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VERTICAL DRAINS IN MARINE GYTTJA. DEVELOPMENT OF SETTLEMENTS DRAINS VERTICAUX EN GYTTJA MARINE. DEVELOPPEMENT DES TASSEMENTS

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SYNOPSIS: Organic matters such as gyttja are characterized by low strength properties and deformation parameters. This leads to stability and deformation problems in connection with construction of embankments. The low permeability reduces the rate of settlements and the rate of increase in shear strength. After primary consolidation and elimination of excess pore water pressure secondary settlements (creep) are an important matter in connection with road and railway construction. Essential information is gathered during preloading of a 3 m to 15 m deep marine gyttja layer situated in a 75 m wide valley in stiff glacial deposits. Drainage is accelerated through vertical drains. Settlements and slope deformations are followed in detail during preloading with initially 8.5 m sand fill followed by additionally 4.5 m fill after 8 months of consolidation and totally follow up of the deformations over a period of 4 years. Initial information after partial unloading is collected.

INTRODUCTION

A/S Storebæltsforbindelsen, SBF (The Great Belt Link, Ltd.) constructs a road and railroad embankment on gyttja deposits as part of the approaches to the 18 km bridge and tunnel crossing at Great Belt. Construction of road and railway embankments on weak soils are faced with stability and settlements problems. Low permeability results in long term primary consolidation and correspondingly slow increase of undrained strength. When replacement with better fill material is impossible due to depth or cost, preconsolidation is at hand, but still time consuming and with stability problems along slopes. General experience shows that vertical drains installed in inorganic clays and silts are a suitable method for increased consolidation and strength increase. Less information for the use in organic matters are at hand. For organic matters such as marine gyttja prediction of settlements are possible based on traditional soil parameters, but are also dependent on preconsolidation in the deposits. Main problems are time for development of primary consolidation (vertical and horizontal permeability), the rate of secondary consolidation (creep) and the effect on the creep rate after unloading of overburden. The reliability of the function of vertical drains during a period of several years without clogging or damage during heavy deformations (settlements, rupture) is important. Full scale experience has been collected during consolidation of a 15 m deep marine gyttja deposit under 13 m of sandfill preload over a period of 4 years successfully conducted by SBF and under detailed supervision in the field by the Danish State Railways, DSB.

SOIL CONDITIONS

A reclaimed area close to the sea (mean level 0 m) consists of a top layer of marine gyttja between the soil surface in level -1 m and underlying stiff late glacial or glacial deposits in level -3 m to -4 m. In a valley with a width of 75 m in the glacial deposits the thickness of gyttja increases to 15 m. The variation of the late glacial surface is shown on Figure 1 and a cross section on Figure 2. Based on geotechnical investigations in the field and laboratory a characteristic profile of the gyttja strata is shown on Figure 3. With a water content, $w \sim 120\%$ the deformation properties during normal consolidation ($\sigma_1' > \sigma_{pc}'$) is based on a consolidation ratio $Q = 0.3$ determined on general Danish experience (consolidation index $C_c = (1+e) Q$. Code of Practice, DS415). The evaluation of the undrained shear strength, c_u is based on in situ

vane shear tests, c_u . Mean values equal to $c_u \sim 15 \text{ kN/m}^2$ at ground level increasing to $c_u \sim 50 \text{ kN/m}^2$ at a depth of 15 m are recorded. Based on a unit weight of the gyttja $\gamma = 13 \text{ kN/m}^3$ ($\gamma' = 3 \text{ kN/m}^3$) the effective vertical

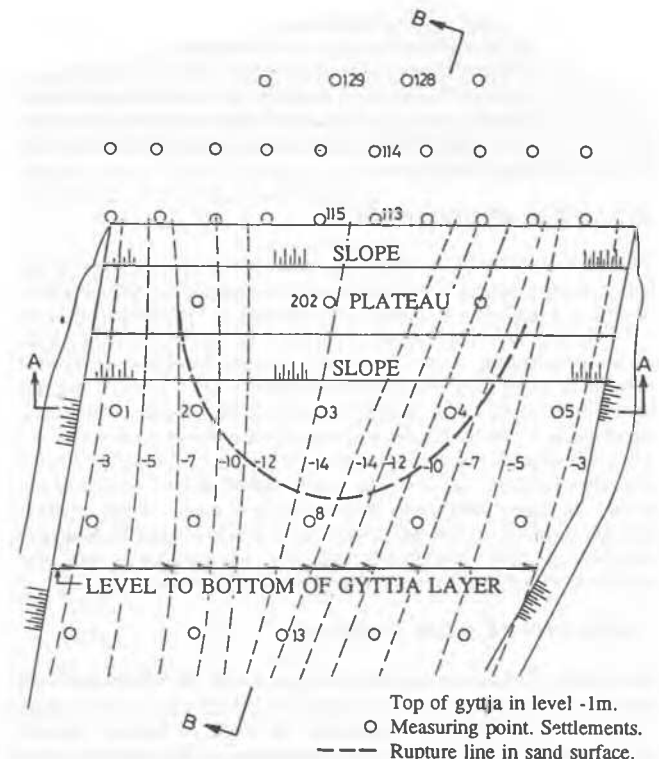


Fig. 1. Location map. Partial area of gyttja valley and embankment. Cross sections A-A and B-B, cf. Fig. 2 and 5.

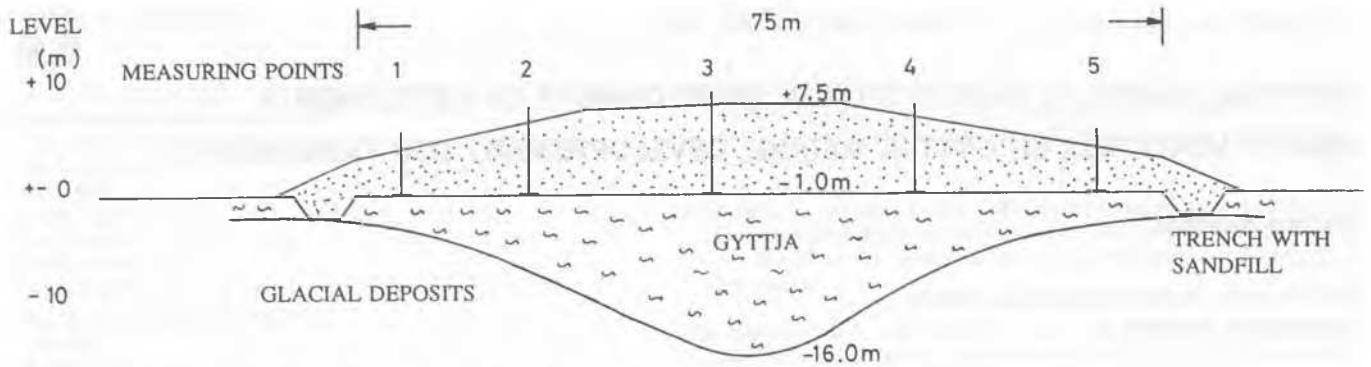


Fig. 2. Cross section A-A, Fig. 1, in gytja valley. Measuring points Nos. 1 to 5 and initial sandfilling.

stress increases from $\sigma'_0 = 0$ to $\sigma'_0 \sim 45 \text{ kN/m}^2$. The ratio of strength increase is:

$$\Delta c_v / \Delta \sigma'_0 \sim 0.8$$

The measured vane shear strength, c_v , exceeds the values for a normally consolidated layer ($c_v \sim 0.8 \sigma'_0$) with a constant value of approximately $\Delta c_v \sim 15 \text{ kN/m}^2$. This difference is referred to as an overconsolidation, $\Delta \sigma'_{pc}$ due to creep (secondary consolidation) and stress variations (changes in ground water level). The overconsolidation in the gytja layer is:

$$\Delta \sigma'_{pc} \sim 15/0.8 \sim 19 \text{ kN/m}^2$$

This leads to preconsolidation stresses in the layer:

$$\sigma'_{pc} \sim \sigma'_0 + \Delta \sigma'_{pc} \quad (1)$$

When vertical stresses in the gytja layer are increased due to weight from embankment fill, reconsolidation equal to a stress increase $\Delta \sigma'_{pc}$ will be followed by a primary consolidation governed by change in excess pore water pressure. After 100% theoretical, primary consolidation, δ_{100} at the time t_{100} further deformation due to secondary consolidation (creep) at low excess pore water pressure will take place with a strain rate ϵ_s per log cycle of time and initial layer thickness, g .

EVALUATION OF SETTLEMENTS

For a rail road embankment with a final mean surface in level +3 m or 4 m above original ground level two options were evaluated. A partial excavation of gytja to a depth of 6 m to level -7 m followed by sandfilling to level +6 or if necessary to level +8 in total 13 m to 15 m sandfill for preconsolidation of the remaining 0 m to 9 m gytja. A direct installation of vertical drains in the gytja layer and a preconsolidation of the 3 m to 15 m gytja layer with 3.5 m to 8.5 m sandfill suggested by the contractor (Boskalis, Oosterwijk B.V., Holland) as an alternative solution (later increased by 4.5 m to a total maximum in the central part of 13 m). An evaluation of the two approaches indicated that the system with vertical drains - if functioning through the whole construction period of 2 or 3 years - would ensure a sufficient degree of consolidation and thus lead to a preconsolidation after excavation to final embankment level. The preconsolidation was also expected to reduce the future secondary consolidation rate.

EVALUATION OF SLOPE STABILITY

The stability problem was located at the slope end of the embankment with a height of 8.5 m sandfill and 15 m gytja. The vertical drain system stopped at the slope edge. Based on experience the Code of Practice (DS415) recommends a reduction factor for determination of the undrained shear strength, c_u based on in situ vane shear strength, c_v :

$$c_u = 0.7 \times c_v \quad (\text{for } I_p \sim 70\%)$$

The safety against failure during the construction is reduced to a partial coefficient $\gamma_c \sim (1.5)^k$ (from $\gamma_c \sim 1.5$). This very small safety margin calls for an in situ deformation control to avoid unacceptable undrained rupture deformations. The increase in undrained shear strength during consolidation together with the removal of the surcharge will ensure the final slope stability. The main part of the embankment fill does not present stability problems and the settlements thus represent one dimensional, vertical consolidation.

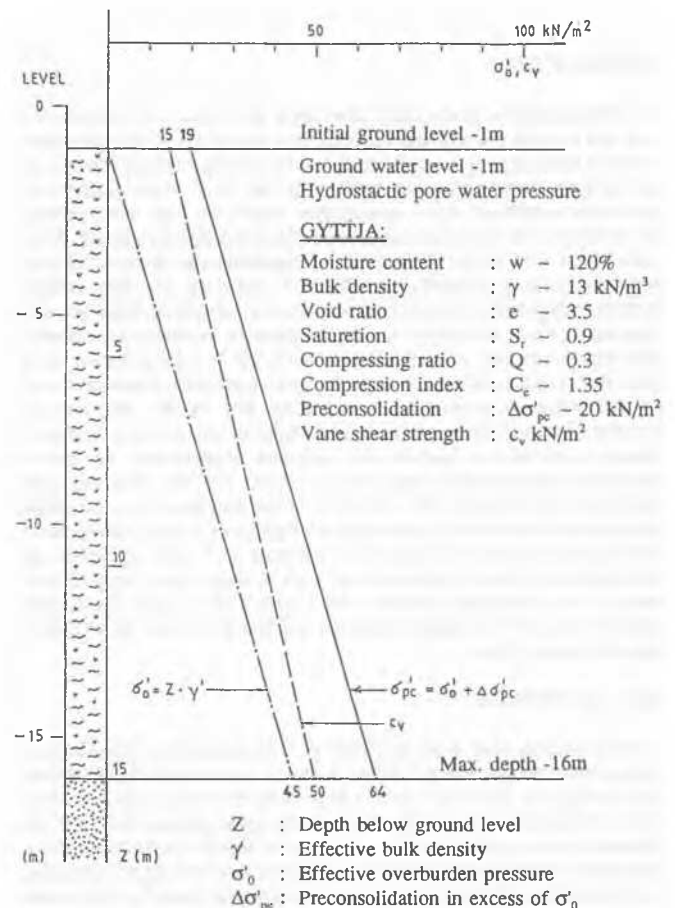


Fig. 3. Initial soil conditions with mean parameter values. Vertical stress and vane shear strength distribution.

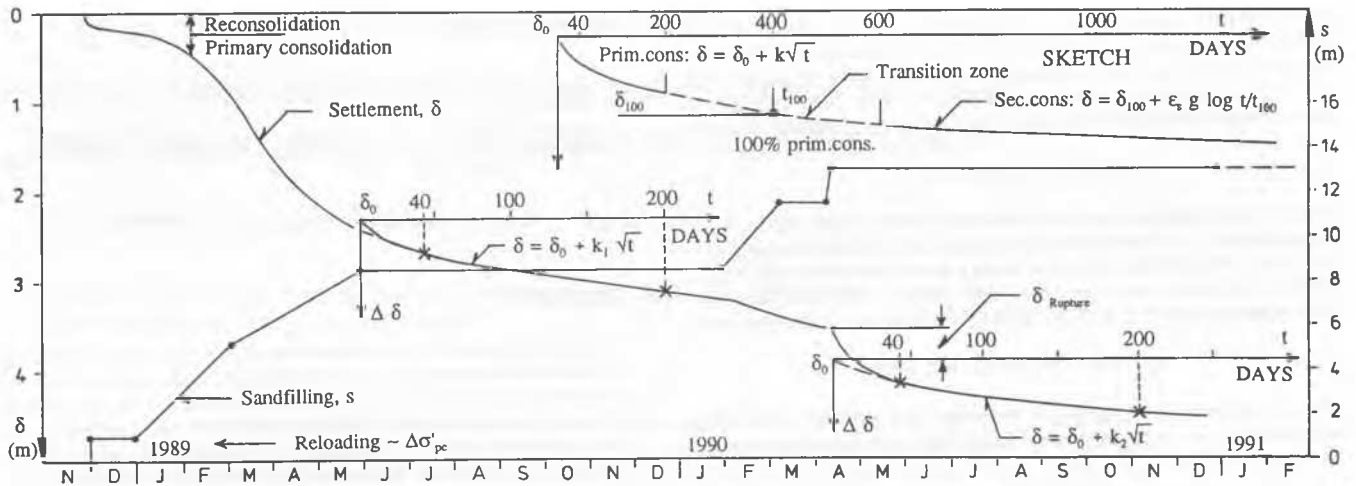


Fig. 4. Development of sandfilling, s and settlements, δ with time. Point No. 3. Sketch showing general development of primary and secondary settlements.

INSTALLATION OF VERTICAL DRAINS

By placing a 1 m sand layer on geotextile directly on top of the gytja the area was prepared for mechanical installation of vertical drains in December 1988. The Membra drain system has a size of 3 mm x 100 mm and consists of a corrugated polypropylene plate covered by non-woven geotextile. The drains are placed through the established sandlayer, the gytja and into the underlying soil (partly a sand layer) secured by a bottom anchor. The drains are placed in a triangular configuration with a distance of 1.5 m and cut 0.3 m above the top of the sand surface. Drainage takes place through the sandfill and the (sand-) layers beneath the gytja.

EMBANKMENT

The establishment of the embankment was started in January 1989 by installation of 0.5 m compacted sand layers with a planned rate of a new layer each week and a maximum thickness of $s \sim 8.5$ m. After the establishment of the embankment (Figure 2) followed by a period of consolidation during 8 months the thickness of sandfill was increased by additional 4.5 m.

DEFORMATION MEASUREMENTS

Before sandfilling 23 reference points were established. 1 x 1 m steel plates were placed on the gytja surface for vertical settlement control on varying gytja thicknesses. Supplementary points were placed in the embankment slope as well as in 3 rows in the gytja surface in front of the slope (Figures 1 and 5) for determination of vertical heave and horizontal movements.

DEVELOPMENT OF SETTLEMENTS

The initial establishment of 1 m sandfill ($\gamma \sim 19 \text{ kN/m}^3$) corresponds to the predicted overconsolidation, $\Delta\sigma'_{pc} \sim 19 \text{ kN/m}^2$, thus representing a reconsolidation. The preliminary settlements develop rapidly during 1 month (Figure 4) and the deformations correspond to an uniaxial modulus of compressibility, $K \sim 2000 \text{ kN/m}^2$. Settlements during the following filling up in steps of 0.5 m sand are registered as a number of loading steps under partially drained conditions. After final filling the settlement/time curve represents a state of normal consolidation initially influenced by the previous filling rate. As indicated on Figure 4, the settlement rate after 40 days from the final filling ($t = 0$) follows the equation:

$$\delta = \delta_0 + \text{constant} \sqrt{t} \quad (2)$$

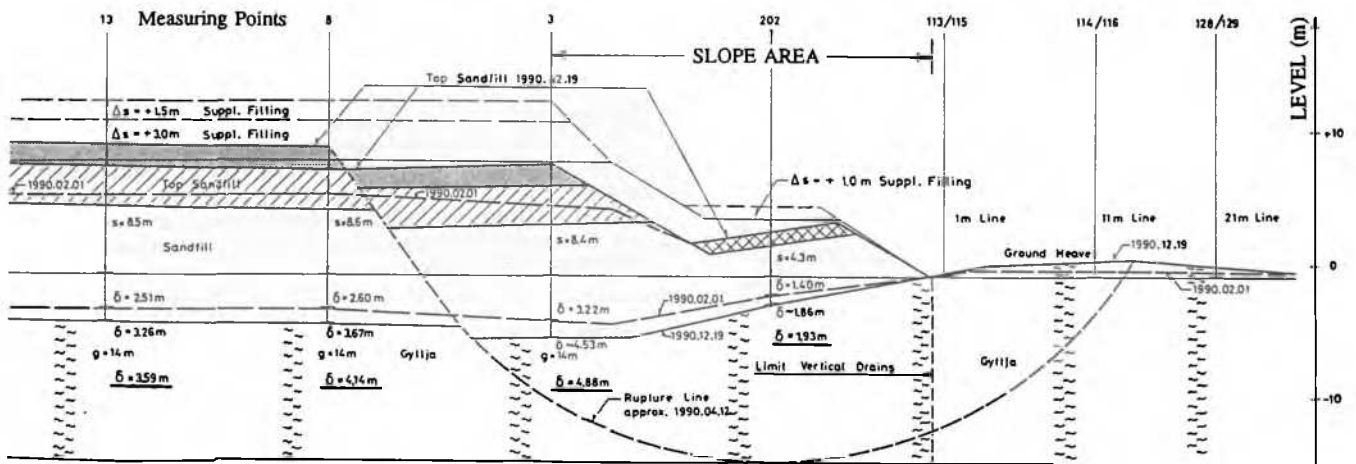


Fig. 5. Longitudinal section B-B, Fig. 1. Sandfilling, Settlements. Heave of gytja surface in front area. Local rupture in slope section. Initial gytja thickness, g . Thickness of sandfill, s . Latest settlements $\delta = x.xx$ m measured 1992-09-25.

The accordance to this deformation rate is very close during a period from 40 to 200 days independent of the thickness of the gyttja layer ($g \sim 5$ m to 15 m) indicating that the consolidation process is governed by the drains (horizontal flow). δ_0 refers to deformations observed during the period from 40 to 200 days and the time $t = 0$. During a period from 200 days to approximately 500 days the deformation rate changes to secondary consolidation:

$$\delta = \delta_{100} + \text{constant} \log \frac{t}{t_{100}} \quad (3)$$

The test results indicate that 100% theoretical, primary consolidation, δ_{100} is obtained after $t_{100} \sim 300$ to 400 days independent of the layer thickness ($g \sim 5$ to 15 m). Observed deformations during a period from 400 to 500 days to 900 to 1,200 days after the latest load change shows that secondary deformation rate in per cent of the initial gyttja thickness g is expected to be:

$$\epsilon_s \sim 3\% \text{ to } 5\% \text{ per log cycle of time}$$

The development of the degree of consolidation, U with time of primary consolidation, t in a cylindrical soil column with a central drain is expressed by Barron (1944):

$$t = \frac{d_1^2}{8 c_{vh}} \left[\ln \frac{d_1}{d_2} - \frac{3}{4} \right] \ln \frac{1}{1-U} \quad (4)$$

The diameter d_1 and d_2 represents computational values for the drained soil column and the drain. Soil column: Area $1.95 \text{ m}^2 \sim \pi (d_1/2)^2$, $d_1 \sim 1.58$ m. Drain surface: $2 \times (0.1 + 0.003) \text{ m}^2/\text{m} \sim \pi d_2$, $d_2 \sim 0.066$ m, Hansbo (1981). With a degree of consolidation $U \sim 97\%$ and $t_{100} \sim 300$ to 400 days this corresponds to a coefficient of horizontal consolidation:

$$c_{vh} \sim 10^{-7} \text{ m}^2/\text{s}$$

As shown on Figures 4 and 5 the embankment height was increased by 4.5 m sandfill after 8 months of consolidation with a maximum of 8.5 m sandfill. The initial 3 m sandfill was placed at a slow rate during 1 month followed by a consolidation period of approximately 1 month. The last 1.5 m fill was placed during 3 days and resulted in a local rupture (Figure 1) in the embankment slope (1990-04-12) as shown in Figures 4 and 5. Apart from an initial greater deformation rate in the local area resulting from the rupture, a primary consolidation process was recorded in the whole area due to the new load conditions. The settlement rate after the 4.5 m surcharge indicates unchanged function of the drain system even in the rupture zone. Corresponding values of measured and calculated settlement are shown on Figure 6. The calculations are based on the parameters in Figure 3. The graph shows close agreement taken in consideration that the deformations in point Nos. 2 to 4 include plastic deformations in the slope area and are influenced by the local rupture (vertical deformation during rupture approximately 0.5 m).

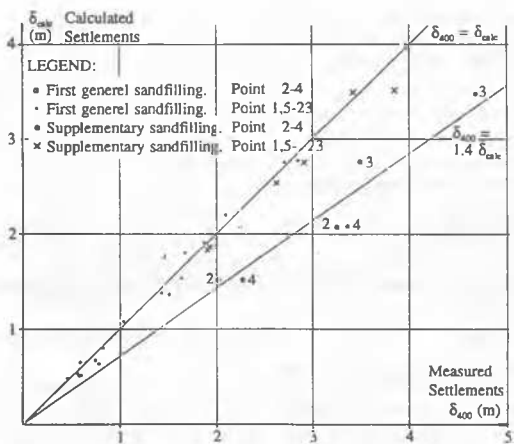


Fig. 6. Correlation between calculated and measured settlements ($t \sim 400$ days) in the whole embankment area. Point Nos. 2 to 4 include plastic and rupture deformations.

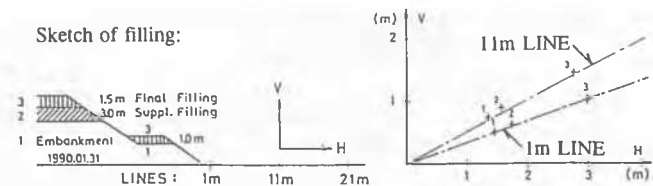


Fig. 7. Surface deformations in front of slope.

STABILITY

The rate of sandfilling was increased from 0.5 m per week in the first part of March 1989, resulting in an increased rate of plastic deformations in the slope area (points 2 to 4, 113 to 115 and 202, Figures 1 and 4) resulting in a surface crack in the sandfill. The maximum thickness of sandfill was 5 m. After a delay of 3 weeks the filling was resumed in the critical area at a lower rate to a maximum thickness of 8.5 m without further problems. After 8 months of consolidation additional sandfill was placed on the embankment. The first 3 m fill was placed with a rate of approximately 0.5 m per week without problems. After consolidation during 1 month the last 1.5 m was placed in less than a week resulting in a local rupture as indicated on Figures 1, 4 and 5. The deformation rate due to the fracture in the sandfill and gyttja layer was reduced to a consolidation rate after a couple of weeks. In front of the slope and outside the drainage system surface movement of the gyttja layer was measured (1 m and 11 m line, Figure 5). After the plastic deformations during the initial filling (1) and the supplementary filling (2) (Figure 7) an increased deformation during the rupture caused by the last 1.5 m filling (3) was observed. The direction of surface movements during the plastic deformation and the rupture is unchanged. The success of the low cost solution has only been achieved due to the detailed and advanced deformation measurements.

CONCLUSIONS

Settlements during reconsolidation are calculated from the modulus of compressibility, K , the time of settlement being short. Settlements during primary consolidation can be based on the consolidation ratio $Q \sim f(w)$ from oedometer tests, and the knowledge of preconsolidation, σ_{pc}' and the final stress level. σ_{pc}' can be evaluated from the variation of the in situ undrained vane shear strength, c_u . The secondary consolidation (creep without excess pore water pressure) is determined by the strain rate, ϵ_s (% per time log cycle). It seems that this value is greater in situ than determined by oedometer test. Preliminary results during the last part of the observation period indicate a moderate reduction of ϵ_s in connection with a partial unloading of the surcharge. Stability evaluation based on in situ vane shear strength and a correction factor is possible.

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