Verification of Field Settlement of In-situ Stabilized Dredged Sediments Using Cone Penetration Test Data

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ABSTRACT

Utilization of in-situ mass stabilization for geotechnical applications is increasing. Laboratory tests may have drawbacks on valuations of engineering parameters for estimations of settlement of in-situ stabilized soil mass. Factors such as compression, mixing work, homogeneity and curing temperature may influence the differences in mechanical properties between laboratory test results and achieved field values. Therefore, utilization of appropriate in-situ mechanical parameters may be required during design analyses. Various in-situ tests are available for use in geotechnical context. Among others, cone penetration test (CPT) is one of most widely used in-situ tests. Numerous CPT empirical correlations are available for use in conventional soils. Utilization of such CPT empirical correlations for in-situ stabilized soils has to be examined. In this paper, the in-situ constrained modulus was evaluated using conventional CPT empirical correlation and utilized as oedometer modulus in finite element analysis for estimation of settlement of preloaded in-situ stabilized dredged sediments. The results show that, computed settlement values fall within the range of measured one. These findings suggest that, the cone penetration test and its empirical correlations, which were established for conventional soils, can also be utilized in stabilized soils.

KEYWORDS: field settlement, cone penetration test, in-situ constrained modulus, finite element method, PLAXIS, in-situ stabilization, contaminated dredged sediments, preloading

INTRODUCTION

Millions cubic meters of sediments will be dredged in Baltic Sea in the near future. About 4,000,000 cubic meters of sediments will be dredged at the Port of Gävle alone during Port expansion project. It is estimated that out of these 4,000,000, about 1,000,000 cubic meter is highly contaminated (Fossenstrand, 2009; SMOCS, 2010). Handling of these harmful sediments requires proper attention for sustainable development. In Sweden, Sea disposal of contaminated dredged sediments is banned. The Swedish environmental protection agency states, "Wastes are forbidden to be dumped at the Sea. Waiver is given where waste can be dumped without detrimental effects to human health and environment" (Swedish EPA, 2001). Therefore, sustainable management of contaminated dredged materials is motivated to beneficial reuse. The management of Port of Gävle identified stabilized contaminated dredged materials (SCDM) as alternative material to natural construction materials for structural backfill. Consequently, the Port of Gävle decided to consider the utilization of the SCDM as a base for new land, thus, reducing the use of natural resources and saving money compared to other handling alternatives of the contaminated dredged sediments (SMOCS, 2010). Nevertheless, the use of in-situ stabilized contaminated dredged sediments in geotechnical applications requires strong justification with regard to the achieved in-situ stiffness and strength parameters. Accordingly, it is common practice to carry out advance tests in the laboratory to obtain adequate information on stiffness and strength behavior of stabilized material before full-scale project can take over. Even so, in-situ stabilization of large volumes of soil mass may result into different stiffness and strength in contrary to presumed laboratory test results. According to Åhnberg *et al.* (2001), a number of factors such as compression, mixing work, homogeneity, curing temperature and stress situation may influence the differences in mechanical properties between laboratory test results and attained field values and that, the laboratory test results has to be utilized as a base for field operations. It follows that; appropriate in-situ soil mechanical parameters have to be utilized during geotechnical design. In-situ soil tests are of primary interest to geotechnical engineers compared to laboratory tests. The in-situ soil tests provide quick information on the behavior of soil at a particular site. Numerous in-situ soil tests are available, among others; the cone penetration test (CPT) is one of the most widely used in-situ soil tests. The CPT has many advantages over laboratory tests, some of those are:

- Insight soil stratigraphy is obtained.
- Conservative idealization of particular site into homogenous soil is avoided.
- The test is easier to conduct at minimum supervision and workmanship error.
- Instantaneously engineering parameters are obtained.

Several CPT empirical correlations, which were established for conventional soils are available for use. However, not much research has been done to verify if the CPT correlation developed for conventional soils are also applicable to the stabilized soil mass. The aim of this paper is to verify the measured field settlement by finite element analysis (FEA). The FEA utilizes the oedometer modulus, which is evaluated as in-situ constrained modulus using CPT empirical correlations. The CPTs data and measured field settlement values were taken from the large-scale field test at the Port of Gävle in Sweden.

PORT OF GÄVLE

The Port of Gävle, owned by the Municipality of Gävle is one of the ten biggest import and export harbors in Sweden. The Port lies in the inlet of the Gulf of Bothnia, about 200 km north of Stockholm (Figure 1). According to the Swedish meteorological and hydrological institute (http://www.smhi.se/en), Gävle has a similar climate as the rest of central part of Sweden with an average air temperature of -5°C in January and +17°C in July. Thousands of ships call at this Port every year exporting and importing various products to and from northern and central parts of Sweden. Examples of products are coffee, steel, timber and oil. The Port is currently in a phase of expansion to increase its cargo handling capacity by 40%. The increase will boost the Port handling capacity from the current 5 million tons to over 7 million tons per year (Fossenstrand, 2009, SMOCS, 2010). The project phases include dredging of sediments in order to deepen and widen the entrance channel, construction of new environmental quays and expansion of existing container terminals. Ports and harbors are identified as 'hot spots' for contaminated sediments. Because of environmental restrictions, the contaminated dredged sediments at the Port of Gävle will be amended by a stabilization solidification technology. The amended materials will be utilized as structural backfill in acquiring new land from the Baltic Sea. Investigations on strength-deformation characteristics of the stabilized dredged sediments were carried out in laboratory to found out the desired design recipe for structural backfill. A large-scale field test was carried out at Granudden terminal (Figure 1) to verify laboratory test results for in-situ stabilized dredged sediments.

LARGE SCALE FIELD TEST

The stiffness and strength parameters measured in laboratory may differ from the one achieved in the field, not only because of compression due to preloading but also due to curing temperature and stress distribution. Therefore, laboratory tests can only be used as a base for assessments of appropriate type of binder and the mixing ratio (Åhnberg et al, 2001). Thus, in order to find out the in-situ strength-deformation characteristics of the SCDM fill, a large-scale field test was carried out. A basin of 30 m x 30 m was obtained from the Baltic Sea at Granudden terminal. The basin comprised of natural ground to the south, a quay wall to the west side, sheet pile wall to the north side (Baltic Sea side) and to the east is a separation wall of plastic curtail (Figure 5). Freshly contaminated dredged sediments at average water content of 450% were mixed with binders using the on-site pugmill. The mix proportional of 180 kg of binders (cement: 35%, fly ash: 35% and Merit®5000: 30% by weight) per cubic meter of freshly dredged sediments were applied. The mix was then discharged under water in the reclaimed area (Figure 2). From the laboratory test results, it was predetermined that, a minimum unconfined compressive strength (UCS) of 140 kPa and a permeability of less than 10^{-9} m/s would be achieved within 91 days (d) of curing (Fossenstrand, 2009).

Backfilling of the reclaimed area began on October 29, 2010 and continued through January 23, 2011, when the final layer of the SCDM was placed in the reclaimed area. The total SCDM of about 9,000 cubic meters were utilized in the large-scale field test. The curing period was counted from the last day of backfill (i.e. January 23, 2011); the slurry form of SCDM fill prevented an immediate application of preloading weight. Thus, the application of first phase of preloading weight was done on February 6, 2011. The settlement monitoring systems on the surface of SCDM fill and environmental monitoring mechanism such as borehole for collection of leachate were instrumented prior to preloading. The fill area was preloaded with selected gravel material (Figure 3). The first phase of preloading involved the application of 18 kPa, followed by additional load of 37 kPa after 30 days of curing under the first phase of preloading weight. The field settlements of SCDM were continuously recorded at F1, F2, F3 and F4 starting from February 6, 2011 (Figure 3 and Figure 5). Figure 4 presents measured field settlements from February 6, 2011 to June 1, 2011 in time scale. No significant additional settlements were recorded after this period, for the two phases of preloading. CPTs were done at different locations within the field after 28d (P1, P2, P3, P4, P5 and P6), 91d (P11-01, P11-02, P21-01 and P21-02) and 150d (P11-03, P11-04, P21-03 and P21-04) of curing. Detailed CPT locations are shown in Figure 5.

Figure 1: Geographical location of the Gävle Municipality, Port of Gävle and Granudden terminal (Photo: Courtesy of Port of Gävle; map: http://www.umsl.edu/services/govdocs/wofact2001/geos/sw.html)

Figure 2: On-site pugmill mixing freshly dredged materials with binders (right); the mix was discharged underwater into a reclaimed area (left) (Photo: Courtesy of Port of Gävle)

Figure 3: Monitoring instruments and preloading layer of selected gravel on finished SCDM fill (Photo: Courtesy of Port of Gävle)

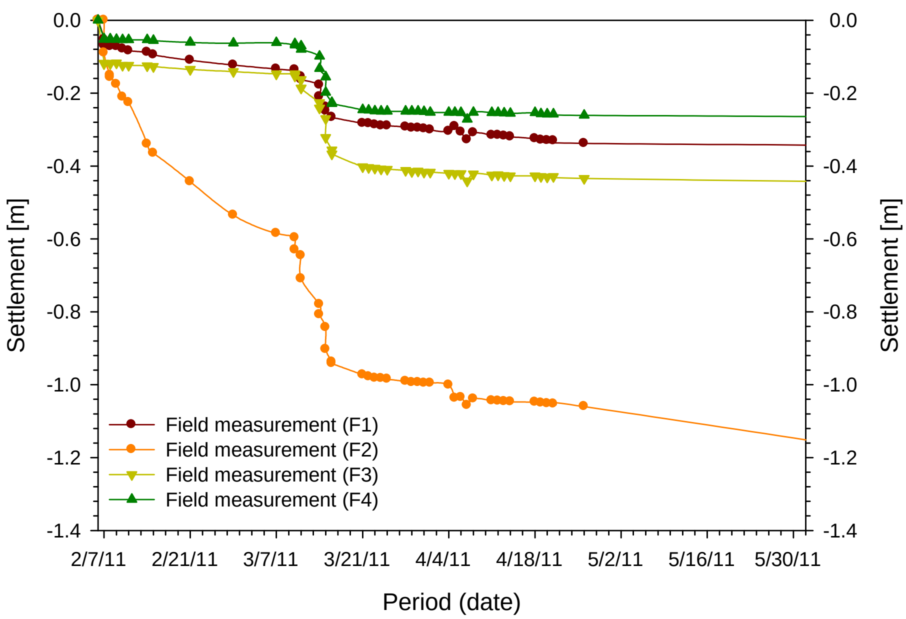

Figure 4: Progressive settlement measurement
(Data: Courtesy of Göran Holm at SGI, WSP and PEAB)

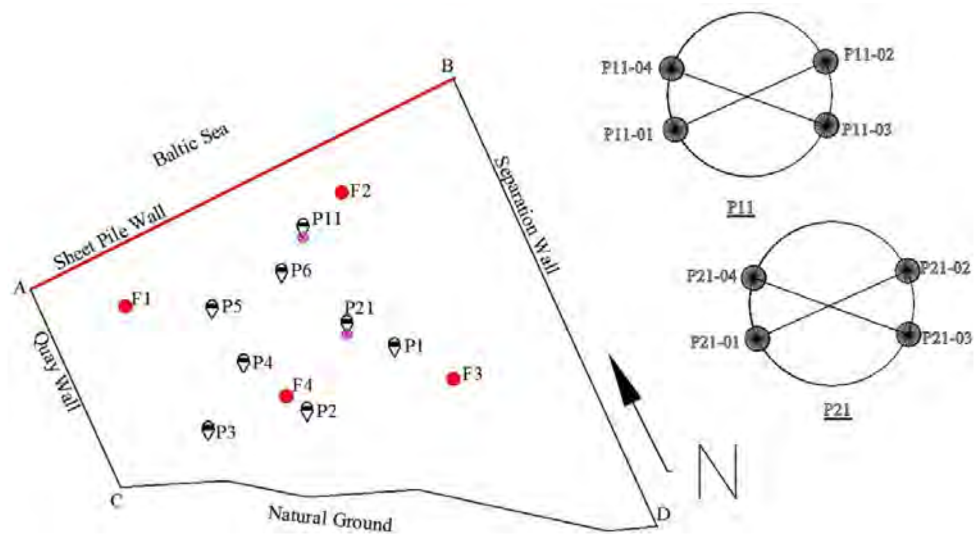

Figure 5: Plan view of detailed CPT locations in relation to settlement monitoring points (Data: Courtesy of WSP and PEAB)

SETTLEMENT ANALYSIS

The verification of field settlements involved simulation of field backfill operations in the finite element method (FEM). The finite element software 'PLAXIS 2D' (Brinkgreve *et al.*, 2010) was utilized in one-dimensional consolidation settlement analyses for the SCDM fill. During the computations, plane strain conditions were assumed and 15 nodes triangular elements were used in a fine mesh with refined SCDM clusters, which gave enough accuracy in reasonable time. The analyses utilized the Mohr-Coulomb (MC) material model under undrained condition case (B) (Brinkgreve *et al.*, 2010). This constitutive model assumes the material isotropic, homogeneous and elastic perfectly plastic, obeying the MC failure criteria. The MC model requires two stiffness parameters and one strength parameter. The primary stiffness parameters include; effective Young's modulus, E' and effective Poisson's ratio, ν' while, the secondary (alternative) stiffness parameters include oedometer modulus, E_{oed} and shear modulus, G. These stiffness parameters relates to each other by

$$E_{oed} = \frac{E'(1-\nu')}{(1+\nu')(1-2\nu')} \tag{1}$$

$$G = \frac{E'}{2(1+\nu')} = \frac{E_u}{2(1+\nu_u)} \tag{2}$$

where E_u and ν_u are the undrained Young's modulus and Poisson's ratio respectively. The shear modulus is independent of drainage conditions (i.e. the undrained shear modulus is equal to the effective shear modulus) and therefore, the effective and undrained Young's moduli are not the same (Muir Wood, 1990). The undrained Poisson's ratio for saturated clay soil normally ranges between 0.4 and 0.5 (Chua and Tenison, 2003; Muir Wood, 1990). The SCDM fill layers utilized the value of ν_u of 0.4 for all layers and all CPT points. PLAXIS also recommends this value of ν_u in MC model under undrained condition case (B). For the SCDM layers the undrained shear strength was utilized as the only strength parameter in MC model under undrained condition case (B).

In-situ constrained modulus

In terms of CPT data, the relationship between the in-situ constrained modulus and measured CPT quantities has been empirically established (Robertson, 2009; Larsson and Mulabdic, 1991; Lunne *et al.*, 1997). The correlation is presented as

$$M = \alpha_M (q_t - \sigma_{v0}) \tag{3}$$

where q_t is the corrected cone resistance, σ_{v0} is the total vertical stress and α_M is the constrained modulus cone factor. Robertson (2009) suggested the following correlations for selection of the constrained modulus cone factor

If $I_c \geq 2$

$$\alpha_M = Q_{t1} \qquad when\ Q_{t1} \leq 14 \tag{4}$$

$$\alpha_M = 14 \qquad when\ Q_{t1} > 14 \tag{5}$$

If $I_c < 2$

$$\alpha_M = 0.03\left(10^{0.55 I_c + 1.68}\right) \tag{6}$$

where Q_{t1} is the normalized cone resistance for clay soil and I_c is the soil behavior type index. The normalized cone resistance, Q_{t1} for clay is given by

$$Q_{t1} = \left(\frac{q_t - \sigma_{v0}}{\sigma'_{v0}}\right) \tag{7}$$

where σ'_{v0} is the effective vertical stress whereas the soil behavior type index, I_c can be calculated from

$$I_c = \left[(3.47 - \log Q_{t1})^2 + (\log F_r + 1.22)^2\right]^{0.5} \tag{8}$$

where F_r is the normalized friction ratio. The calculated I_c of the SCDM from the Port of Gävle had values between 2.8 and 4.7, with an average value of 3.6. These values classify the SCDM from Port of Gävle as normally consolidated soil in accordance with Robertson (2009) chart.

Undrained shear strength

The in-situ undrained shear strength, S_u can be estimated from CPT data (Robertson, 2009; Robertson *et al.*, 1986; Abu-Farsakh *et al.*, 2003; Houlsby and Teh, 1991, Kulhway and Mayne, 1990) by the following correlation

$$S_u = \frac{q_t - \sigma_{v0}}{N_k} \tag{9}$$

where N_k is the cone tip factor. Lunne *et al.* (1986) reported the N_k values for Scandinavian clays in the range of 15 to 21. Robertson (2009) suggested the average N_k value of 14 for insensitive fine-grained soils. The SCDM from the Port of Gävle had an average sensitivity value of 1, which according to Robertson (2009) can be regarded as insensitive NC clay. Thus, the N_k value of 14 was adopted in equation (9) for the SCDM fill from Port of Gävle.

Computations

The evaluated input parameters of the SCDM fill showed significant variations in stiffness and strength parameters along the entire depth and across the field. Thus, depending on the observed variations in measured CPT quantities, the SCDM fill was subdivided into layers of thickness between 0.23 and 0.30 m. These thicknesses were considered in close interval with CPT sampling and sufficient to capture the variations of stiffness along the entire depth. **Table 1** presents an example of the input parameters E_{oed} and S_u for SCDM fill layers based on CPT data at point P3 only. All layers at this CPT point were assigned an average unit weight of 13.85 kN/m^3 and the hydraulic conductivity of 5.5 x 10^{-7} m/s in vertical direction.

PLAXIS input parameters in MC model involved the use of secondary stiffness parameters, where oedometer modulus evaluated as in-situ constrained modulus in equation (3) and an assumed undrained Poisson's ratio of 0.4 were utilized. The plastic calculation type was used during layer placement and preloading. After completion of backfill operations, preloading weight was placed and all initial displacements were reset to zero. This was done in order to mimic field operation, where recording of the field settlements stated after preloading the SCDM fill. Since it is customary to preload the stabilized mass in phases, PLAXIS consolidation analyses for the SCDM fill were carried out in two phases of preloading.

In the first phase, the SCDM fill was preloaded with 18 kPa. However, due to partial submergence into the ground water of the placed gravels, which resulted into reduced effective vertical stress, the average effective preloading weight of 10 kPa was utilized during simulations. Consolidation analyses followed for the period of 30 days.

The second phase of preloading started after 30 days, with an application of an average additional effective load of 37 kPa, followed by consolidation analysis to minimum excess pore pressure (EPP). Note that, due to differences in maximum depths of the SCDM fill (**Table 2**), the consolidation period to minimum EPP varied between CPT points. Simulations of the field settlements involved the use of stiffness evaluated after 28, 91 and 150 days of curing. For simplicity in PLAXIS 2D simulations, the evaluated stiffness and strength parameters were assumed to be homogenous across the CPT section of the SCDM fill; therefore, the SCDM fill were modeled with maximum depth of the fill at a particular CPT point.

An example of computation is presented for CPT data from point P3 for which the curing time was 28d. Figure 6 presents the numerical model showing boundary conditions. The Anchor rod shown is for illustration purpose only; the length of the anchor rod is equal to the width of the model in the actual simulations. Figure 7 displays one-dimensional deformed mesh at the end of consolidation and, Figure 8 shows the maximum vertical deformation of 0.20 m of the SCDM fill for a settlement analysis performed on the section at CPT point P3. Similar computations were carried out for the other CPT points.

Table 1: Input parameters for PLAXIS simulation of SCDM fill at CPT point P3

Layer ID	1	2	3	4	5	6	7	8	9	10	11	12	13	14
E_{oed} (kN/m^2)	849	2387	1083	1189	2924	1493	4376	6603	2728	1545	1250	383	589	407
S_u (kN/m^2)	16	27	18	19	29	21	35	43	27	20	18	10	12	10

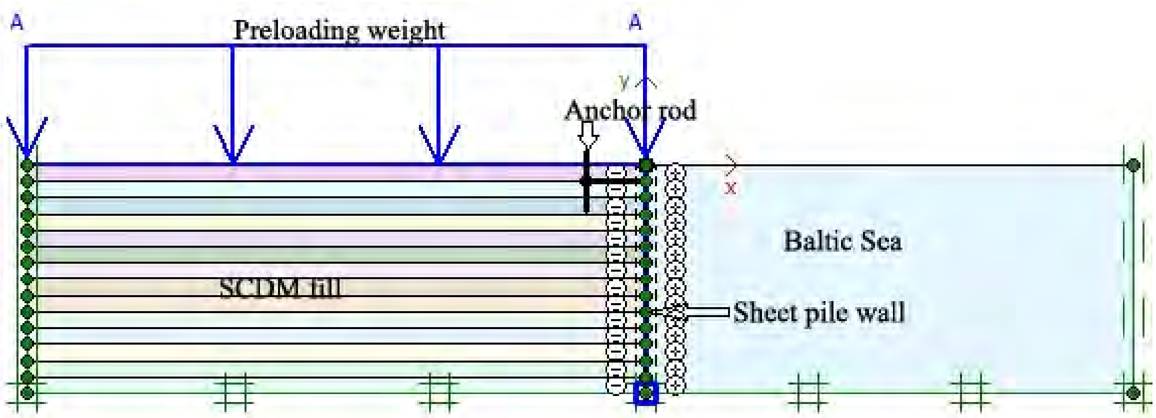

Figure 6: Section model showing boundary conditions for the SCDM fill (based on the CPT data at P3)

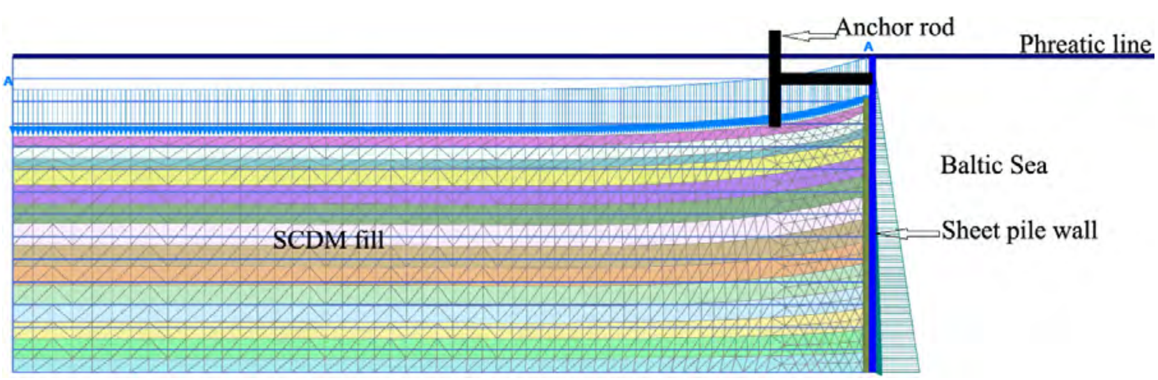

Figure 7: Section model showing exaggerated (scaled up five times) deformed mesh at the end of consolidation of the SCDM fill (based on the CPT data at P3)

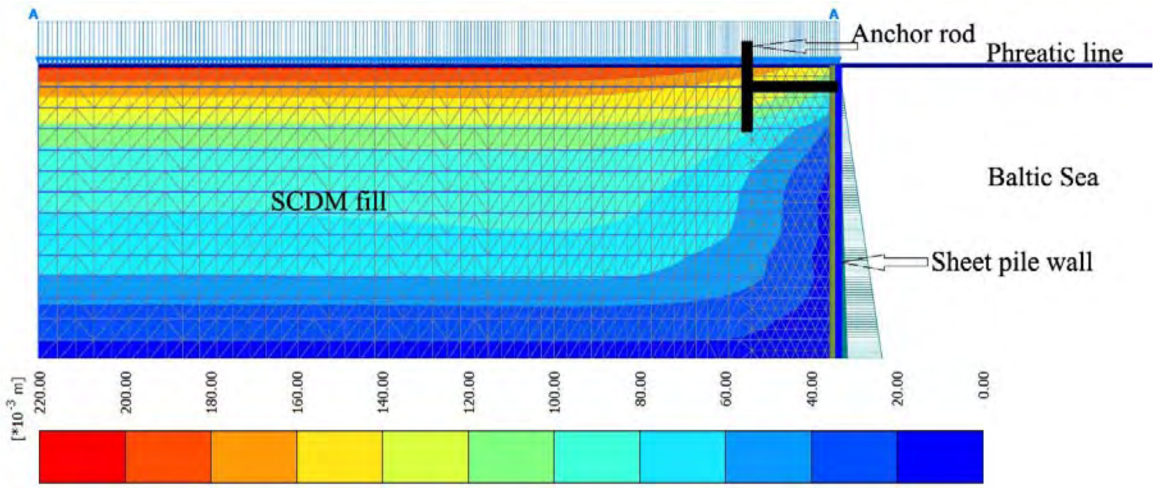

Figure 8: Section model showing the maximum vertical deformation of 0.21 m in the SCDM fill (Based on CPT data at P3)

RESULTS

Verification against field settlement

Figure 4 presents the measured field settlement values from Port of Gävle in time scale. In order to validate the applicability of the evaluated oedometer modulus for estimation of settlement, the computed settlement values were compared with measured settlement values by cumulative settlement curves (Figure 9 to Figure 12). Note that, due to the lack of CPT repetitive data, spread of CPT points and variations in depths of the SCDM fill (**Table 2**), comparisons of the measured settlement values against computed settlement values were not restricted to the CPT points, which were located nearby the monitoring points. Thus, the comparisons aimed at finding out the evaluated stiffness and curing period corresponding to the field stiffness, which resulted into the measured settlement values during the two phases of preloading. Results show that, measured settlement values at F1 corresponded well with computed settlement values from CPT point P21-03 during the first phase and CPT points P1 and P11-01 during the second phase of preloading (Figure 9). The measured settlement values at F2 correlated with the computed settlement values from CPT point P21-01 (Figure 10). The measured settlement values at F3 matched quite well with computed settlement values from CPT points P1 and P11-01 during the first phase and CPT points P4 and P11-02 during the second phase of preloading (Figure 11). While, the measured settlement values at F4 showed comparable correlations with the computed settlement values from CPT point P5 during the first phase, and CPT points P2 and P21-03 during the second phase of preloading (Figure 12).

Table 2: The magnitude of computed final settlement values for all CPT data

CPT Point	Maximum Depth (m)	Curing period (days, d)	Settlement (ν_u = 0.4) (m)
P1	7.25	28	-0.35
P2	6.09	28	-0.24
P3	4.06	28	-0.20
P4	8.12	28	-0.42
P5	8.70	28	-0.12
P6	8.12	28	-0.88
P11-01	6.44	91	-0.33
P11-02	6.21	91	-0.43
P11-03	6.44	150	-0.45
P11-04	6.90	150	-0.32
P21-01	6.44	91	-1.09
P21-02	7.72	91	-1.64
P21-03	7.25	150	-0.25
P21-04	7.44	150	-0.88

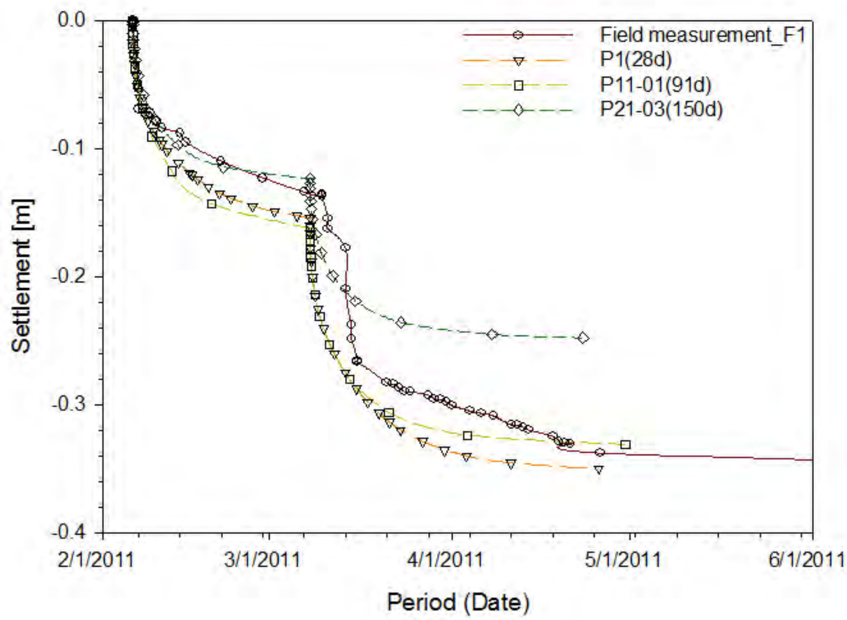

Figure 9: Comparison between measured values at F1 and computed settlement values from CPT point P1 (28d), P11-01 (91d) and P21-03 (150d) versus time

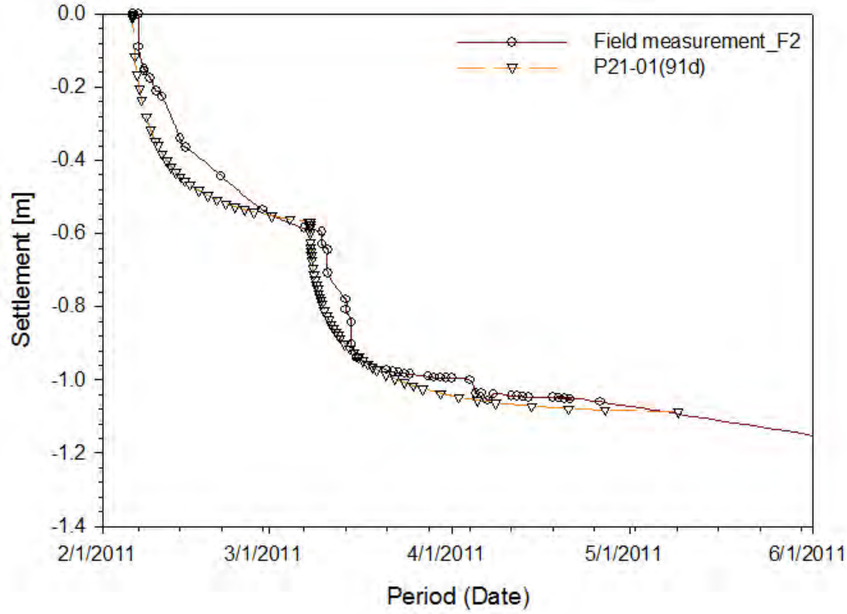

Figure 10: Comparison between measured values at F2 and computed settlement values from CPT point P21-01 (91d) versus time

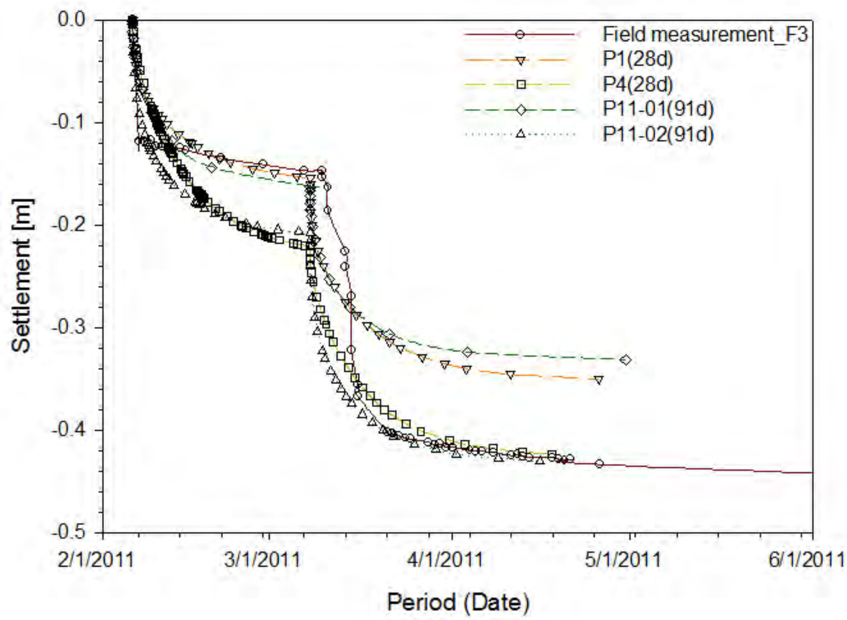

Figure 11: Comparison between measured values at F3 and computed settlement values from CPT point P1 (28d), P4 (28d), P11-01 (91d) and P11-02 (91d) versus time

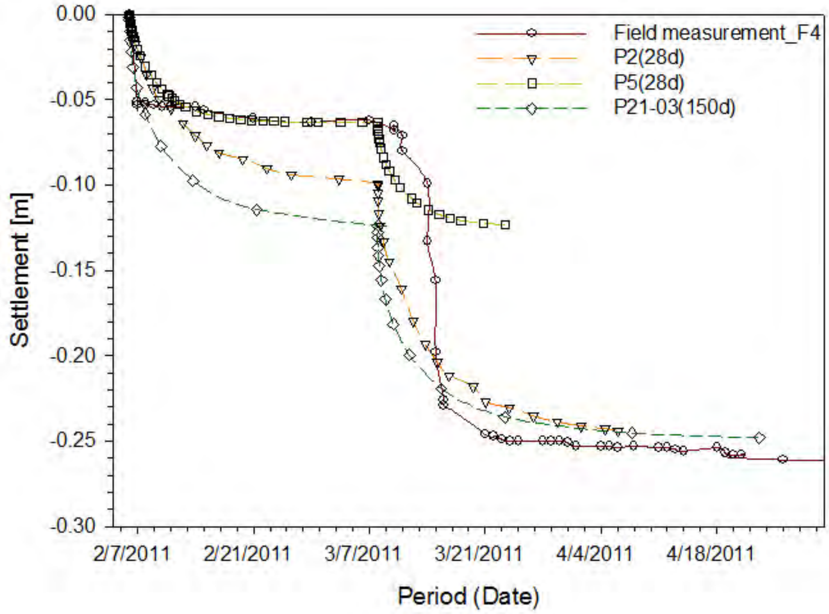

Figure 12: Comparison between measured values at F4 and computed settlement values from CPT point P2 (28d), P5 (28d) and P21-03 (150d) versus time

DISCUSSION

The field settlements of stabilized dredged material due to preloading weight were analyzed using finite element software, PLAXIS 2D. The computed settlement values suggests that, most of measured settlement values were due to stiffness evaluated following 28 days of curing under the first phase or second phase of preloading weight. With exception to Figure 10, computed settlement utilizing the SCDM stiffness obtained after 91 or 150 days of curing resulted into decreased or same magnitudes of settlement as computed magnitudes of settlement using the SCDM stiffness evaluated following 28 days of curing. Both field and computed settlement showed inconsistency and variations in magnitudes of settlement within the same field under the same preloading weight. These variations suggest that, the SCDM fill exhibited different stiffness and strength parameters within a short distance across the field and along the entire depth of the fill. The authors postulate uneven distribution of preloading weight and curing air temperature as contributing factors for the absence of CPT repetitive data and discrepancies in both measured and computed settlement values.

Effect of preloading weight

Preloading weight applied on stabilized soils helps to remove air pockets and bring soil particles closer together for effective hydration reaction, which results into homogenous stabilized mass and increased vertical stresses (Åhnberg, 2007). During stabilization solidification process at the Port of Gävle, the first stabilized layer was placed approximately 90 days prior to the final layer. Because of three months interval of placement, layers placed earlier had probably gained enough strength (and presumably with large volume of locked water) to resist further compression. Thus, preload weight applied after placement of the final layer had little or no compression effect on the first layers. In addition to that, at a certain period of curing, water may become lodged in between solid particles by cementation effect. This may result into pore blockage, preventing inflow or out flow of water; this may subsequently impede any development or dissipation of excess pore pressure (EPP) under applied load. The essence of EPP is to receive and gradually transfer the load into soil skeleton as water dissipates, giving enough time for soil particles to smoothly reorient and compact uniformly. In the absence or presence of disproportional EPP, the preloading weight may induce uneven jagged stress causing collapse of water voids. Consequently, layers with larger water voids become prone to compression on application of preloading weight. Thus, uneven effective vertical stress from preloading weight across the field, which ranged from 39kN/m^2 to 49kN/m^2 as evaluated from CPT data contributed to variations in stiffness across the field. For instance, compared to measured field settlement values, the computed settlement values from CPT points P1 and P11-01 (Figure 9), CPT points P4 and P11-02 (Figure 11) and CPT points P2 and P21-03 (Figure 12) suggest that, at these locations the SCDM fill was initially preloaded with weight lower than the presumed effective value of 10 kPa. According to Åhnberg *et al.* (2001), in order to avoid uneven quality in different sections of a stabilized area, it is important to apply preloading weight in a uniform manner, that is, the same height of the fill should be placed at the same time after stabilization in the entire working area.

Effect of curing temperature

The hydration process occurs immediately upon mixing cement with soil and other additives in the presence of water. The reaction results into hardened and aggregated soil particles (EuroSoilStab, 2002). This process is slow, proceeding from the surface of cement grains and the center may remain unhydrated. Furthermore, at low temperature (below 4°C) the process can be impaired considerably (Sherwood, 1993; Maher *et al.*, 2004). Therefore, the hydration process requires sufficient moisture at a favorable temperature to proceed. Underwater discharge assured sufficient moisture at minimum variation of temperature. However, the mixing and placement of SCDM at the Port of Gävle commenced in the end of October 2010 through the end of January 2011, during which, the average air temperature in Gävle may reach -5°C. Thus, the entire fill operation was carried out during unfavorable temperature for effective hydration reactions to proceed. This contributed to irregularity and variations in stiffness and strength along the entire depth of the SCDM fill. The CPT data revealed very soft layers at the bottom of the SCDM fill at CPT points P6 and P21. These layers were prone to compression upon application of preloading weight, this explains the reason for higher computed and measured magnitudes of settlement (Figure 10 and **Table 2**).

CONCLUSIONS

Verification of one-dimensional field settlement of stabilized contaminated sediments in finite element method (FEM) was presented. The finite element analysis in PLAXIS 2D software, utilized in-situ constrained modulus evaluated from CPT empirical correlations. The computed settlement values gave quite comparable results with measured ones. These findings conclude that, CPT data and CPT empirical correlations established for conventional soils can be utilized in stabilized soils as well.

With regard to trend and magnitude of settlement of the SCDM fill from the large-scale field test at the Port of Gävle, the following conclusions are drawn:

- Repeatability of CPT data in stabilized material was unlikely; thus, variations in computed magnitudes of settlement within the same field were inevitable.
- The representative stiffness for settlement computations of SCDM fill in structural backfill was achieved during 28 days of curing. However, several CPTs are required to give suitable representative of in-situ mechanical parameters in the SCDM fill.
- The mechanical parameters evaluated from the CPT data before the end of hydration reaction should be regarded as valid at that particular time and location of CPT point.
- As a result of mixing and placing the materials at very low temperature (below 4°C), inadequate hydration reactions occurred, this caused variations in stiffness and strength properties between layers along the entire depth.

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