The Settlement of a Bridge Abutment on Friction Piles

Le Tassement de la Culée d'un Pont sur Pieux

by L. Bjerrum, Norwegian Geotechnical Institute, Blindern, Oslo
and
W. Jönson and C. Ostenfeld, Skjoldsgade 10, Kövenhavn Ö, Denmark

Summary

The abutments of the Aggersund Bridge were placed on timber piles driven to a depth of approximately 16 m. Below the pile points there is about 15 m of a highly compressible silty clay.

The paper gives the results of the settlement observations from the time of construction in 1940 until 1955. The settlement of the southern abutment is now 80 cm, of which about half is believed to be a secondary time-effect.

The tilting of the abutment is analysed and some conclusions concerning construction of bridge abutments are drawn. A comparison is made between observed and calculated settlements.

Sommaire

Les culées du pont d'Aggersund furent placées sur pilotis de bois battus à une profondeur d'environ 16 m. Au-dessous des pointes des pieux il y a environ 15 m d'une argile silteuse très compressible.

Cette étude donne les résultats des observations du tassement depuis la construction en 1940 jusqu'en 1955. Le tassement de la culée sud est actuellement de 80 cm, dont on estime que la moitié environ est un effet secondaire en fonction du temps.

Le basculement de la culée est analysé et certaines conclusions sont tirées relativement à la construction des culées de ponts. On compare les tassements observés avec les tassements calculés.

Introduction

The foundations of the Aggersund Bridge, Denmark, have already been described (Bjerrum, Ostenfeld and Jönson, 1948).

The present paper is concerned with the settlements of the southern abutment, Fig. 1, which is built on friction piles. The settlements have now been observed over a period of 15 years and the interpretation of the settlement curves shows some characteristic features of this common type of abutment foundation.

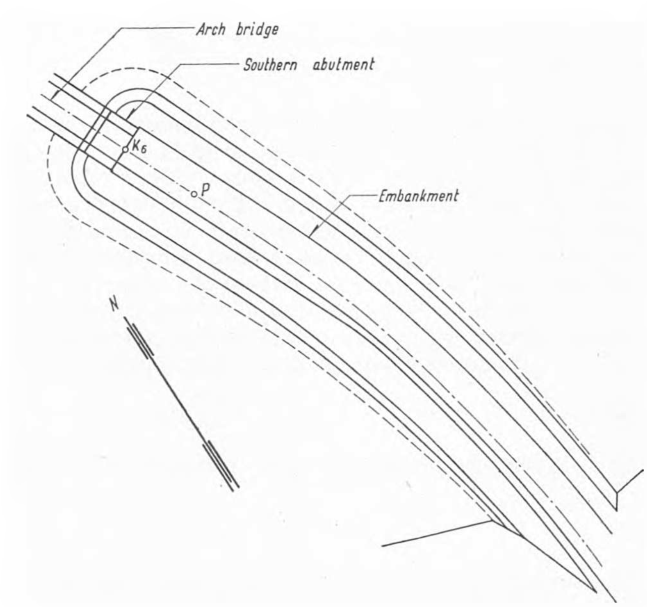

Fig. 1 Plan of southern abutment and southern approach
Plan de la culée sud et de la rampe d'accès sud

Geological Description

A soil profile at the site of the southern abutment of the bridge is shown in Fig. 2. The upper layer is about 7 m thick and consists of an alluvial marine sand with shell debris and organic matter, and stratified with layers of silty clay.

Below this deposit a silty clay is encountered, a post-glacial marine formation from the Littorina Sea. This layer, which is about 21 m thick and is responsible for the settlements of the

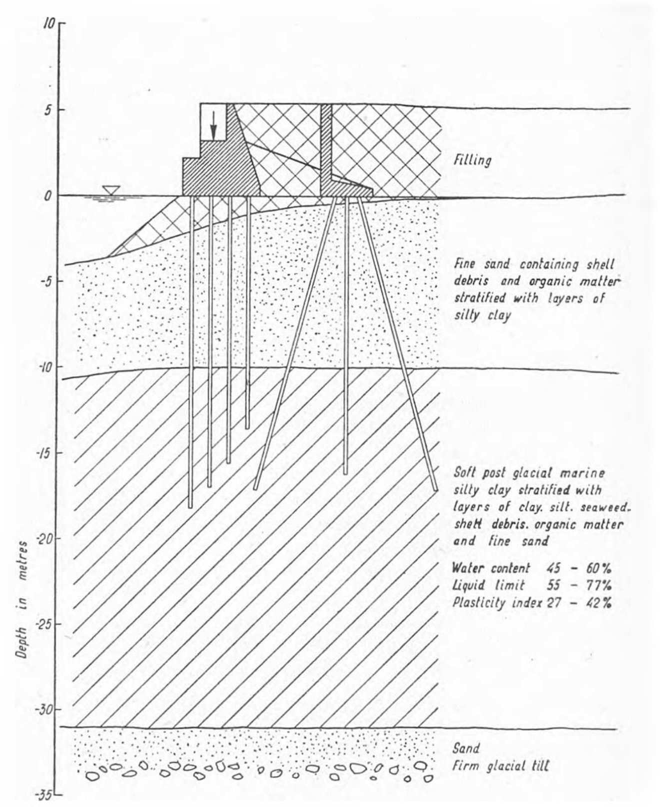

Fig. 2 Longitudinal section through southern abutment
Coupe longitudinale de la culée sud

abutment, is very heterogeneous and stratified with a varying content of clay, silt, seaweed, shell debris, organic matter and fine sand. The water content varies from 45 to 60 per cent, and the liquid limit between 55 and 77 per cent.

At a depth of about 31 m below water level, a firm glacial till is encountered, and compared with the overlying strata this layer can be considered practically incompressible.

Foundation of the Abutment

As driving concrete piles to the firm till proved to be very difficult and expensive, the abutment was founded on timber friction piles about 16 m long. Hence below the pile points there is about a 15 m thick layer of highly compressible silty clay.

In order to distribute the load over as large an area as possible the piles were driven as raking piles with an inclination 1:4.

The reaction from the 90 m long reinforced concrete arch span together with the weight of the abutment amounts to about 1345 ton, and 4 rows of 15 piles were placed in the front of the abutments to take this load (Fig. 2).

The earth pressure acting on the front wall of the abutment is taken back by side walls and anchored in the rear by a pile trestle system, as shown in Fig. 2.

In addition to the reaction at the abutment, the underlying soil is loaded with the weight of the bridge approach, which was built as a 5 to 6 m high earth embankment with a crown width of 11 m. In order to reduce the settlements of the abutment resulting from the embankment, the fill was placed 2 years before construction, and carried 2 m higher than the design height. During the construction period, the embankment height was reduced to the design level.

The settlement of a point, *P*, Fig. 1, on the embankment was measured and showed that at first large settlements were experienced, but when the embankment load was reduced the rate of settlement decreased appreciably, and it can be concluded that the overloading had the desired effect. In the first 13 months the observed settlement was 130 cm, the loading was reduced and in the next 11 months, during the construction period, a settlement of only 10 cm was measured. The final settlement of point *P*, due to the embankment load alone, would have been approximately 150 to 160 cm. The maximum calculated settlement, based on oedometer tests, was 110 cm, an error of about − 30 per cent.

The Pile Foundation

The driving of the timber piles was made with a floating pile driver and a 3-ton monkey. Originally it was intended to drive all piles to 18 m below water level but the resistance to driving

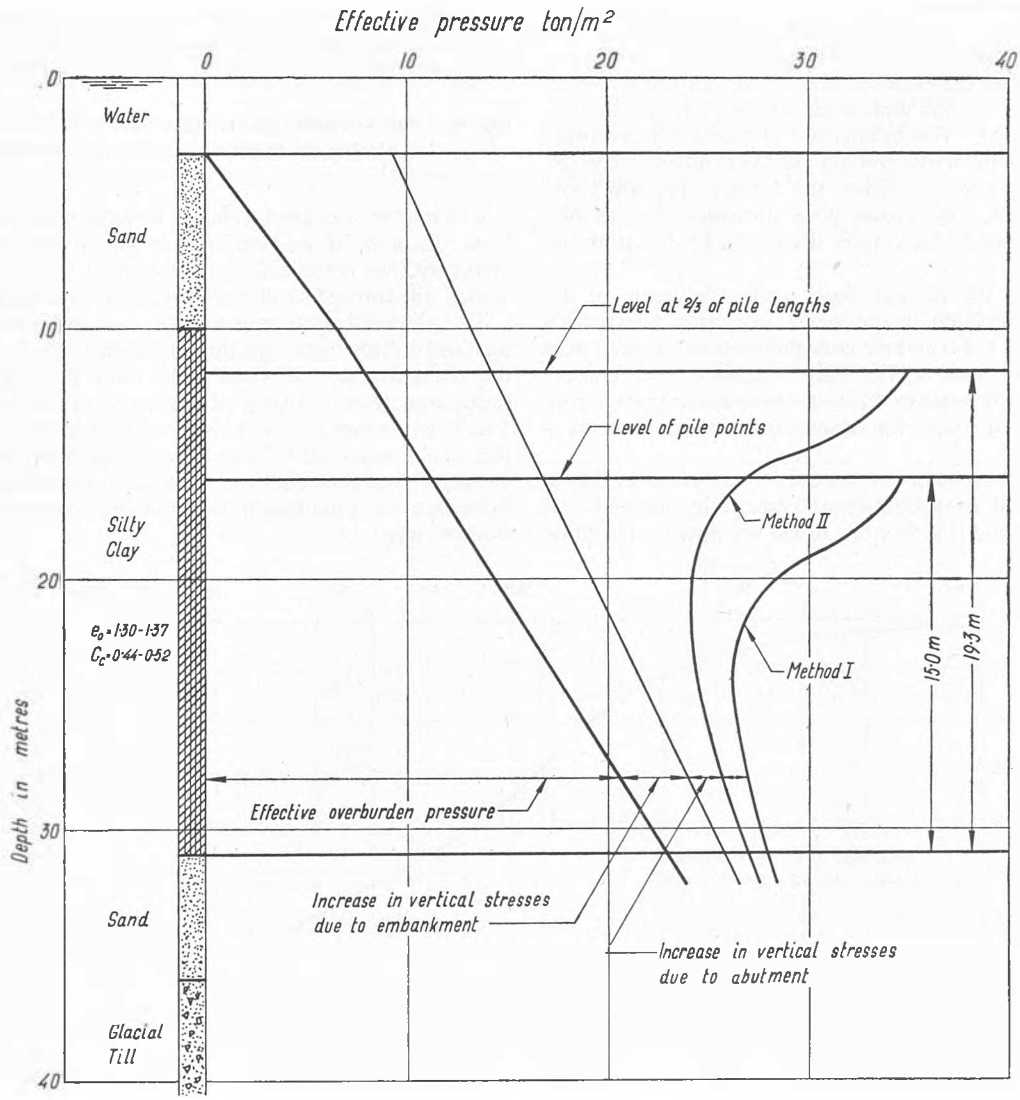

Fig. 3 Settlement calculation data for southern abutment
Connées de tassement calculées de la culée sud

was so great that an average depth of only 16 m was obtained. The piles were re-driven but increased resistance made further penetration impossible.

Loading tests were carried out on a pile driven vertically for this purpose. Immediately after driving the bearing capacity was 97 tons, but after one week it increased to 108 tons. Four weeks after driving the failure load was 130 tons. The major part of the pile bearing capacity is probably due to adhesion between the pile shaft and the soil. This corresponds to an average shear strength of the soil of 6·0 ton/m² (4 weeks after driving).

The average load on each pile, excluding any live load, amounts to about 22 ton: for this load the test pile showed a settlement of approximately 2 mm.

Settlement Calculation and Comparison with Observations

The estimation of the settlement of a friction pile foundation is involved and two suggestions for calculating the settlement of such a foundation are considered. The first, method I, is to consider the whole load transferred to the pile points and then to calculate the settlement assuming the load distributed uniformly at this depth over an area enclosed by the perimeters of the piles. The second, method II, is similar to the above but more conservative, and assumes the load transferred to a depth equal to 2/3 the pile length.

The total pile load is 1345 ton, and since the piles are of unequal length an average length of 16 m is taken. The piles are raking piles, and the enclosed area at the bottom is 74 m², and at a depth of 2/3 the embedded length 11·67 m, the enclosed area is 64 m². For calculation purposes it is assumed that consolidation due to the embankment is complete when the abutment load is applied. Since the timber piles are comparatively pervious, any excess pore pressures due to pile driving are assumed to have been dissipated by the time the load is applied.

The increase in the vertical stress under the centre of the abutment was calculated in the usual way using Newmark's charts (NEWMARK, 1942) and the consolidation settlements were calculated on the basis of Terzaghi's Consolidation Theory. The oedometer e-log *p* curves on the samples taken from boring K_6, Fig. 1, were corrected for sample disturbance (SCHMERTMANN, 1953).

The data for the settlement computation are given in Fig. 3, and the calculated consolidation settlement by method I is 40 cm and by method II is 50 cm. It was not possible to obtain an *E* value for the calculation of the initial, elastic settlement, so a figure of 20 per cent of the calculated consolidation settlement was assumed. This gave total settlements of 48 cm and 60 cm for methods I and II respectively.

For the calculation of time settlement curves, a double drained layer of thickness 16 m was assumed, since it is known that the timber piles will assist drainage considerably. The calculations were made in the usual way according to Terzaghi's theory and the comparison between the calculated and observed settlements is shown in Fig. 4, the observed settlements shown being the mean of four observation points situated at the corners of the abutment.

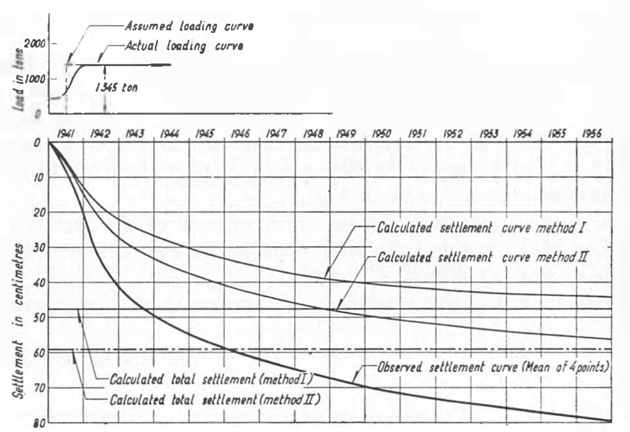

Fig 4 Time settlement curves for southern abutment
Graphiques du tassement en fonction du temps de la culée sud

As a matter of interest it should be mentioned that the vertical front pile used for the loading tests was carried up through the abutment, but completely out of contact with it, and this unloaded pile showed similar settlements to the abutment.

The observed settlement in 1956 was approximately 80 cm; included in this figure are the settlements due to the weight of the embankment. A reasonable value for this part of the settlement, from the time of construction of the abutment, is 5 to 10 cm; hence the total observed settlement of the abutment due to the pile load is 70 to 75 cm, compared with calculated values of 48 and 60 cm from methods I and II respectively. In both cases the calculated rates of settlements underestimate the rate observed.

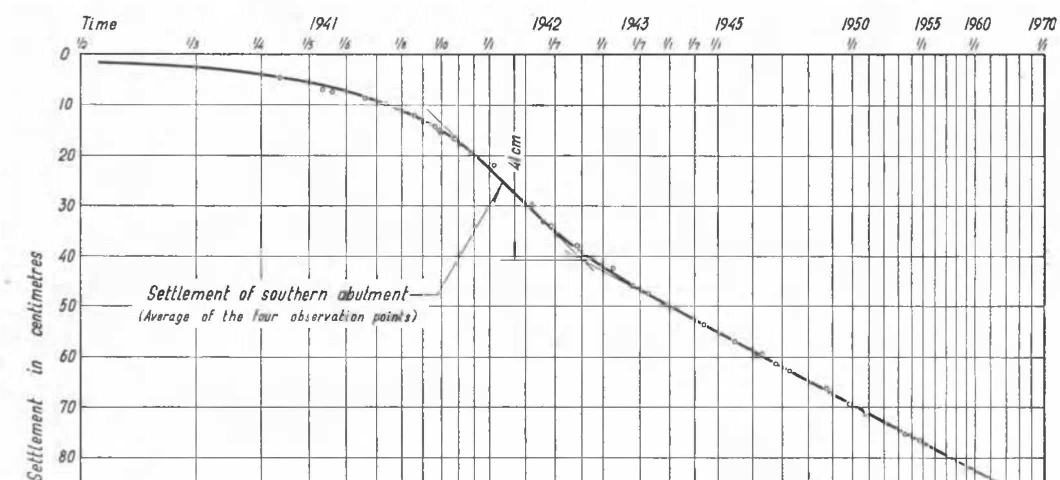

Fig. 5 Log time plot of settlements of southern abutment
Courbe log temps des tassements de la culée sud

A settlement *versus* log time plot of the observed mean settlements is shown in Fig. 5. It can be clearly seen the primary consolidation was completed towards the end of 1942, and that the following settlement, characterized by a straight line plot, is the secondary time-effect.

The total observed settlement may therefore be divided into the following contributions:

Initial settlement and primary consolidation settlement	about 41 cm
Secondary time-effect	about 39 cm
Total	80 cm

Unfortunately insufficient time readings were taken in the oedometer tests, so that it is not possible to distinguish between calculated primary consolidation and secondary time-effect settlements.

It may be mentioned that the abutment was constructed 50 cm too high in order to compensate for the estimated settlements.

Tilting of the Abutments

The differences between the observed settlements of the two front points and those at the back are plotted in Fig. 6. This figure indicates, therefore, the tilting of the abutment in the longitudinal direction of the bridge. The horizontal movement of the abutment was measured from the time the arch span was placed (1–7–1941) and these observations are plotted in Fig. 7.

1–1–1941—1–7–1941—The abutment settled more at the landward side than at the seaward side, i.e. it tilted backwards. This is easily explained by the fact that the stresses in the compressible soil below the pile points, due to the embankment load, are smaller under the front piles than under the trestle system at the back (see Fig. 2).

1–7–1941—The arch was placed on the abutment and instantaneously it turned in the opposite direction. This phenomenon can be explained by considering the stresses in the compressible soil. The reaction from the arch is taken mainly by the four rows of front piles, which are placed centrally under this reaction. The trestle system at the back, which is designed only to take the earth pressure from the front wall, will carry only a very small part of the load from the arch. Hence the stresses in the compressible soil will increase, more under the front piles than under those at the back, and as the two piles groups are connected to a stiff system, the abutment will tilt forward.

1–7–1945—It was necessary to jack up the arch and adjust the roller bearings to the correct position. The horizontal displacements caused by the turning of the abutment was then about 12 cm. At the same time, the steel plates supporting the roller bearings were lengthened.

1–7–1945—1–7–1956—The tilting of the abutment continued and if plotted in a semi-log diagram a straight line is found, corresponding to the secondary settlement of the abutment.

Conclusions

(1) The settlement of the southern abutment of the Aggersund bridge is about 80 cm, 15 years after the construction.

(2) About half of the observed settlement is secondary time-effect, the remainder being immediate settlement plus primary consolidation settlement. In this case, therefore, the secondary time-effect is of considerable importance.

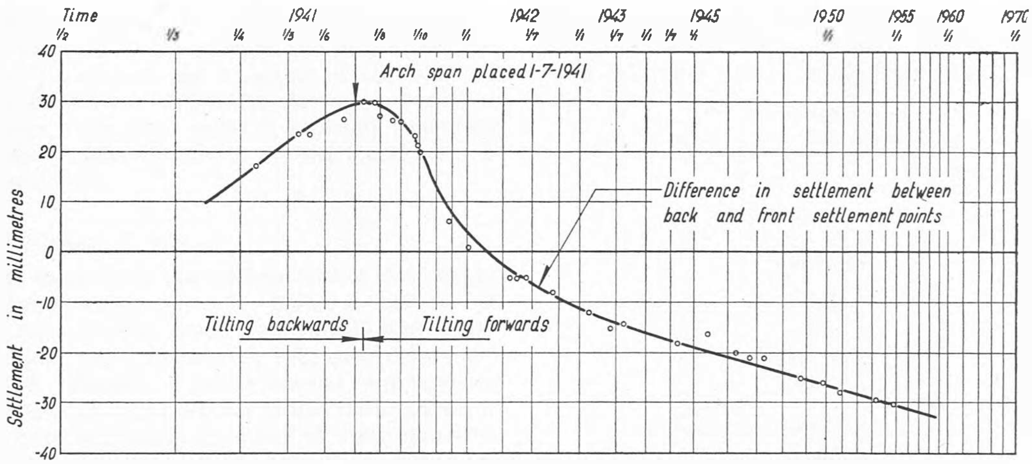

Fig. 6 Difference in settlements between back and front observation points southern abutment
Différences de tassement entre les repères arrière et avant de la culée sud

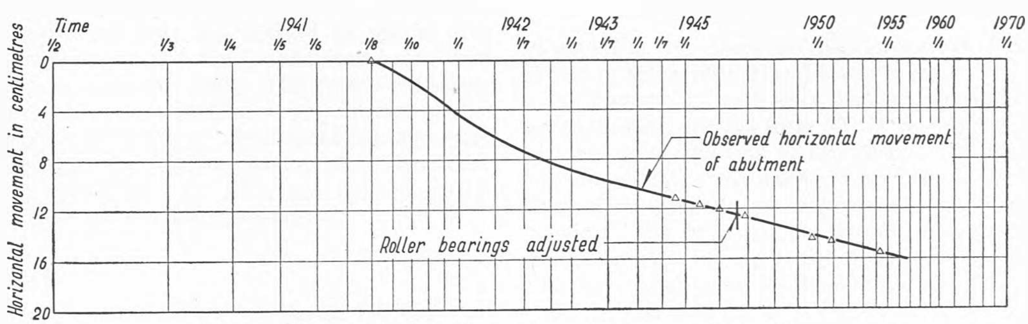

Fig. 7 Observed horizontal movement of southern abutment
Mouvement horizontal observe de la culée sud

(3) The settlement calculations underestimated both the total, the final settlement, and the rate of settlement. The more conservative method of estimating the settlement, i.e. assuming the pile load transferred to a depth equal to 2/3 the embedded pile length, gives a more reliable estimate than the method which assumes that the total pile load is transferred to the pile points.

(4) In spite of the fact that the settlement of the abutment exceeds the height allowance (50 cm) made in the abutment by 30 cm the vertical movement of the abutment has given no trouble to the bridge.

The stiff connection between the front pile group, which carries the bridge reaction, and the rear pile group, which is designed to take the horizontal earth pressure, has, however, resulted in an unpleasant tilting of the abutment. This tilt is not caused by the horizontal component of the earth pressure, but can be explained simply by the difference in stress increase in the compressible clay below the pile points. The bridge reaction is taken mainly by the front pile group, while the rear pile group carries a much smaller load, resulting in differential settlements, and hence a tilt accompanied by horizontal displacements. This proved to be so dangerous for the bridge that it was necessary to adjust the roller bearings.

The authors wish to express their gratitude to Mr N. Simons, Norwegian Geotechnical Institute, for valuable assistance in the re-calculation of the settlements.

References

BJERRUM, L., OSTENFELD, C. and JÖNSON, W. (1948). Foundations for the Aggersund Bridge, Denmark. *Proc. 2nd International Conference on Soil Mechanics and Foundation Engineering*, Vol. 4, pp. 107–112

NEWMARK, N. M. (1942). Influence charts for computation of stresses in elastic foundations. *Bull. Univ. of Illinois Eng. Expt. Stat.*, **40**, No. 12

SCHMERTMANN, J. H. (1953). Estimating the true consolidation behaviour of clay from laboratory test results. *Proc. Amer. Soc. Civ. Eng.*, **79**, Separate No. 311