Long-term settlement observations of a bridge foundation on clay of very high plasticity

Observations en matière d'affaissement à long terme de fondations de pont sur un argile de très haute plasticité

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ABSTRACT

Four bridge piers, all of which are resting on preconsolidated clay of very high plasticity, have, since erection, had a measured settlement of between 0.25m and 0.65m. This paper presents the settlement observations carried out over a period of more than 75 years. The settlements progress in a nearly linear manner when plotted against log time. Together with the observed fast progress of the settlement, considering the extremely small permeability of the clay, this strongly suggests that primary consolidation cannot be the primary cause of the observed settlements. Therefore, other causes must be found to explain the observed behaviour. Some form of creep seems obvious. Perspective calculations strongly suggest that the observed behaviour is primarily caused by plastic deformation, developed as a result of applied loads approaching the bearing capacity of the clay (too little safety against bearing capacity failure). The piers with the smallest bearing capacity reserve have the greatest settlements, and the slope of the seabed plays a crucial role.

RÉSUMÉ

Quatre piles de pont, reposant toutes sur de l'argile préconsolidé de très haute plasticité, présentent, depuis leur érection, un affaissement mesuré allant de 0,25 m à 0,65 m. Cet article présente les observations en matière d'affaissement effectuées sur une période de plus de 75 ans. L'affaissement progresse de façon pratiquement linéaire lorsqu'il est calculé par rapport à l'instant de relevé. Lié au caractère rapide de la progression d'affaissement observée, compte tenu de la perméabilité extrêmement faible de l'argile, une telle linéarité semble pratiquement exclure la possibilité que les affaissements constatés soient essentiellement dus à la consolidation primaire. Par conséquent, il convient de chercher d'autres causes au comportement observé. On constate à l'évidence une certaine forme de fluage. Les calculs de perspective suggèrent avec insistence que le comportement observé est essentiellement le fruit d'une déformation plastique, liée au fait que les charges appliquées se rapprochent de la capacité portante de l'argile (marge trop faible par rapport à un défaut de capacité portante). Ce sont les piles soumises à la pression la plus forte qui se sont le plus affaissées, et l'inclinaison du fond marin joue un rôle déterminant en la matière.

Keywords : Bridge pier, clay of very high plasticity, slickenside, preconsolidated, Paleogene, settlement observations, consolidation, creep, oedometer modulus, permeability, undrained shear strength, vane shear strength, effective shear constants, plastic deformation, yielding, bearing capacity failure, safety ratio, stability, undrained condition, drained condition, PLAXIS calculations.

1 BRIDGE FOUNDATION

The old Little Belt Bridge which connects the island of Fyn with the peninsula of Jylland was opened on 14 May 1935.

The bridge crosses Little Belt (Lillebælt) on 4 piers, which are numbered 1-4 in the direction from Fyn to Jylland, cf. Fig. 1. Water depth at pier 1-3 is approx. 30 m, while at pier 4 it is only approx. 23 m.

Figure 1. The old Little Belt Bridge is 1.178 metres long and 20.5 metres wide. The passage height is 33 metres. Fyn is seen in the distance. Photo: H.C. Steensen.

The lower part of the piers consists of a caisson that was cast on shore and launched. Out in the water, the caisson was turned by ballast, and the elevated undersea portion of the piers were cast. Then the pier was put into place on the bottom of the belt by ballast and helped into the substrata by drilling the ground up through the concrete pipes surrounding the caisson, cf. Fig. 2. Finally, the pipes was cast with concrete.

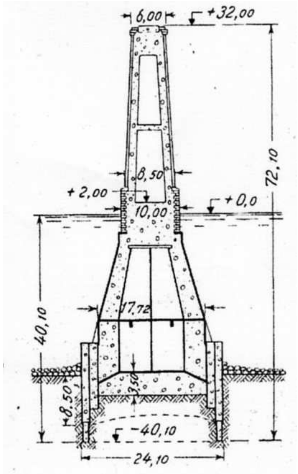

Figure 2. Cross-section of pier 3 [1]. The caisson in the bottom of the pier is surrounded by a wall of pipes

Next, the water was pumped out of the pier, and an approx. 3.5 m high work chamber was dug under the caisson by hand. This work, which despite high external water pressure, was carried out at atmospheric pressure, led to huge settlements due to soil failure below the pipe wall.

L. Bjerrum estimated from back calculations of this soil failure that the undrained shear strength of the clay was of the magnitude of 100 kPa. The work chamber was then cast in concrete, after which the settlement measurement of the pier began. Finally, the part above the sea was cast, and the steel bridge was laid on top of the piers [2].

Table 1. Summary of geometry of pier, geometry of seabed, and forces acting on the soil beneath the piers [1], [3], [4]. The forces address the bottom of the work chamber.

	Pier 1	Pier 2	Pier 3	Pier 4
Foundation area	938 m^2	938 m^2	938 m^2	632 m^2
Soil pressure due to dead load* (kPa)	355	374	388	464
Stress increase due to dead load* (kPa)	326	347	362	437
Pier bottom level	-34,2	-31,6	-31,6	-24,7
Seabed level, N	-30,3	-30,6	-30,0	-19,6
Seabed slope**, N	3,5°	-7,5°	4,5°	9,4°
Pipe wall level, N	-40,1	-39,9	-39,9	-27,7
Seabed level, S	-30,3	-26,1	-31,1	-22,7
Seabed slope**, S	3,5°	0°	-7,6°	-4,0°
Pipe wall level, S	-40,1	-39,1	-39,9	-30,3

**: positive when the seabed rises away from pier.

*: The upward pressure (forces of buoyancy) is deducted.

2 MEASUREMENTS OF SETTLEMENTS

After the work chambers below the piers were sealed up in 1932/33, levels to the top of the piers have constantly been taken in order to measure the settlement. The outcomes in the form of a 78-year unbroken series of measurements are shown in Fig. 3.

During this period, it was repeatedly proven necessary to ballast some of the piers' chambers in order to control movement. Such incidents are marked in Fig. 3, showing that it is these variations in the load that cause most of the minor deviations from the almost perfect straight lines in the figure.

In contrast to the other piers, Pier 1 has an approximately 30-year settlement-free period. But in 1990 (year 57), it suddenly began to settle again at a high rate, while Pier 2 also increased its settlement rate. These events paradoxically coincided with emptying of Pier 1, which until then was the only pier to have been completely full of water.

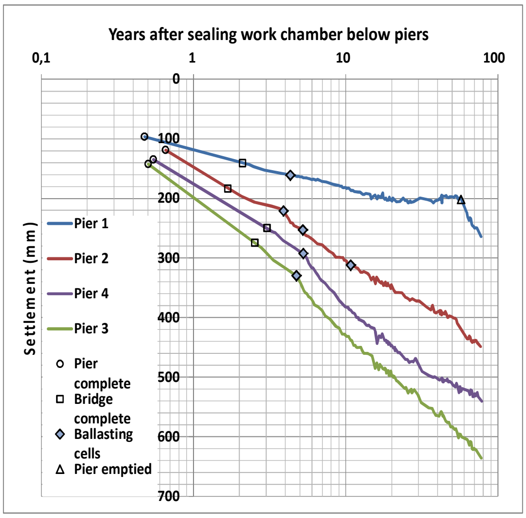

Figure 3. Settlements (mm) of pier 1-4 from 1932 to 2010.

However, there is no plausible physical explanation as to why emptying should result in settlements. On the contrary, emptying reduces the load on the subsoil, while the upward current forces from the water flow that the emptying initiates from the Little Belt and in towards the permeable base of the pier reduces the effective unit weight of the clay.

Another reason must therefore be found for the surprising development in the settlement of Piers 1 and 2. This could be an unfavourable change in the seabed geometry, as a consequence of the strong current in the Little Belt, although the emptying may just have been a triggering factor.

The settlement data in Fig. 3 provides a unique opportunity to test and match our calculation models and our knowledge of the Little Belt Clay's unique properties. It is, therefore, not the first time that geotechnical engineers have directed attention towards the old bridge and its large settlements.

Arne Jeppesen [1] did this in 1948, and in 1963 new investigations were performed in connection with the investigations for a new bridge crossing Little Belt. They included 3 deep borings performed 50 - 100 m from pier 1 and 4 and many oedometer and triaxial tests. On this basis, new strength and deformation parameters were derived and new calculation models was developed [5]. Most recently, new investigations were launched in 2010 in connection with the design of a fixed link across the Fehmern Belt between Denmark and Germany. The results from these investigations are, however, only incorporated in this work to a limited extent.

Investigations on the piers' concrete also started in 2010. These have so far shown that the measurement points are not likely to be substantially affected by the expansion from alkaline silica reactions [6].

3 PROPERTIES OF LITTLE BELT CLAY

The bridge piers are based on Little Belt Clay which is a clay of very high plasticity that was deposited in an Eocene ocean that covered large parts of Denmark and the North Sea. In the Danish Little Belt, the clay was deposited on an undersea bedrock ridge, so that deposit lies relatively high. Later the Ice Age glaciers have removed overlying younger pre-Quaternary deposits, and erosion from the strait has subsequently removed the even younger Ice Age deposits, so the Little Belt Clay is now exposed on the seabed. The glacial processes seem not to have disturbed the clay location significantly. There is thus no sign that the glacial ice has folded the clay or pushed it up in floe deposits surrounded by younger deposits. Deep drilling in the area supports the picture of deposits left in the right place, which means that there is uninterrupted Paleogene clays of very high plasticity to around level -130.

The Little Belt clay is strongly pre-loaded by the weight of the eroded younger layers and by the weight of the many glaciers from the Quaternary period.

As a consequence of the heavy pre-loading, the clay is today subject to horizontal pressure that significantly exceeds the vertical pressure. In the surveys in 1963 L. Bjerrum [7] thus measured an at-rest coefficient $K_0 = 1.9$ approximately 7 m below the level of the seabed.

In addition, the clay is slickensided because it is crisscrossed by old sliding surfaces. The slips might be caused by ice dynamics or release of ice pressure.

Table 2. Indeks properties of Little Belt Clay [5].

Natural water content	38 - 48 %
Plasticity indeks	57 - 210 %
Unit weigth	17,4 - 18,7 kN/m^3

The carbonate content is mostly small, but highly calcareous layers without any apparent regularity have been found (min. 1% and max. 52%).

3.1 *Deformation properties*

In connection with the investigations in the sixties, oedometer IL-tests with 33 tube samples taken from the borings were performed [8].

The tests were typically run in stages up to a preconsolidation stress of 3,000 kPa and hereafter unloaded to the *in situ* stress – i.e. to between 25 and 400 kPa. Finally, the tests were reloaded in steps back to the preconsolidation stress.

The oedometer modulus E_{oed} is determined as the secant modulus on the reloaded curve for stress increases of between 20 and 600 kPa. The results are summarised in Fig. 4, where the derived relation between E_{oed} and the unload stress σ'_{unl} is shown.

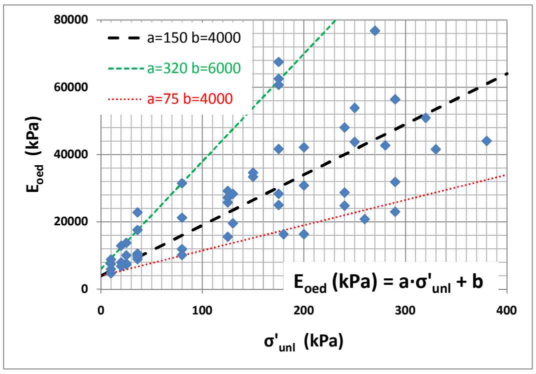

Figure 4. Oedometer modulus measured by IL-tests [8].

The results showed that the smaller increase in stress the higher oedometer modulus, and that there is no significant difference in the stiffness of the clay in the three borings.

The clay's permeability coefficient is measured in the range $5 \cdot 10^{-13} < k < 5 \cdot 10^{-11}$ m/sec [8].

The results of 18 IL tests, where samples were horizontally consolidated, documents that the permeability in the horizontal direction is around the same as in the vertical direction. It is still an open question, however, whether slickenside contributes to a significant increase in the clay's regional permeability.

3.2 *Strength parameters*

The undrained shear strengths were measured with triaxial tests (UU- and $CU_{U=0}$) and in situ vane tests [8]. The results are compared in Fig. 5, and it is seen that they are moderately correlated.

As a rather cautious mean value, $c_u = c_v/3$ is derived, where the vane strength c_v represents the strength of the clay matrix and the reduction is due to slickenside and strain rates.

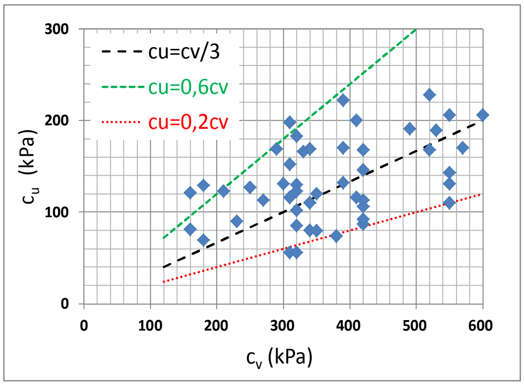

Figure 5. Comparison of vane shear strength c_v and undrained shear strength c_u derived from triaxial tests [8].

Trend lines for the vane shear strengths from the borings represent the strengths before placing the piers:

- Lower bound: c_v (kPa) = 9.86·d(m) + 146
- Upper bound: c_v (kPa) = 9.86·d(m) + 226

where d(m) is the depth below the sea bed. The lower bound strength is measured near pier 4 while the upper bound strength is measured near pier 1.

Also the effective shear constants c' and φ' were measured in triaxial tests (mainly $CU_{U=0}$ and CD) in 1964 [8]. The results are summarised in Fig. 6, where also the derived effective shear strength constants are presented. The derived values are smaller than expected from our general experience.

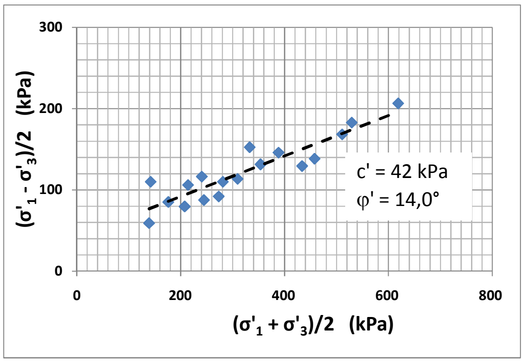

Figure 6. Selected triaxial test results and the derived effective shear strength constants [8].

4 SETTLEMENTS AND CALCULATIONS

The settlement progress is on the whole rectilinear in a log t view, as shown in Fig 3. The settlements and their present rates are surprisingly large – although rather varying despite relatively uniform loads, geometry and subgrade:

Table 3. Summary of settlements 1932-2010.

	Pier 1	Pier 2	Pier 3	Pier 4
Total settlement 2010 (mm)	264	449	636	541
Settlement rate 2010 (mm/year)	3,03	1,83	1,86	0,95
Settlement rate in log t (mm/lct)	64*/465**	156	231	201

*: before year 25 (1958)
*: after year 57 (1990)

The settlements and settlement rates in Fig. 3 and Table 3 are attempted modelled using PLAXIS calculations where, as described in section 5, a traditional Mohr-Coulomb model is applied using the soil parameters from section 3 and forces and geometry from Table 1.

As expected, the calculations show that it is not possible to satisfactorily model the timeline and size of the settlements. It has thus not been possible to clearly identify a consolidation process in the settlement measurements. The timelines seem completely dominated by a creep process. The question now is what this strong creep is due to, and why it varies so much from pier to pier.

5 CREEP AND BEARING CAPACITY

L. Bjerrum [7] assessed that the strength of the clay can be divided into a reliable mechanical element and a less reliable electro-chemical element, where the latter is dependent on the water that binds to the mineral surface. He believed that for as long as the mobilised strength does not exceed the mechanical element, the risk of creep phenomena is minimal. The clay is in the elastic area where the elasticity theory and traditional calculation methods can be used to predict the related deformations.

If, on the other hand, the mobilised strength exceeds the mechanical element, part of the displacement shear strength will be transferred via the bound water, and there will therefore – with plastic clay – be a risk of creep of a dimension that will increase the more the mobilised strength exceeds the reliable mechanical element. The clay passes out into a “semi-plastic” area, where the deformations cannot be determined by the customary methods. On minor overstepping of the mechanical element of the strength there is only yielding at individual points and the supplementary deformations in relation to the elastic deformations are only minor. As the mobilised strength approaches the failure value, the extent of the yielding and plastic deformations increases. In the failure state there is yielding in the entire failure surface, resulting in very large deformations.

As a consequence, the resistance to failure is a vital parameter in calculating the dimensions of the current piers’ settlement. According to D.J. D’Apolonia et al. [9] plastic deformations begin to occur when the failure resistance is less than approximately 2 for the plastic clays.

In order to investigate whether this applies, we have conducted a number of PLAXIS calculations of the piers’ bearing capacity, with incorporation of the presence of pipe walls, slope of the seabed and drainage. The calculations have focused on pier 3. An axial symmetric geometric model was used, applying a traditional Mohr-Coulomb material model with deformation and strength parameters as stated in the preceding sections. The permeability of the concrete is estimated to be equivalent to that of the clay.

The calculations show that the degree of utilisation of the bearing capacity for pier 3 is relatively high in both the undrained and drained condition, with a safety ratio of less than 1.5. As a consequence of the high utilisation of the soil strength, in all of the calculations made there are large zones with plastic points, resulting in plastic deformations, and the settlement rate deviates considerably from the classical consolidation theory (s-shape in log t depiction).

The undrained condition is also investigated with effective strength parameters, which gives reason for a safety ratio of less than 1 (failure) indicating that the parameters are too small. To achieve the same safety ratio as with the undrained strength parameters (lower bound) the effective parameters must be increased to φ'= 15.6° and c'=55 kN/m^2, which corresponds to the design values for the new Little Belt Bridge.

The analyses performed also show that the slope of the seabed is of great significance to the safety ratio. A few degrees' variation in the slope leads to significant changes in the safety ratio, which can explain the very large differences in the piers' settlement. Pier 1, which stands in a hollow, thus has the lowest settlement, while pier 3, which stands with the steepest seabed, has the greatest settlement.

6 CONCLUSIONS

The settlements' straight-line course in a log t– depiction, their rapid appearance and their size indicate that they are primarily caused by some type of creep, and that they are only due to consolidation on a secondary basis.

Calculation of the Little Belt clay's bearing capacity using PLAXIS shows that the piers with the smallest bearing capacity reserve have developed the largest settlements, and the slope of the seabed plays a crucial role.

Calculations of the bearing capacity performed in both the undrained and drained condition with effective strength parameters show that the bearing capacity is smallest in the undrained condition.

In summary, the results indicate that the bridge piers have been relatively close to failure in the undrained condition, so that there has been rapid, strong settlement development as a consequence of plastic deformations. As the excess pore pressure has drained away and the soil has moved towards the drained condition, there has gradually been more resistance to bearing capacity failure, resulting in diminishing plastic deformations [10].

Pier 1 and 2's settlement rates increased drastically in 1990 probably because the seabed geometry was changed by sea currents. But it could, at worst, be due to a reduction of the effective strength parameters against residual values.

7 ACKNOWLEDGEMENTS

Thanks to Knud V. Christensen, Rail Net Denmark, John Frederiksen, Rambøll Danmark A/S, and Femern Bælt A/S for releasing settlement measurements and as-built documentation.

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