SOIL IMPROVEMENT

Dry soil mixing is widely employed as the predominant method for improving ground conditions in the soft and sensitive clays of the Nordic region. This technique primarily relies on cement and lime as binding agents, which significantly adds to the emission of CO₂. Global increase in CO₂ emission due to cement production over the last 100 years [4] has increased 1500 times. While numerous studies have explored sustainable binders to reduce CO₂ emissions, vibro replacement offers a promising alternative solution to the same geotechnical challenge, presenting a more sustainable option that has already been proven effective.

The demand for natural aggregates in stone column construction raises concerns about environmental impacts, including transportation pollution and waste management. Utilizing recycled materials addresses these concerns by promoting waste reuse and recycling. Additionally, substituting primary aggregates with alternative materials like crushed or recycled rock from tunnel excavations or blasting offers sustainability benefits by reducing environmental impact and promoting waste diversion. However, materials like crushed concrete and crushed brick, though popular alternatives, face limitations due to their fine content and suitability for specific soil types. Challenges such as high sulphate content or organic and acidic environments restrict their applicability in certain contexts [5].

HOW TO MEASURE THE SUSTAINABILITY WITH EXAMPLES

The EFFC calculator [6] serves as a pivotal tool for evaluating the CO₂ emissions associated with different methods and materials, replacing numerous company-specific calculators and ensuring standardized

measurement of carbon output throughout the sector. Covering emissions from raw material extraction to foundation installation, this calculator facilitates the comparison of embodied carbon across various geotechnical solutions. Embraced widely by European geotechnical firms, some companies utilize it to compare projected and actual emissions, with the aim of minimizing their overall carbon footprint. For the examples shown on that paper, this tool has been utilized. Below are two examples demonstrating the quantification of equivalent CO₂ emissions for different soil improvement techniques using the EFFC calculator, providing a clearer visualization of their impact:

1st example:

A typical infrastructure project was chosen (see Fig. 4) to compare dry deep soil mixing, stone columns & vertical drains with respect to function, economy and CO₂-emission.

- Design of a 100m of road, 15m wide, 1m of dry crust + 9m of clay
- 35kPa load from embankment
- Settlements criterion according to Swedish Transport Administration: 35cm in 40years (Ref. speed 40-80km/h)

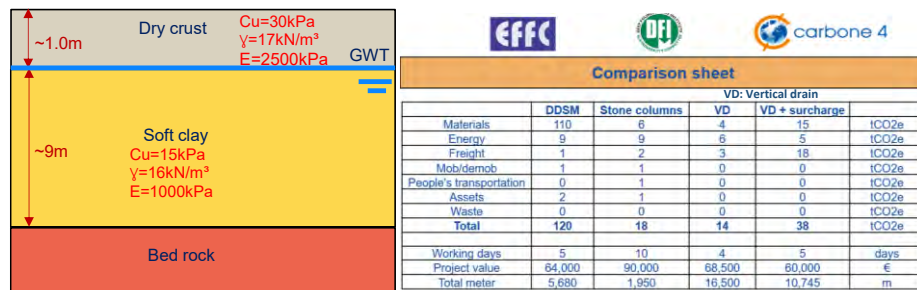


Figure 4. Soil profile and comparison table for all the methods analyzed.

As it can be seen from the comparison charts below (see Fig. 5), emission factor of DDSM is found 85% - 90% higher than stone column and vertical drains.

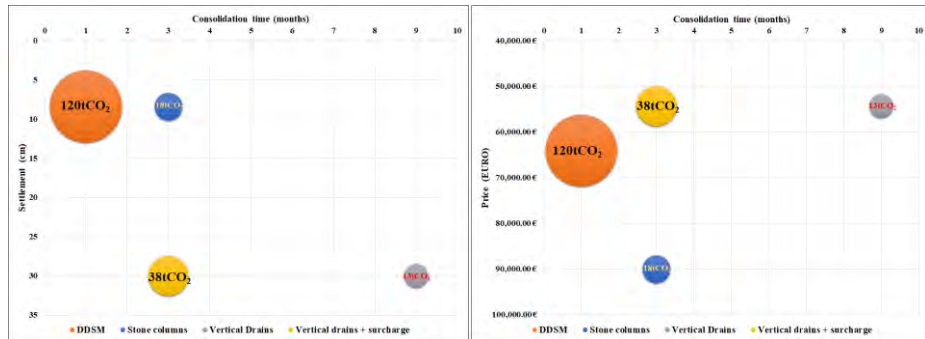


Figure 5. Comparison of all included methods with respect to settlement, price vs consolidation time and equivalent CO₂ emission.

2nd example:

An industrial project that consists of 2m-10m of sulphide bearing clay soil where 5-10m of embankment is going to be placed. The original solution was with DDSM with a coverage ratio of 100%. Area of improvement is 85,000m². Alternative solution was to execute stone columns with a surcharge load to eliminate the settlements. In Fig. 6 is the typical soil profile and a cross section showing the calculation of settlements without and with stone columns resulting in an improvement factor of ≈ 3 .

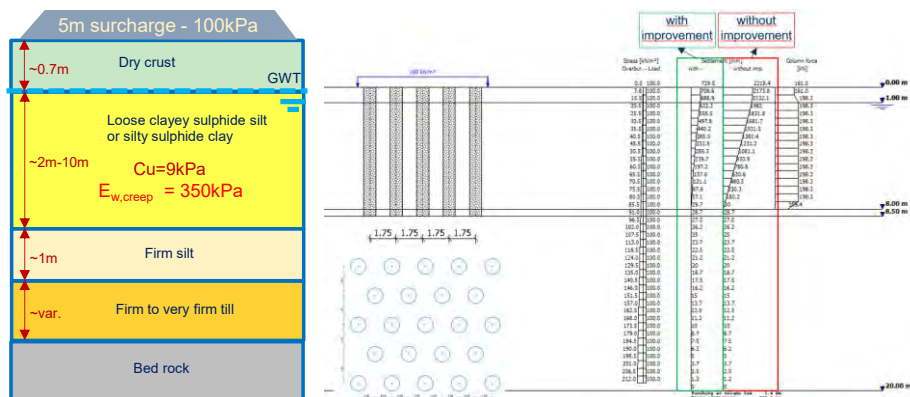


Figure 6. Typical soil profile and calculation of settlement.

EqCO₂ emission calculations for both DDSM solution and stone column solution are performed by using EFFC DFI Carbon Calculator Tool v4.0 and the results are shown below. As it can be seen clearly, with stone column solution the carbon footprint can be reduced ca. 90%.

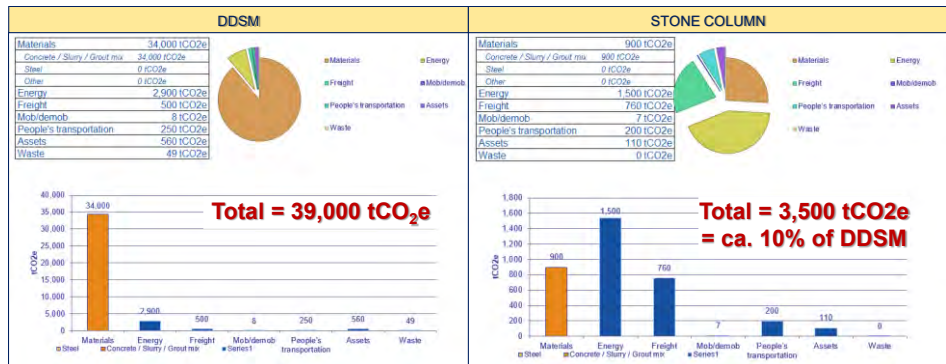


Figure 7. EqCO₂ calculation comparison for the same improvement area with DDSM and Stone columns. .

CONCLUSIONS

In conclusion, the Nordic countries of Sweden and Norway are basically aware of the need to get more sustainable solutions into the geotechnical projects. Soil improvement techniques play a vital role in addressing these challenges, ensuring the stability and longevity of infrastructure projects in urban areas and beyond. While methods like stone columns offer effective solutions, their application encounters limitations in certain soil conditions, necessitating ongoing research and innovation. Initiatives such as the VIBE project in Norway exemplify efforts to pioneer sustainable ground improvement solutions, incorporating recycled materials and advanced techniques to mitigate environmental impacts. Moreover, the adoption of tools like the EFFC calculator facilitates the evaluation and comparison of the environmental footprint of different soil improvement methods, guiding decision-making towards more sustainable practices.

ACKNOWLEDGEMENT

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