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# Type A prediction of settlements for railway box culvert in road embankment on clay till
## Prédiction de type A des tassements d'un tunnel ferroviaire dans un remblai sur un dépôt argileux

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ABSTRACT: As part of the extension of the Danish Motorway System an existing railway line situated in a narrow valley had to be crossed. A box culvert was constructed in situ for the railway line, covered by a 20 m high embankment for the motorway. The type A prediction for settlement of the box culvert is described and compared with 18 years of settlement records. The main deposit in the area is a preconsolidated clay till of medium plasticity, some 60-70 m thick, overlaying the prequaternary deposits of black micaceous clay. The culvert was designed as a segmented box culvert as very large values of negative skin friction rendered a piling solution inappropriate. Based on extensive oedometer tests the settlement of the culvert was predicted subdividing the embankment into fifteen 20x30 m loading sections, with a total load of 1593 MN. The maximum settlement of the culvert is 300 mm.

RESUME: Dans le cadre de l'extension du système autoroutier danois, une ligne ferroviaire située dans une vallée étroite devait êter franchie. Un tunnel ferroviaire fût construit in-situ pour la voie ferrée, et recouvert par un remblai de 20 metres de hauteur pour l'autoroute. La prédiction de type A des tassements du tunnel est décrite et comparée avec 18 années d'enregistrements des tassements. Le principal dépot de ce site est un couche de moraine argileuse préconsolidée de plasticité moyenne, d'épaisseur 60-70 m, et recouvrant les dépôts pré-quaternaires d'argiles noires micacées. Le tunnel fût conçu comme une conduite à section carrée segmentée présentant de très importantes valeurs négatives de frottement superficiel rendant la solution des pieux inappropriée. Basé sur de nombreux essais de consolidation à l'œdométre, le tassement du tunnel fût prédit en subdivisant le remblai en quinze sections de chargement de 20x30 m, avec une charge totale de 1593 MN. Le tassement maximum du tunnel est de 300 mm.

## 1 INTRODUCTION

In connection with the extension of the motorway network in southern Jutland, Denmark, the layout crossed a narrow valley in a glacial landscape with an existing railway line (Figures 1, 2, 14).

Based on a thorough site investigation, including 17 geotechnical borings, and an extensive laboratory programme it was decided to cross the valley on a 20 metres high embankment of fill with a tunnel for the existing railway.

The main deposit in the area is a clay till of medium plasticity

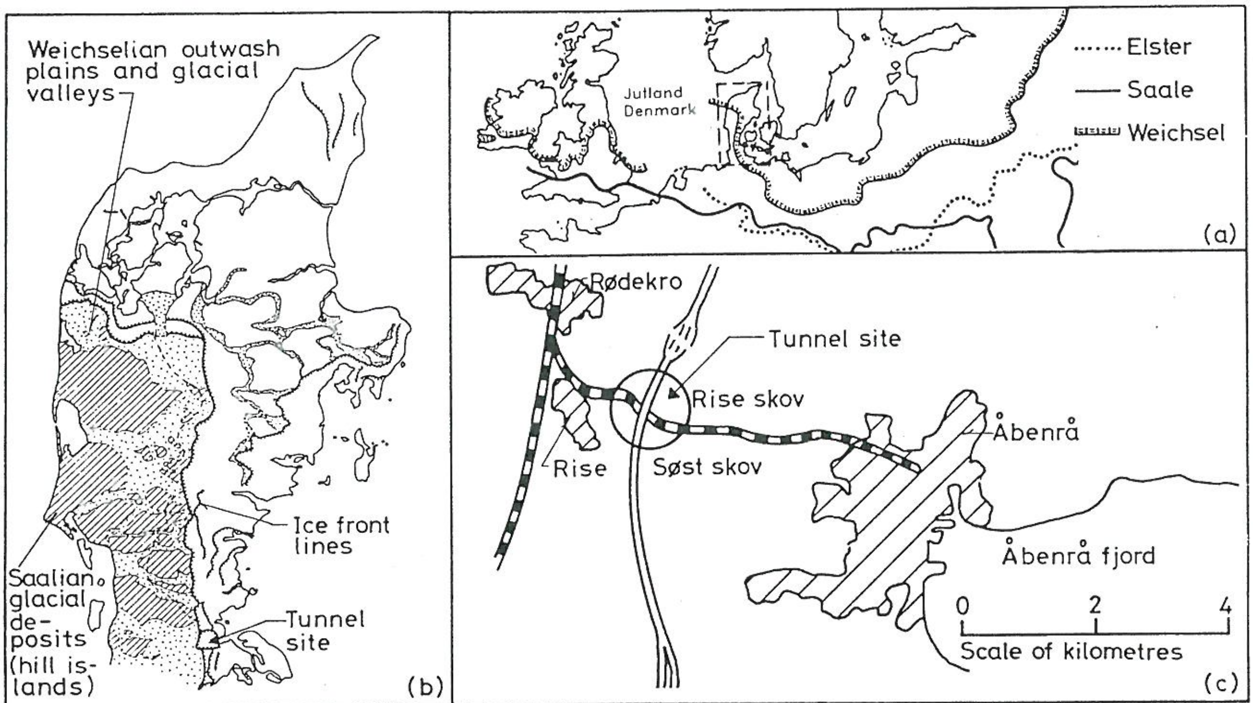


Figure 1. Location of railway culvert. (a) Extent of the Scandinavian glaciation during Elster, Saale and Weichsel Glacial Ages (after Woldstedt, 1954); (b) Main features of the Quaternary surface in Jutland, Denmark, with the maximum extent of the (youngest) Weichsel Glaciation (after Hansen, 1965); (c) Construction site on Rødekro - Aabenraa railway line (shaded areas denote towns).

($w \approx 20\%$, $w_L \approx 30\%$, $I_P \approx 17\%$), some 60-70 metres thick, overlaying the prequaternary deposits of black micaceous clay (from Miocene) at level -35 to -45 metres. Over the clay till solifluction deposits of sandy clay and sand from glacial time and very soft peat and gyttja (organic clay) from postglacial time are present. It was decided to replace the very soft deposits with compacted granular fill.

Various designs for the railway tunnel were considered. Large values of negative skin friction rendered a piling solution inappropriate and as relatively large settlements were expected the tunnel was designed as a segmented box culvert, allowing differential settlements between tunnel segments (cf. Figure 14).

## 2 SITE INVESTIGATION

The construction site is situated in one of the strongly accentuated valleys leading from Aabenraa Fjord towards the outwash plains of Tinglev. The landscape was formed during the last part of the latest glaciation (Weichsel period; see Figure 1) by glaciers progressing through the present Aabenraa Fjord. The hills are end moraines formed by the glaciers and may consequently comprise disturbed layers and floes. In the valleys there is clear evidence of kettle holes later filled by meltwater deposits and solifluction deposits from the surrounding hills. Subsequently, the lower areas were filled with peat and gyttja (post glacial). The clay tills in the area are of medium plasticity which could be due to mixing with glacial meltwater deposits.

A plan of the construction area, indicating the 17 geotechnical borings and 10 soundings of the site investigation is shown in Figure 2.

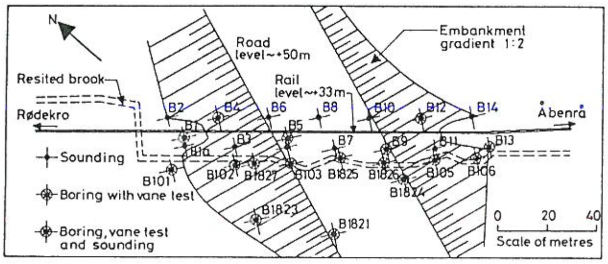


Figure 2. Plan of construction site (extent of site investigation).

The longitudinal section of the soil profile along the railway line appears from Figure 3. The numbers K1-K9 on the Figure indicate samples used for oedometer testing. Grain size curves for the present clay till are shown in Figure 4, and for comparison a typical Danish clay till is included.

The interpreted surfaces of the clay till and the glacial clay deposits at large, based on the borings and soundings are shown in Figures 5a and 5b, respectively. Especially the extent of the solifluction deposits is important as increased settlements were anticipated in this deposit. The undrained shear strength profile measured by field vane is shown in Figure 6 corresponding to the borings indicated on Figure 2. The depth has been taken as depth below the top of the glacial clay deposits (cf. Figure 5b). The strength increases with depth, but it is apparent that the shear strength of the solifluction deposits at the top is on the lower side. Note that values of $c_u$ in excess of 500 $kN/m^2$ are not unusual for Danish clay tills.

Due to the relatively high clay content, 30-40%, the natural water content in the glacial deposits is high, i.e. $w \approx 20\%$, compared with typical Danish clay tills (with clay contents $\leq 20\%$,

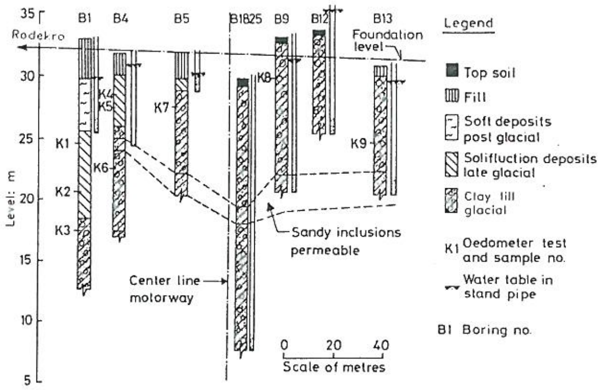


Figure 3. Longitudinal section of soil profile along railway axis.

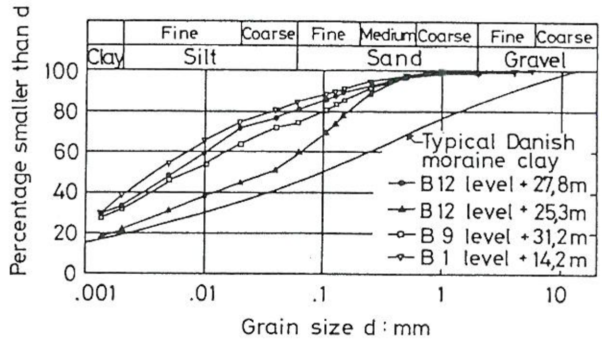


Figure 4. Grain size distribution for the clay till.

water contents of 9-15% and $w_L \leq 24\%$, $I_P \leq 12\%$).

The very soft deposits of peat and gyttja (see Figure 3) towards Rødekro (NE) with water contents $w$ in excess of 100% were replaced by granular fill.

## 3 LABORATORY TESTS

With the nine intact samples indicated on Figure 3, oedometer tests were carried out by the Danish Geotechnical Institute (DGI). Data for the samples are shown in Table 1.

The oedometer is a special type with floating ring developed by Jacobsen (1970) for tests on firm soils. The samples obtained in the field have a diameter of 70 mm and samples are then trimmed to the oedometer sample size of 60 mm diameter and 30 mm height. The deformation of the oedometer system itself is less than 0.001 mm for a load increment of 1200 $kN/m^2$.

To reduce bedding effects in firm soil with stones, the samples are cast in gypsum against the pressure heads, and only one filter disc with a diameter of 20 mm is used in the bottom plate to reduce deformation by penetration of the filter stone. Any stones protruding from the sample surface are very carefully removed and the hole is filled with gypsum.

As the first part of the loading in the oedometer is an adaptation to the ring the first stress increments are rapidly applied to avoid swelling. Further, the actual slope of the first part of the primary curve is mainly a reflection on the success of accurate sample preparation. The estimation of the preconsolidation pressure based on the Casagrande construction is hence uncertain (as is generally true for firm soils) and is in Table 1 based on the reloading part of the consolidation curve after initial loading and unloading.

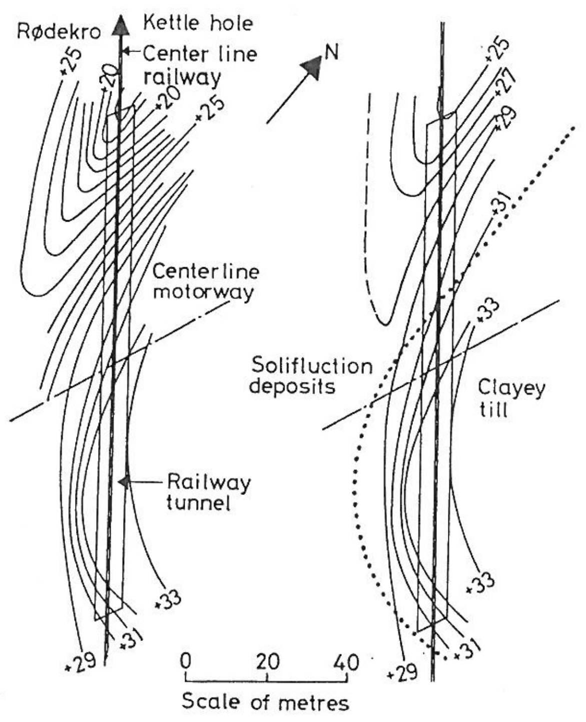


Figure 5. Level in metres of interpreted clay surface: (a) Clay till; (b) Glacial clay deposits.

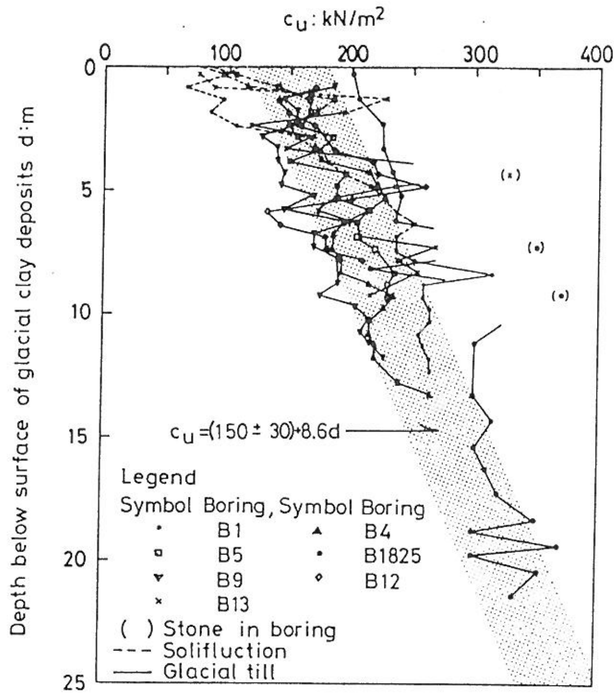


Figure 6. Undrained shear strength profile based on field vane tests corresponding to longitudinal section of Figure 4.

The standard DGI procedure was adopted for oedometer testing:

(i) Stepwise loading to a vertical stress below the preconsolidation pressure $\sigma'_{pc}$ taking care to avoid swelling of the sample (to reduce the effect of sample disturbance).

(ii) Unloading to the estimated minimum geological vertical effective stress $\sigma'_{min}$ (in this case estimated as the depth below the surface of the glacial clay deposits multiplied by the effective unit weight of the clay).

(iii) Stepwise loading from $\sigma'_{min}$ to a stress level in excess of the expected stress level due to the embankment load for determination of the oedometric tangent moduli $K_t = E_{oed} = 1/m_v$.

For samples K2 and K3, representing the highest preconsolidation pressure, reloading cycles from two different unloading levels $\sigma'_a$ were carried out in order to find the variation in oedometer modulus with the unloading stress level. The results could thus be used to represent other depths in the profile.

The initial tangent moduli $K_t(=1/m_v)$ for reloading may be found as function of the minimum applied effective unloading stress $\sigma'_a$ (Figure 7). Extensive test series on clay tills (f.inst. Jacobsen, 1970; Kristensen et al, 1995) suggest a linear $K_t$, $\sigma'_a$ relationship and the test results here indicate

$$K_t = 5\ MN/m^2 + 380\ \sigma'_a \tag{1}$$

$$K_t = 5\ MN/m^2 + 600\ \sigma'_a \tag{2}$$

for samples K2 and K3 respectively.

Considering all nine samples to represent the glacial deposits the tangent modulus as function of the unloading stress is entirely within the range (cf. Figure 8)

$$K_t = 5\ MN/m^2 + (500 \pm 125)\ \sigma'_a \tag{3}$$

Judged from Figure 8 and Table 1, combined with general trends for solifluction deposits it was assumed that the upper limit in

Table 1. Data for oedometer samples.

| Sample No. | Clay type | Below clay surface $d$ (m) | Water content $w$ (%) | Unit weight $\gamma$ $(kN/m^3)$ | In situ stress $\sigma'_0$ $(kN/m^2)$ | Field vane strength $c_u$ $(kN/m^2)$ | Precons. pressure[a] $\sigma'_{pc}$ $(kN/m^2)$ | Precons. pressure[b] $\sigma'_{pc}$ $(kN/m^2)$ | Modulus/ strength[c] $K_t/c_u$ |
|---|---|---|---|---|---|---|---|---|---|
| K1 | solifluction | 0.5 | 18.8 | 20.9 | 90 | 70 | 180 | 240 | 117 |
| K4 | solifluction | 1.3 | 23.4 | 20.0 | 35 | 185 | 625 | 820 | 86 |
| K5 | meltwater | 2.3 | 22.0 | 20.4 | 45 | 150 | 360 | 980 | 108 |
| K7 | glacial till | 2.3 | 19.6 | 20.9 | 84 | 158 | 480 | 1040 | 145 |
| K8 | glacial till | 3.6 | 20.3 | 20.6 | 62 | 125 | 460 | 750 | 213 |
| K2 | solifluction | 4.5 | 20.5 | 20.6 | 134 | 200 | 440 | 850 | 132 |
| K9 | glacial till | 5.3 | 21.0 | 20.8 | 75 | 190 | 680 | 690 | 150 |
| K6 | glacial till | 6.3 | 18.4 | 21.0 | 89 | 190 | 410 | 550 | 206 |
| K3 | glacial till | 7.5 | 15.0 | 21.7 | 167 | 240 | ? | 890 | 233 |

[a] $\sigma'_{pc}$ using Casagrande [b] $\sigma'_{pc}$ using Equation 4 [c] values using Equation 3

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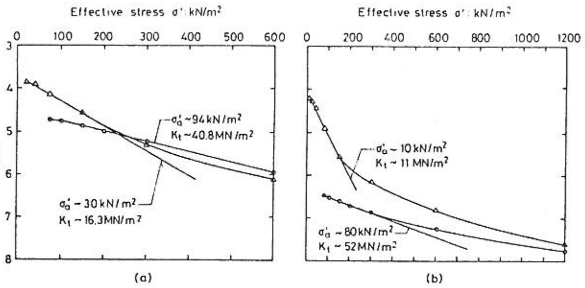


Figure 7. Reloading cycles for samples (a) K2; (b) K3.

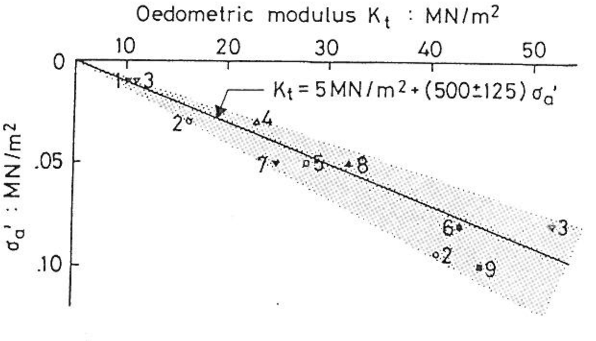


Figure 8. Oedometer moduli for reloading cycles in all oedometer tests (number in parenthesis indicate sample no.).

equation (3) applied for the south eastern part of the tunnel whereas the lower limits is more likely for the north western part where solifluction deposits are present (see Figure 3).

The modified secondary compression indices from the oedometer tests $C_{\alpha\varepsilon}(=\partial\varepsilon/\partial\log_{10} t)$ are shown in Figure 9 as function of the consolidation stress. It is apparent that $C_{\alpha\varepsilon}$ increases with the consolidation stress and hence it is questionable whether the preconsolidation pressure has been exceeded in the tests. However, if $\sigma'_{pc}$ is assumed to correspond to $C_{\alpha\varepsilon} \approx 0.2\%$ a reasonable agreement with Equation 4 below is obtained.

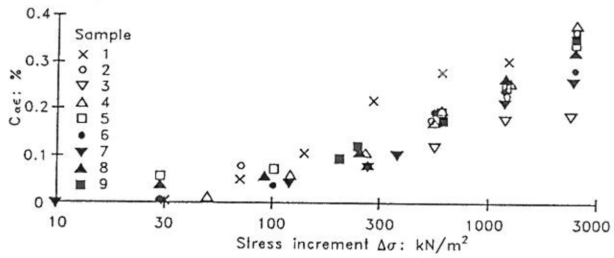


Figure 9. Modified secondary compression indices $C_{\alpha\varepsilon}$ for oedometer tests as function of consolidation pressure $\sigma'$ (=$\Delta\sigma$).

## 4 USE OF CORRELATIONS

Use of global correlations for soil parameters as functions of, $I_p$ OCR etc. (Ladd et al, 1977) were not originally adopted in the present study as the conducted in situ and laboratory tests formed a sufficient basis for the type A prediction required. Moreover, the Author shares the views expressed by Schmertzmann and Morgenstern (1978) that global correlations can be directly misleading and should only be employed if the correlation parameter has a dominant theoretical or logical relationship to the physical behaviour predicted by the correlation. The SHANSEP approach (Ladd et al, 1977) has however been shown to work well for prediction of OCR in Danish clay tills (Steenfelt and Foged, 1992). Based on results for the same type of clay till as in the present project, the formula

$$\frac{(c_u/\sigma'_{v0})_{oc}}{(c_u/\sigma'_{v0})_{nc}} = OCR^{0.8} \tag{4}$$

with $(c_u/\sigma'_{v0})_{nc} = 0.35$ would be expected to apply. The corresponding values of $\sigma'_{pc}$, shown in Table 1, are however much higher than inferred using the Casagrande method.

The stiffness values of Eq. (3) may be compared with values reported by Stroud and Butler (1975) for boulder clays with $I_p \approx 20\%$. They found values of $1/(m_v c_u) \approx K_t/c_u \approx 100$ (range from 60 to 140). The actual $K_t/c_u$ values for the oedometer tests are shown in Table 1, where the range is from 50 to 230 increasing with depth.

It would thus not be advisable to base settlement estimates on global trends, but regional correlations or "comparable experience" are helpful (use of the actually measured tangent moduli was in fact advocated by Simons, 1975).

## 5 SETTLEMENT ANALYSIS

To assess the settlement of the railway culvert the embankment was subdivided into fifteen 20 x 30 m loading sections (preliminary calculations indicated that further extension of the grid to the north or south had insignificant influence on calculated values of settlement). The corresponding loads from the fill and the tunnel are summarised in Figure 10. The load consists of approximately 58 MN net replacements of soft deposits and 1535 MN embankment fill and concrete, i.e. a total load of 1593 MN. The embankment was carried out as a sandwich construction with alternating 0.5 metre thick layers of sand/gravel and glacial till.

In the calculation of the in situ stresses the average ground water table is assumed at level +31 and the average effective unit weights are taken as $\gamma' = 10 kN/m^3$ for fill, $\gamma' = 11 kN/m^3$ for solifluction deposits and $\gamma' = 11.5 kN/m^3$ for clay till.

The vertical stress increments at any depth may be found by means of Boussinesq's stress distributions for rectangular uniformly loaded areas. For contributions from non-neighbouring sections adequate accuracy is obtained when the load is considered a point load at the centre of the section.

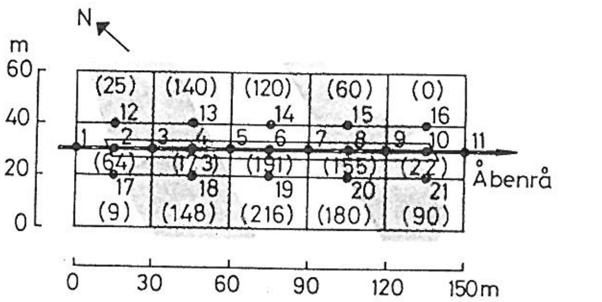


Figure 10. Sectional subdivision of embankment fill and culvert (numbers are points where settlement calculation is carried out and numbers in parenthesis are loads in MN for the section).

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The stress increment due to embankment, fill and culvert below points 1-21 on Figure 10 are easily calculated. Assuming the prequaternary surface at level -40 metres (nearby very deep borehole, see Dinesen et al, 1977) the overlying glacial clays are subdivided into 8 layers of increasing thickness with depth (from app. 2 to 20 metres). The settlement corresponding to layer i with stress increment $\Delta\sigma_i$ in the middle of the layer may now be computed using the oedometer moduli Equation 3 as

$$\Delta\delta_i = \varepsilon_i \Delta h_i = \frac{\Delta\sigma_i}{5\,MN/m^2 + (500 \pm 12)\sigma'_{ui}} \Delta h_i \quad (5)$$

where the unloading stress $\sigma'_{ui}$ is the estimated minimum geological vertical stress before placing the fill and culvert. Equation 5 is only valid when $\sigma'_{ui} + \Delta\sigma_i$ is well below the preconsolidation pressure $\sigma'_{pc}$, i.e. corresponding to an unloading-reloading cycle in the oedometer test. If $\sigma'_{pc}$, is exceeded (only actual for the top layers below the middle of the embankment) $\varepsilon_i$ may be estimated directly using the oedometer compression diagrams. However, with the range of oedometer moduli indicated by Equation 3 the range of expected settlements would include a minor exceed of $\sigma'_{pc}$.

The contribution to the settlements from the prequaternary micaceous clay below level -40 metres may be disregarded as the clay is overconsolidated and the stress increments at that depth are small. Local experience suggests a conservative value of oedometer modulus of $K_t \approx 5\,MN/m^2 + 250\sigma'_u$ (i.e. smaller than the value for the glacial till). For the estimated thickness, 106 m, of micaceous clay the settlement contribution for the middle part of the tunnel is less than 0.01 m.

Figures 11 and 12 show the resulting vertical settlement predictions for the box culvert axis and the corresponding rotation about the axis for primary consolidation. A positive rotation $r_x$ corresponds to increasing settlements from NW to SE ($r_x$ = settlement at horizontal distance x from the axis divided by x).

For all practical purposes primary consolidation is completed for $T_v \approx 1$ corresponding to an average degree of consolidation of $U \approx 0.93$. The maximum average drainage path for the upper part of the glacial deposit is H = 4 to 5 metres as more sandy and permeable inclusions are present some 8-10 metres below the culvert. The major contribution to the settlement (60-70%) is from the top 15-20 metres of the deposits. At 10 metres depth the vertical unloading stress is $\sigma'_u \approx 115 kN/m^2$, i.e. corresponding to an oedometer modulus of $K_t = 62.5\,MN/m^2$ according to Equation 3. The time for primary consolidation is consequently of the order

$$t_c = \frac{T_v H^2}{c_v} = \frac{T_v H^2 \gamma_w}{k\ K_t} \approx \frac{1 \cdot 4.5^2 \cdot 10}{10^{-10} \cdot 62.5 \cdot 10^3} = 3.2\ 10^7 \sec \quad (6)$$

i.e. one year. The value of permeability, $k = 10^{-10}\,m/\sec$, is an average value determined in the oedometer tests. The range is $5\ 10^{-11} - 2\ 10^{-10}\,m/\sec$, decreasing for increasing effective stress.

Based on the rates of secondary compression from the oedometer tests an approximate average $C_{\alpha\varepsilon}$ may be defined as function of the consolidation pressure (cf. Figure 9).

For the middle part of the tunnel site with the highest consolidation stresses and maximum estimated settlements the excess stresses from the fill and tunnel vary from app. $320 kN/m^2$ at level 31 to $100 kN/m^2$ at level -40 (i.e. from top to bottom of the clay till).

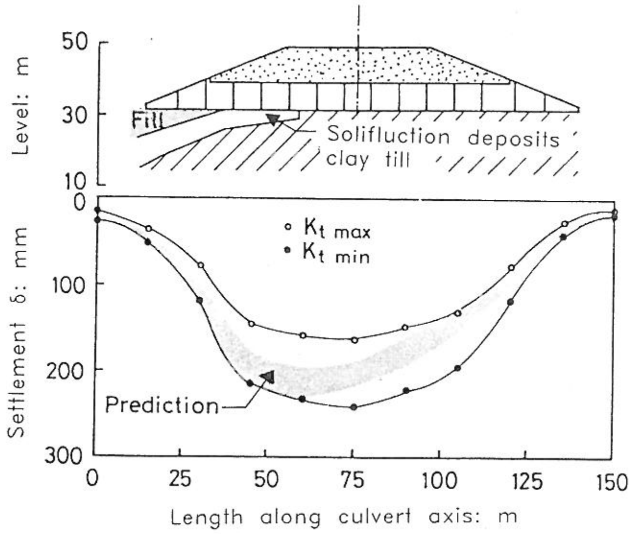


Figure 11. Settlement prediction corresponding to end of primary consolidation for tunnel axis ($T_v = \pi/4$, $U = 0.89$).

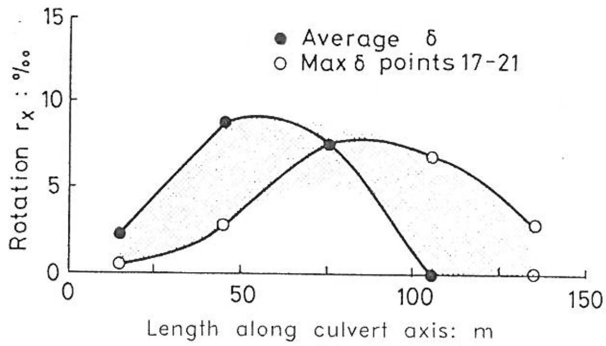


Figure 12. Predicted magnitude of rotation about railway axis at end of primary consolidation (shaded area is prediction range).

With an average $\Delta\sigma' \approx 280 kN/m^2$ over the top 15 metres and $\Delta\sigma' \approx 165 kN/m^2$ over the remaining 55 metres the range of $C_{\alpha\varepsilon}$ - values would be 0.08 - 0.15% and 0.07 - 0.12%, respectively (cf. Figure 9). The range of expected maximum secondary settlement is consequently 50 to 90 mm/log cycle of time.

With the estimated $t_c \approx 1$ year (Equation 6) the secondary settlement over a 10 years period was thus expected to be less than 90 mm for the middle part of the tunnel. Furthermore, a linear relationship between primary and secondary settlement was expected.

## 6 SETTLEMENT OBSERVATIONS

The settlements for all major bridges and tunnels in the railway network were regularly monitored in a period after construction by the Danish State Railways up to 1995/96. Considering the rather large predicted settlements of the Rødekro-tunnel a very extensive monitoring programme was established, measuring the vertical movement of four points in the extremes of the base of each tunnel segment. All 60 measuring points were established and zero levellings taken in March 1976. The start time for consolidation was taken as July 1, 1976 when placing of the embankment fill started. The embankment was finished late August, 1976, see Figure 14.

Mean values of the settlements from July 1, 1976 to June 14, 1994 for all 15 tunnel segments are shown in Figure 13, using the special $\sqrt{t}$, $\log_{10} t$ diagram introduced by Brinch Hansen (1961). This is standard for plot of time-curves from oedometer tests with a squareroot scale from $t$=0 to $t = t_c$ and log scale for $t \geq t_c$.

In comparison to other settlement-time diagrams the $\sqrt{t}$, $\log_{10}t$ diagram is more versatile and reflects in an elegant way the theoretical background of the applied consolidation theory.

It appears from Figure 13 that with t = 1 year (the estimated time of primary consolidation) as boundary between the $\sqrt{t}$ and $\log_{10} t$ scales the approximate straight lines for the observed settlements, corresponding to primary and secondary consolidation respectively, do in fact intersect close to t = 1 year. As could be expected, the initial settlements are not negligible.

However, the available settlement observations only allow a tentative distinction between initial and primary settlements, which is further obscured by the gradual application of full embankment load.

In Figure 15 the settlements corresponding to (I) completed primary consolidation, and (ii) one decade of secondary consolidation from Figure 13 are shown together with the estimated range of settlements and the recorded settlements 468 and 1808 days after the completion.

In general the predictions agree well with the observations in that the recorded values are close to the maximum prediction for culvert segments 1-6, where the solifluction deposits are present, and close to the minimum prediction for the remainder of the culvert.

The maximum secondary settlement for culvert segment 5, corresponding to Figure 13, is approximately 70 mm (for the first time decade 1-10 years), i.e. within the predicted range 50 - 90 mm/log cycle of time.

The observed rotations of the segments about the axis of the culvert for t = 468 days, i.e. roughly corresponding to the end of primary consolidation are shown in Figure 16 together with the range of predictions. Again the predictions agree well with the observed overall behaviour. The maximum observed rotations correspond to the part where the solifluction deposits are present.

## 7 DISCUSSION

At the time of prediction the designer's immediate reaction was that the predicted settlements were an order of magnitude greater than expected. However, the predictions are certainly borne out by the observations. The misconception that only minor settlements should be expected in Danish overconsolidated clay tills is understandable as these deposits are normally very stiff. One purpose of the Paper is therefore to warn against this generalisation and to encourage alertness when the clay content (per cent of fines less than 0.002 mm) in clay tills exceed some 20%.

Also the investigation emphasises the value of thorough site investigations and laboratory tests. The procedure adopted for interpretation of the oedometer tests with a linear relationship between oedometer tangent modulus and the vertical unloading stress (Jacobsen, 1970) proved adequate for this prediction too.

It is interesting that the ratio between settlements corresponding to secondary consolidation, $\delta_s$ per time decade (1-10 years), and end of primary consolidation, $\delta_c$ ($\approx$ 1 year), is constant, $\delta_s/\delta_c \approx 28\%$, for all tunnel segments (data from Figure 13). A corresponding constant ratio $\varepsilon_s/\varepsilon_c$ is a feature of the

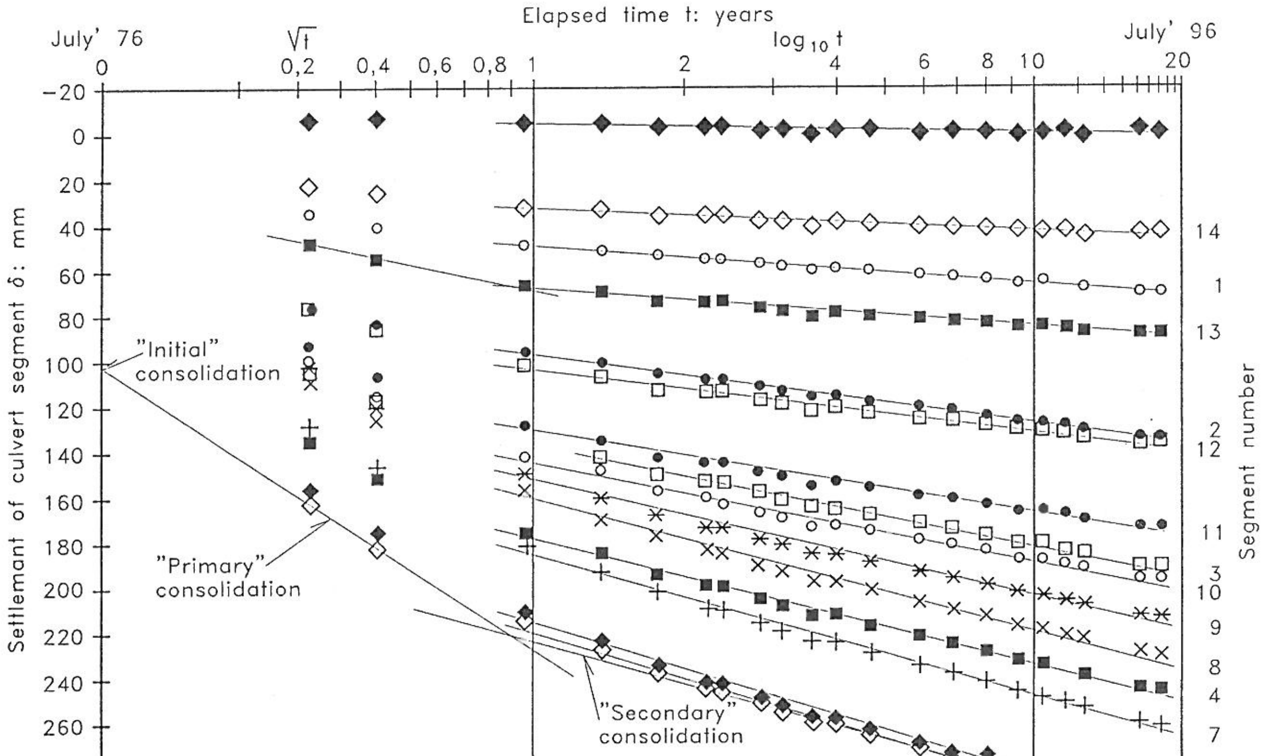


Figure 13. Observed mean settlements of culvert segments 1-15 plotted using combined $\sqrt{t}$, $\log_{10} t$ time scale (change of scale at t = 1 year).

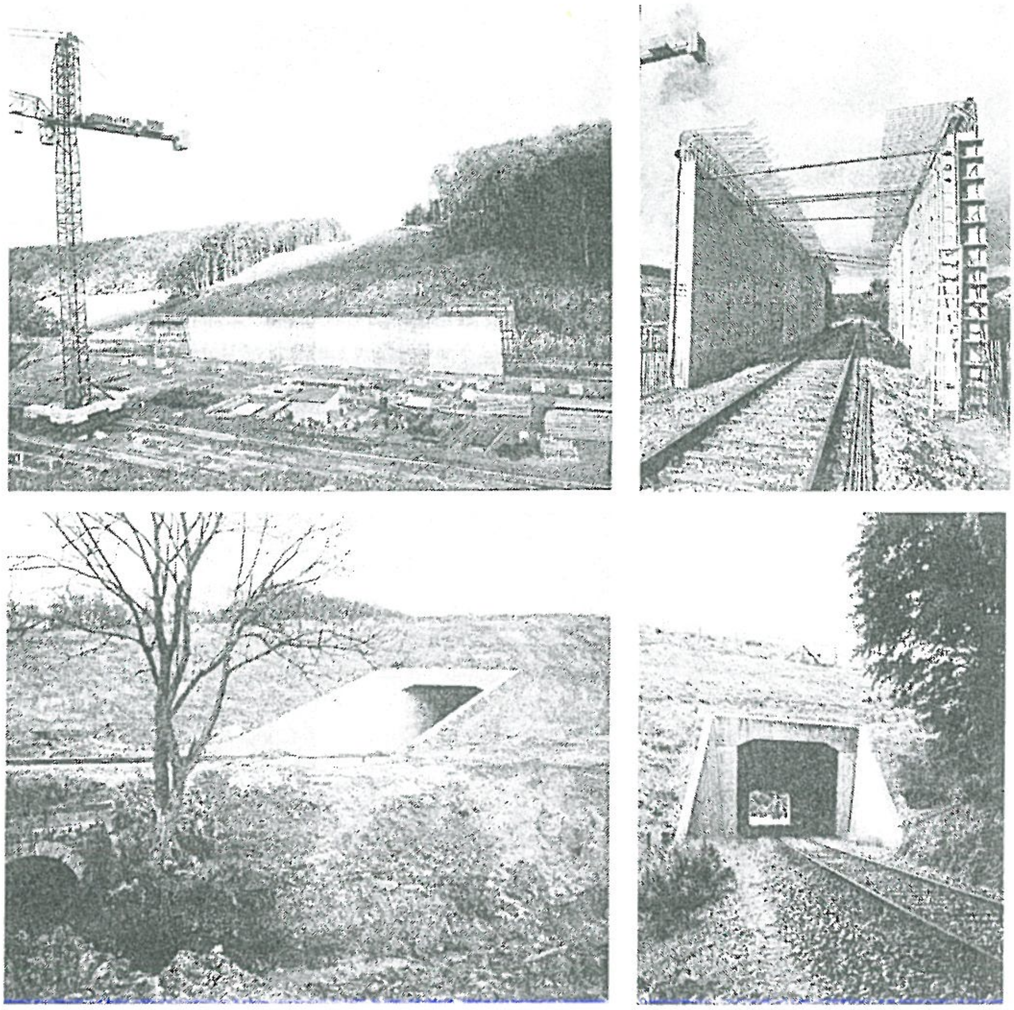

Figure 14. General view of railway culvert and embankment. (a), (b): Views towards NE and NW, September 1975, after completion of bottom plate; (c): View towards E, October 1976, with ancient stone tunnel for brook crossing; (d): View towards N, July 1981. (Photos by B. Schwartzlose).

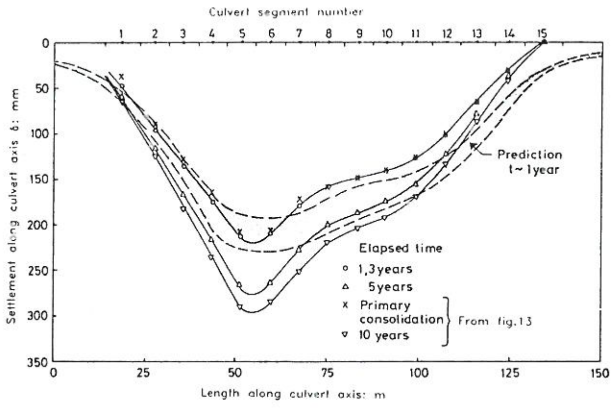


Figure 15. Comparison of observed and predicted settlements for culvert segments.

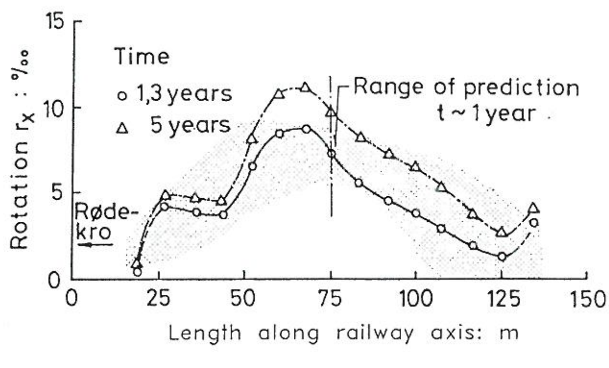


Figure 16. Comparison of observed and predicted rotation of tunnel segments about railway axis for culvert segments.

model law for simultaneous primary and secondary consolidation proposed by Brinch Hansen (1961).

Unfortunately no settlement observations were made during the construction of the embankment, which could have elucidated the distribution between initial and "proper" primary consolidation. Ratios of the same order of magnitude (35-45%, data from Figure 13) between initial and end of primary settlement, are found using elastic theory for strip footings on a layer of 1-2 times the width of the footing. The influence on time-settlement behaviour from the two months construction period of the embankment is negligible for times after approximately three months.

The present case history seems to endorse the Danish practice that the predicted primary settlements, for firm soils, based on conventional consolidation theory and oedometer moduli includes the combined effects on initial and primary consolidation.

The differential settlements and rotations between tunnel segments have as yet not presented any problems although the monitoring teams have reported visible relative displacements between segments. Knowing that differential settlement could occur special joints with bitumen and rubber gaskets were used and only at midheight in the side of the segments are the reinforcing bars through-going.

## 8 SUMMARY AND CONCLUSIONS

The settlement and rotation of a segmented 125 m long railway box culvert in a motorway embankment have been predicted and compared with extensive settlement records. The soils underlying the tunnel and embankment are predominantly 60-70 metres of overconsolidated clay till of medium plasticity, which in parts is covered by solifluction deposits.

The soil profile was established by a thorough site investigation and the deformation properties of the clays were investigated by oedometer tests where the stress-history of the sample was reproduced. Based on a linear relationship between oedometer tangent modulus and unloading stress the settlements due to a total load of 1593 MN over an area of 60 x 150 $m^2$ were calculated in 21 points along and close to the axis of the railway culvert. This allowed prediction of the settlements along the axis and also the rotation about the axis.

The predictions have been compared with an extensive settlement record (60 points monitored 21 times over an eighteen year period) and the predictions reflect well both magnitude and distribution of observed settlements and rotations with a maximum settlement of 300 mm after eighteen years.

Secondary consolidation increases the settlements at constant rates $C_{\alpha\varepsilon}$ for all tunnel segments in accordance with predictions. The rates $C_{\alpha\varepsilon}$ increase linearly with the magnitude of the respective settlement at the end of primary consolidation.

The Paper exhibit the inherent difficulties associated with predictions and performance. The client wants (for short term commercial reasons) type A predictions based on the minimum amount of data rendered probable by a minimum of performance records. Consequently significant data (here pore pressure and settlement distribution with depth etc.) are usually missing for detailed evaluation of the applicability of the predictive method.

However, it is believed that the success of the Type A prediction described is not entirely due to luck but rather to application of a simple rational no-nonsense approach supported by regional, comparable experience.

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