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SETTLEMENT CALCULATIONS OF GLOUCESTER TEST FILL

CALCULS DES TASSEMENTS SOUS REMBLAIS D'ESSAI – TYPE GLOUCESTER

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SYNOPSIS: The paper describes the calculations of the rate and magnitude of settlement under Gloucester Test Fill constructed by Division of Building Research, National Council of Canada during Stage I in 1967-1982. Gloucester Test Fill is one of the best instrumented embankments in the world and therefore ideal for comparing computational results. Settlement analysis was done using parameters determined from the oedometer and triaxial test results. The rate of settlement derived with Terzaghi's 1D-Theory is clearly much slower than the observed settlement. When estimating the primary phase with Terzaghi-Rendulic 2D-Theory and secondary phase with Buisman's equation the results approximate the observations very well. The theoretical time-settlement lines connect to each other pleasantly when the degree of consolidation U_p for the subgrade as a whole is about 80 %. It is possible that the primary and exceptionally large secondary settlement occur simultaneously when $U_p \geq 60$ %. Furthermore, the primary consolidation in bedded subgrade progresses at different rates in different layers.

TEST FILL AND SUBSOIL

The cross-section and the placing of the settlement gauges and piezometers of the Gloucester Test Fill constructed in 1967 are presented in Figure 1 by Lo et al. [1976]. This paper concentrates on the settlements directly below the centreline which correspond to the observations of settlement gauge S-1. The observations have been compared with the results of calculations with methods of classical soil mechanics in Figures 5 and 6. The observations of settlement gauge S-1 have also been handled with the $\sqrt{t}$ - log t -method developed by Brinch Hansen [1961].

The geotechnical layers of subsoil have been presented in Figure 2 [Law & Bozozuk 1979]. According to Figure 2, the division to the geotechnical layers can be defined with the natural water content, plasticity index and the undrained shear strength measured by field vane tests. For settlement calculations the layers have been even subdivided according to the changes of secant modulus defined from oedometer tests.

According to Figure 2, the subsoil consists of layers of clay and silty clay. Under clay layers at about a depth of 19 metres there is a sand and gravel layer with high permeability. During primary consolidation, there is a possibility for the water to flow both vertically up and down and in horizontal directions.

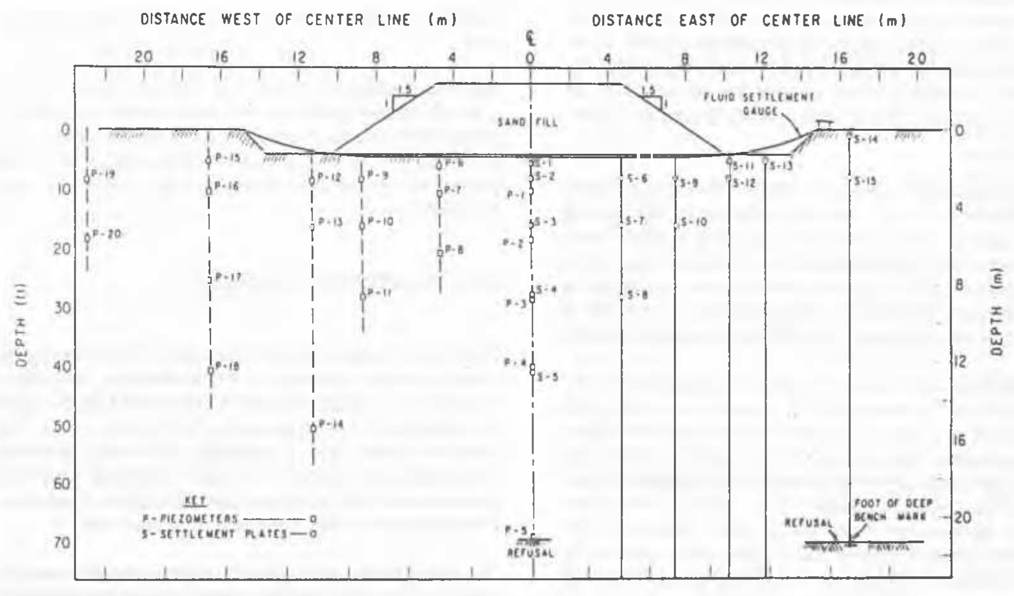

Figure 1. Gloucester Test Fill, Stage 1 (1967-1982). Location of settlement gauges and piezometers [Lo et al. 1976].

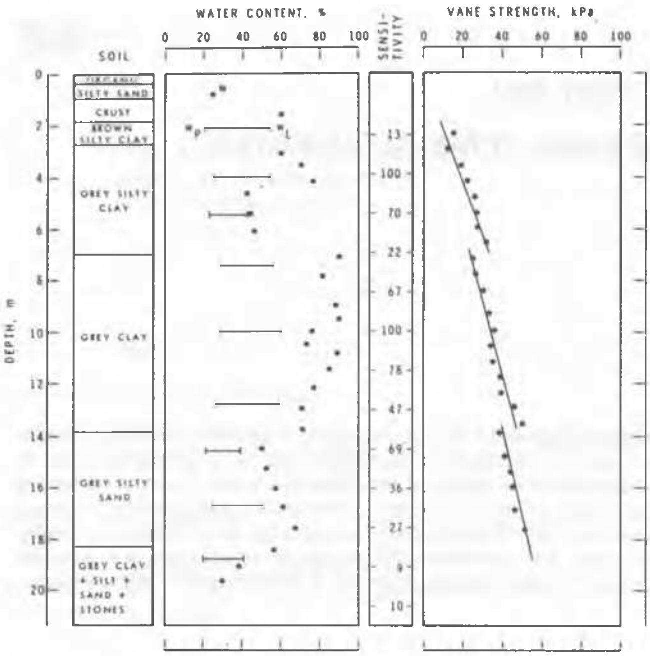

Figure 2. Gloucester Test Fill. Soil Profile [Law & Bozozuk 1979].

CONSOLIDATION STATE AND EXCESS STRESSES DUE TO THE EMBANKMENT

The effective initial vertical and horizontal stresses, σ_{v0}' and σ_{h0}', together with the preconsolidation pressure, σ_p', used in calculations are presented in Figure 3. It has been assumed that the groundwater level is 0.3 metres below the original surface, which will be discussed later in this paper. There are also displayed the total excess stresses, $\Delta\sigma_z$ and $\Delta\sigma_x$, due to the embankment, which have been calculated using the so-called Purdue solution [Perloff et al. 1967] in plain strain conditions. The effect of the 1.2 metres excavation before the construction has been considered while calculating the excess stresses. The Purdue solution takes into account the rigidity of the embankment. It has been evaluated with comparative results that in the case of Gloucester the Purdue solution is more suitable for calculation of the excess stresses $\Delta\sigma_z$ and $\Delta\sigma_x$ than the theory of Boussinesq [Fisher et al. 1982, Korhonen et al. 1986].

According to Figure 3, the subsoil is overconsolidated with respect to stress $\sigma_z' = \sigma_{v0}' + \Delta\sigma_z$, because $\sigma_z' < \sigma_p'$, with the exception of the layers at a depth of about 2...5 metres. That layer can be classified as lightly overconsolidated. Consequently, the overconsolidated layers have a strong effect on the mechanical behaviour of the subgrade, which becomes apparent for example from the settlement observations: the secondary settlement is exceptionally large and the rate of primary consolidation is relatively rapid.

It must be noticed that there have been two different assumptions of the groundwater level [Bozozuk & Leonards 1972, Fisher et al. 1982]. The change of the groundwater level due to seasonal variation can be defined only with reliable long-duration measurements which must be started long before the construction. In connection with Paimio trial embankment which was constructed in Finland 1989 [Vepsäläinen et al. 1991], it was found that the variation of groundwater level has a great impact on the determination of in situ effective stresses and, by that mean, also on the results of settlement analysis. It seems that the in situ groundwater and porepressure conditions have not been taken into account enough when test embankments have been designed.

OBSERVED SETTLEMENT AND BRINCH HANSEN'S METHOD

At the centreline directly beneath the embankment, the observed settlement of gauge S-1 has been approximated with the $\sqrt{t}$ - log t -method proposed by Brinch Hansen [1961]. According to Figure 4, the observed settlement can be approximated with $\sqrt{t}$ -method [Taylor 1948] upto the degree of primary consolidation of $U_p \approx 60$ %. Furthermore, the final part of the primary settlement, when $U_p \approx 60... 80$ %, does not significantly differ from the $\sqrt{t}$-S -line. As a result of the overconsolidated layers, the behaviour of the subsoil as a whole during the primary consolidation, when $S < S_{pf} = 26.9$ cm, $U_p \leq 90$ % and $t \leq$ about 1600 days, differs fundamentally from the behaviour of a normally consolidated subgrade.

The definition of the end-of-primary settlement, $S_{pf} = 26.9$ cm, in Figure 4 was difficult with traditional methods [Casagrande 1936, Taylor 1948]. After many trials, the end-of-primary settlement was finally derived from observations by approximating the relationship between time and settlement using Brinch Hansen's equation (1) [Brinch Hansen 1961].

$$U_p(t) = \frac{S_p(t)}{S_{pf}} = \sqrt[6]{\frac{t^3}{t^3+t_v^3}} \qquad (1a)$$

$$t_v = \frac{\pi H^2}{4c_v} \qquad (1b)$$

U_p (t) is	degree of primary consolidation at time t
S_p (t)	primary settlement at time t
S_{pf}	final value of primary settlement
t_v	characteristic time
H	drainage length
c_v	coefficient of vertical consolidation

Using equation (1) has major advantages compared with for example Casagrande's [1936] and Taylor's [1948] methods, because with equation (1) it is possible to determine the final value of primary settlement, S_{pf}, and the coefficient of consolidation, c_v, from observations while the degree of consolidation is $U_p = 0 \% \rightarrow 100 \%$. It can be seen from Figure 4 that equation (1) approximates the observed settlement during primary phase quite well.

The rate of settlement versus time drawing (Figure 5) gives almost the same value of primary settlement as Brinch Hansen's method in Figure 4. When primary and secondary settlement occur simultaneously as in Taylor's Theory B [Taylor 1942], it is impossible to determine the contact point between primary and secondary consolidation with Taylor's $\sqrt{t}$- and Casagrande's log t -methods.

CALCULATED SETTLEMENT

For the settlement analysis there were a lot of oedometer and triaxial test results available supplied by the constructor. In 1986, the Geotechnical Section of the National Research Council of Canada organized a Workshop on Predictions of the Engineering Performance of the Gloucester Test Fill, Stage II, which was a workshop for testing methods of geotechnical calculations and models. The team of Helsinki University of Technology, Laboratory of Soil Mechanics and Foundation Engineering, also took part [Korhonen et al. 1986, Lepidas & Magnan 1990].

For the present paper, the settlements and the rate of settlement were analyzed using the laboratory results and the field measurements presented by Fisher et al. [1982] and the methods mentioned in Figure 6.

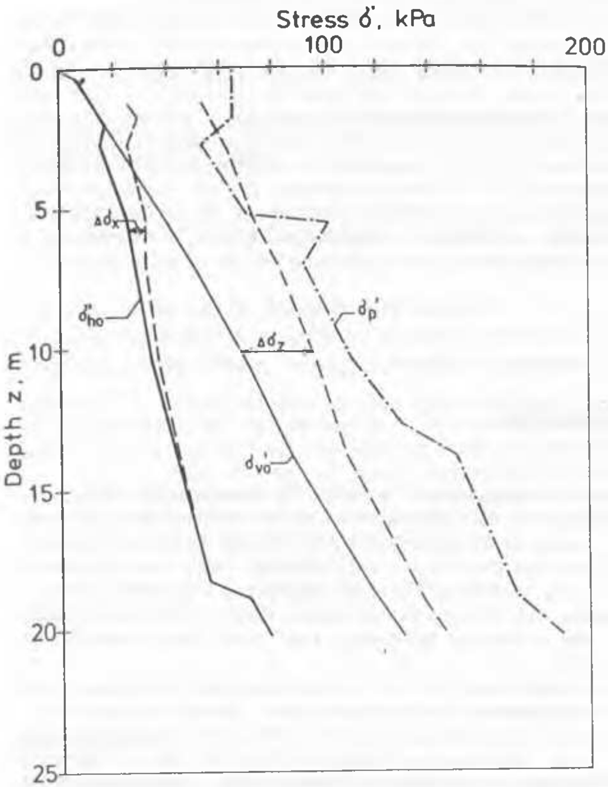

Figure 3. Gloucester Test Fill, Stage 1. In situ - and excess pressures due to the embankment.

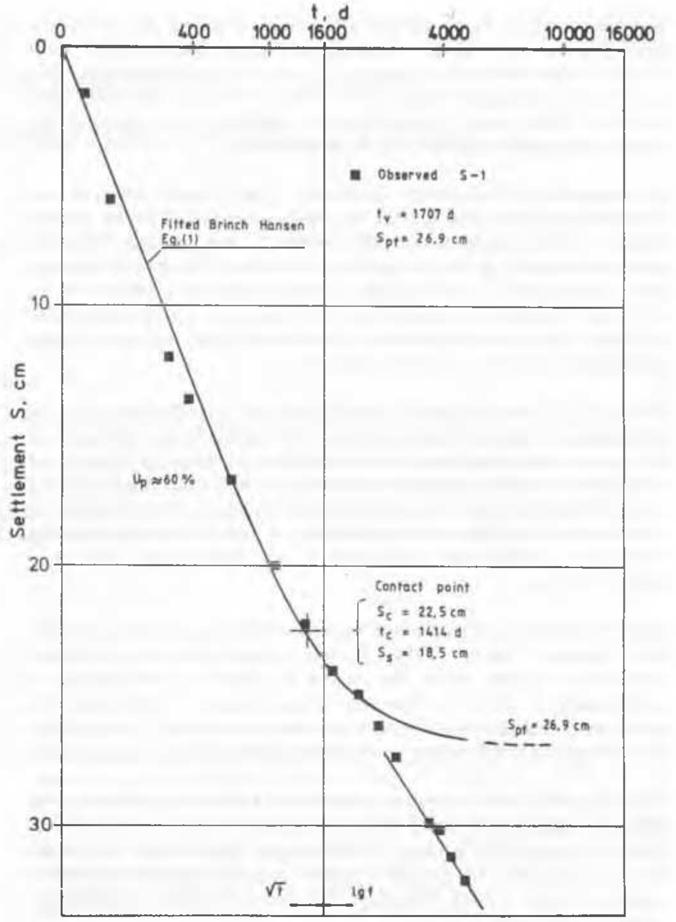

Figure 4. Gloucester Test Fill, Stage 1. Observed and fitted settlements in 1967-1982. Brinch Hansen's √t - log t -scale.

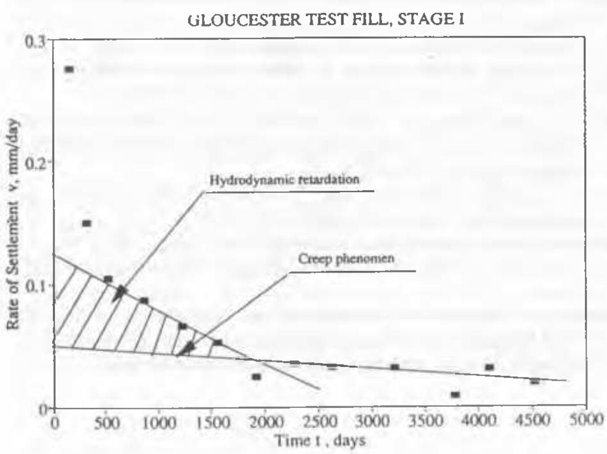

Figure 5. Gloucester Test Fill, Stage 1. Observed relationship between rate of settlement and time in 1967-1982.

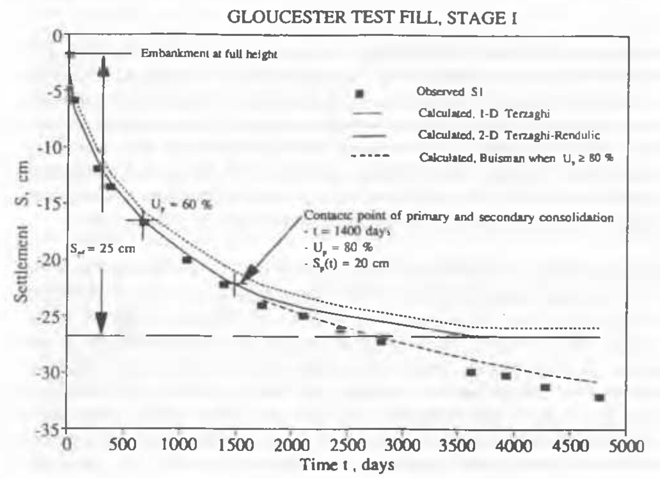

Figure 6. Gloucester Test Fill, Stage 1. Observed and calculated settlements in 1967-1982.

According to Figure 6, the relationship between settlement and time during 1967-1982 calculated with the 1-Dimensional Terzaghi Theory [1925] differs from the values observed evidently 1...2 years after the construction. The results using Terzaghi-Rendulic 2-D Theory [Rendulic 1936] are also presented. When using Terzaghi-Rendulic -method, it was assumed that drainage and strain conditions are two-dimensional.

The magnitude of the primary settlement in plain strain conditions was calculated using the excess stresses $\Delta\sigma_z$ and $\Delta\sigma_x$ derived by Purdue solution (Figure 3). The drained deformation modulus E_d was calculated from the secant modulus, M_s, of the corresponding stress state. The value of Poisson's ratio was taken as 0.3. The final value of primary settlement derived was S_{pf} = 25.0 cm. Therefore, it is quite close to the value S_{pf} = 26.9 cm determined by Brinch Hansen's method which also includes the settlement during construction (measured 1.8 cm in gauge S-1).

The rate of the two-dimensional consolidation was calculated by using the nomograms of Dunn and Razouki [1974]. The permeability coefficients k_v and k_h were taken from the field measurements presented by Fisher et al. [1982]. The use of the nomograms of Dunn and Razouki, which are based on the Terzaghi-Rendulic Theory, was somehow inconvenient. Anyhow, as it can be seen from Figure 6, the results derived with the Terzaghi-Rendulic 2D-Theory approximate fairly well to the observations when $t \leq$ 1600...1700 days.

Figure 6 includes the relationship of secondary settlement and time calculated with Buisman's theory [1936]. It was assumed that the secondary consolidation begins when the degree of primary consolidation is approximately $U_p \approx 80$ %. The time is then about $t \approx 1400$ days. The coefficient of secondary consolidation was derived from the C_α-values which were determined from oedometer test results published by Lo et al. [1976].

According to Figure 6, in this particular case, the 2-dimensional consolidation settlement calculated with the Terzaghi-Rendulic Theory combined with Buisman's method for secondary consolidation, approximates well to the observed settlements. The primary settlement changes gradually to secondary settlement when t = 1400...1600 days. The degree of primary consolidation is then $U_p \approx$ 80...90%.

The initial settlement in undrained state was calculated with CRISP90 [Britto & Gunn 1990]. CRISP90 was not used for other calculations because it has no models for secondary consolidation.

CONCLUSIONS

During the first stage (1967-1982), the subgrade of Gloucester Test Fill remained mainly overconsolidated. The division of the observed settlement into primary and secondary phases was found to be difficult using traditional methods. Without pore pressure observations the clear division would perhaps have been impossible. The contact point of primary and secondary consolidation appears clearly when coordinates of the rate of settlement versus time is used (Figure 5). It is still obvious that the change from primary to secondary phase occurs at different layers at different times.

The best prediction with manual calculations (Figure 6) was derived using the 2-dimensional Terzaghi-Rendulic Theory for primary consolidation combined with Buisman's semilogarithmic time function for secondary stage [Buisman 1936]. The method is not very good for practical design because of the amount of manual work. On the other hand, there are hardly any computer programs for calculating the magnitude and rate of settlement of structures constructed on overconsolidated clay and silt layers which would have reliable mechanical models for simulating the stress-strain-time -relationship on those layers. Most programs have been developed for normally consolidated conditions.

The observed settlements were approximated with Brinch Hansen's [1961] $\sqrt{t}$ - log t -method. Brinch Hansen's method was not very suitable for the case of Gloucester Test Fill, Stage I, because the secondary settlement was almost equal to the primary settlement. The extended Brinch Hansen, equation (1), can be used for approximation of primary settlement when thedegree of primary consolidation U_p = 0 %...100 %. With the theory it is also possible to evaluate the final value of primary settlement S_{pf}, and also the coefficient of primary consolidation c_v for the latter part of primary consolidation (U_p= 50 %... 90 %). In contrast, Taylor's $\sqrt{t}$ -method [Taylor 1948] can be used to determine the coefficient of primary consolidation c_v for the former part of primary consolidation $U_p \leq 50$ %. The value of c_v naturally changes during primary settlement because of the compaction.

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