Negative Friction and Lateral Loading on Piles

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SYNOPSIS An example is first given of a structure founded on piles and having suffered damages caused by negative friction and passive lateral loading on the supporting piles. It appears that the effects of negative friction are in general less disastrous than should be expected from a simplified theoretical approach. The described case shows that, at the contrary, the passive lateral loadings, when not accounted for, become rapidly detrimental. Both influences are time delayed, with the consequence that the critical situation does mostly not occur during construction, but a certain time after completion. The design of a second structure, in which the passive lateral loading and negative friction on the piles were taken into account, and whose construction was successful, illustrated that the passive lateral loading is usually a much more determining factor than the negative friction and even every now and then than the dead and live loads.

GENERAL CONSIDERATIONS

Based on his large experience on pile foundations in the difficult subsoil conditions of Mexico City, Zeevaert (1973) has published an outstanding analysis of the problems of negative friction. By applying the method of Zeevaert, in certain conditions, large values of the negative friction are obtained. In the Netherlands and Norway, some measurements on specially equipped test piles, have shown very large stresses in the lower part of the pile shafts, caused by negative friction.

However, experience shows that damages on constructions resting on pile foundations, for which no or an insufficient account was taken of negative friction, occur less frequently than should be expected, when taking into account the rather larger values of the negative friction, which are expected to exist. When the pile points are embedded in a sufficiently homogeneous layer, in which the resistance to penetration is less than the crushing strength of the pile shaft material, when the total load, including negative friction, becomes excessive, the pile foundation will penetrate into the resistant layer, but because of the homogeneity of the layer, this penetration will be uniform, and as the resistance to penetration is less than the crushing strength, this will happen without deterioration of the piles. As soon as the piles penetrate into the resisting layer, there is a reversal of the direction of the mantle friction in the layers subjected to consolidation, and in these layers the negative friction is temporarily reversed to positive friction. The phenomenon is thus self-breaking.

The conditions outlined above will often occur when the piles are embedded at their lower end in a rather homogeneous sand layer, whose resistance to penetration is generally less than the crushing strength of concrete, or the yield stress of steel. On the contrary, when a pile foundation rests on a rock layer, and especially when some of the piles do not reach the rock, or some are ended on the transition layers of decomposed rock, very large damages can be caused on the construction related to ignored negative friction. It can further be stated that negative friction occuring before the useful load is applied to the pile foundation, can be favourable, as it acts in such cases as a preloading on the pile foundation, and therefore reduces the settlements caused by the application of the dead and live loads on the construction.

On the contrary, the load acting transversally to the pile axis and transmitted by the surrounding soil to the piles, becomes rapidly harmful, if not taken previously into account.

These general considerations are illustrated by two case records.

BRIDGE IN THE MOTORWAY BRUSSELS-OSTEND OVER THE DENDER RIVER

In the motorway Brussels-Ostend, a bridge was built over the Dender river (Fig. 1). It consists of an hyperstatic

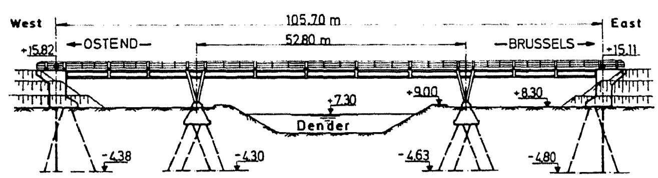

Fig. 1.

prestressed concrete construction founded on 520 mm diameter Franki piles with enlarged bases. The deck slopes slightly down from the Ostend side to the Brussels side. The soil consists of compressible recent and quaternary layers with a thickness of about 12 m resting on the tertiary Ypresian sand-clay formation, in which the piles are founded (Fig. 2). The surface layer, of recent alluvial origin, approximately 5-6 m thick, is composed of very soft sandy and silty clay. The quaternary sand and clay layers are more consistent.

In July 1954 the structure was completed, but next to the abutments, the installation of the approach embankments, approximately 7.5 m high, was not yet started. At the Ostend side, they were practically placed at end January

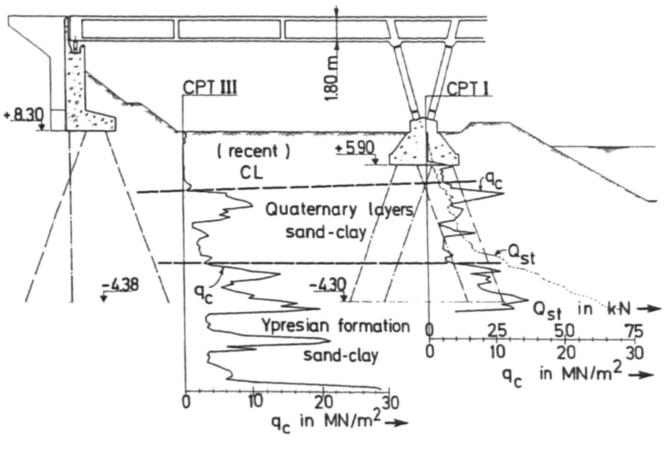

Fig. 2.

1955; at the Brussels side, they were installed between April and end November 1955. The settlements of the structure were surveyed practically from the beginning of the works (Fig. 3). At the completion of the structure, the settlement of the Ostend side abutment was 2mm ; during the embankment placement, it increased up to 49 mm, and

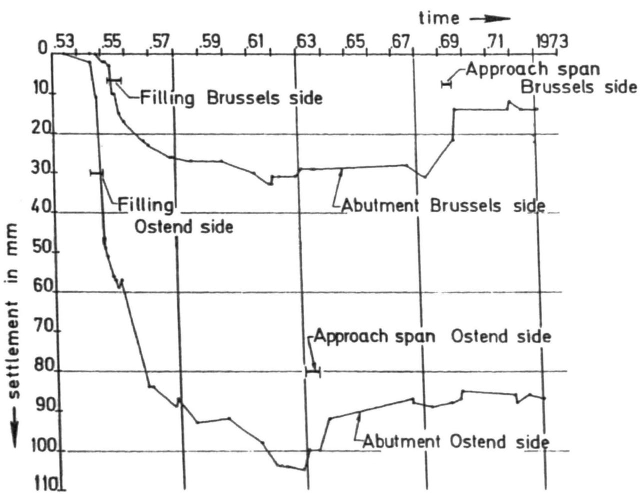

Fig. 3.

afterwards developed at decreasing rate up to 104 mm at end November 1962. The Brussels side abutment showed much less settlements : only 17 mm at the end of the embankment placement and 31 mm at end November 1962. Near the abutments the soil under the embankment settled more than 300 mm.

The horizontal displacements were measured since July 1955. At both abutments, these displacements are larger at the North side than at the South side. Till November 1955, the displacements were relatively low. Afterwards, they developed at a decreasing rate. At the North side, on November 1959, they reached 53 mm at the Ostend side and 44 mm at the Brussels side. The measurements started when the embankment behind the Ostend side abutment existed already since 6 months, and was only in construction at the Brussels side. Hence, the actual displacements the Ostend side were appreciably larger than these at the Brussels side. Simultaneously, the abutments showe a forward rotation at the Ostend side and a backward r tation at the Brussels side. The bridge deck is seated each abutment by means of reinforced concrete self-ali ning bearings. Relatively large movements of the abutm railing related to the bridge railing appeared at the Ostend side.

From the still increasing vertical and horizontal move ments of the abutments, particularly at the Ostend sid fear did arise that the piles were not able to follow the movements.

Half 1955, the head of one pile below each abutment wa digged for inspection. No fissure was observed at that time, and therefore no safeguarding measure was consid red necessary, except the readjustment of the Ostend s de self-aligning bearings. The movements of the abutments went on with time (Fig. 3). Approximately in 195 at the Ostend side, fissures became apparent in the su perstructure of the bridge, notably in the main beams. In September 1961, three batter pile heads were digged below the Ostend side abutment. The backwards battered pile shows a traction fissure passing completely acros the pile (Fig. 4a) and the two forwards battered ones were crushed in compression (Fig. 4b). Moreover it oug

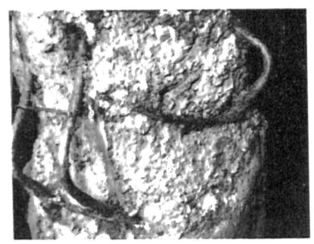

Fig. 4a.

Fig. 4b.

to be feared that the piles were also broken at a certain depth in the soil. Indeed, as soon as the negative bending moment at the pile head becomes larger than the bending resistance of the pile, a plastic hinge is formed at the pile head, the positive field moment suddenly increases and becomes also larger than the plastic hinge moment, so that the pile breaks at a certain dept in the soil. For the backwards battered piles, an additional reason is that the bending moments, due respecti vely to the horizontal soil displacements and the large soil settlements, must be added.

In order to safe the bridge, it was decided (Fig. 5) to underpin the abutment by Miga piles, to take away the em bankment over a length of 23 m and to build an approach span resting at one end on tube piles and at the other end on a floating abutment resting on the embankment. Both the Miga piles and the tube piles consist of steel tubes, 245 mm diameter, 8 mm thick, closed at the botto end. The Miga piles were jacked in elements from below the existing abutment, and the tube piles were bottom driven from the motorway surface. This work was entrust ed to Franki. At the Ostend side, the safeguarding measures were carried out mainly between February and Augu 1963. They caused a reduction of the settlement of the abutment of approximately 15 mm (Fig. 3). On the other hand, the earlier forward rotation of the abutment sligl ly increased. The stability of the Brussels side abutme appeared less precarious. This is partly due to the backward rotation, which results in a relief of the bending of the piles, while on the contrary, the forward rotation of the Ostend side abutment had resulted in an encrease

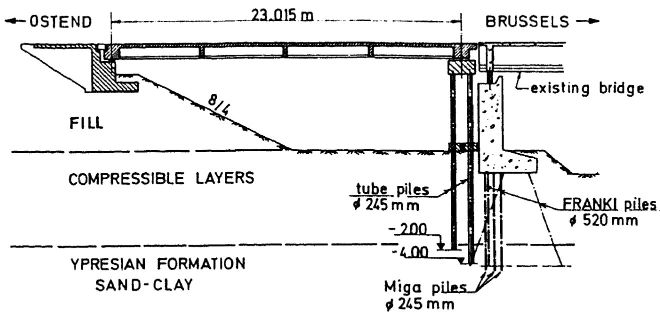

Fig. 5.

of the bending of the piles. Nevertheless, for safety reason, the consolidation measures were also applied on the Brussels side. The works were carried out mainly between March and July 1969. Here too they caused a reduction of the settlement of the abutment of approximately 15 mm (Fig. 3), and also a slight increase of the rotation of the abutment in the same direction as earlier, i.e. backwards.

That the rotations of the abutments increase in the same direction when unloading or loading the soil in their vicinity, points to the fact that the direction of the rotations is essenstially governed by the even slight slope of the bridge deck. The movements of both abutments rapidly stabilized after the consolidation measures were applied.

This example shows :

1) The most critical situation for passive lateral loading is not immediately after placing the embankment but after a certain delay, which can be one or several years.

2) The Ostend side abutment settled some 100 mm, the Brussels side one some 30 mm and the embankment more than 300 mm on both sides. Consequently, the parasitic loading due to the negative friction on the piles must have been large. The observed movements and plastifications show that this loading was however not the most detrimental factor in the behaviour of the piles.

3) The horizontal displacements of the abutments of several ten millimeters limited at first the parasitic bending moments due to passive lateral loading. After some months however, at the Ostend side abutment, the horizontal displacements and forward rotation became too large, causing bending rupture of the piles. The large settlements of the soil favoured the bending rupture at some depth of the backwards battered piles.

In this example, neither negative friction nor passive lateral loading were previously taken into account by the designer. From both influences, the passive lateral loading appears to have been the most detrimental. Would negative friction and passive lateral loading have been considered and assessed with the methods, which were only published later than at the time of the design of the bridge (De Beer 1966, 1976 ; De Beer and Wallays 1968, 1972), some modifications would have been brought to the design. No backwards battered piles would have been used. Indeed, with these piles, the failure hazard is the largest because the bending moments due to the horizontal and vertical soil displacements must be added. From the assessement of the bending moments, very much more reinforced piles or tubed shafts would have been used to support the abutments.

SUPPORT BEAM OF A LOADING BRIDGE LOCATED IN AN ORE STOCK PILING AREA IN ANTWERP

In 1970 a loading bridge was designed for the handling of series of up to 10 m high stockpiles of ores with a

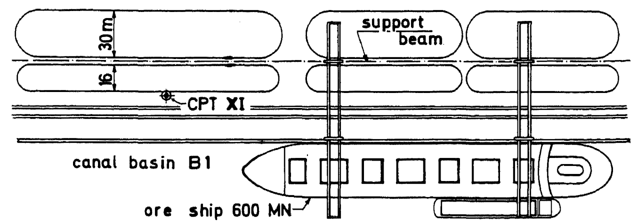

Fig. 6.

unit weight up to 30 kN/m^3 (Fig. 6). Its rear support beam consists of an underground reversed T-shaped beam (Fig. 7a) with pile trestles at 14 m spacings. The alluvial sand, clay and peat surface layer is soft to very soft. Underneath, the tertiary sand layer is dense to very dense.

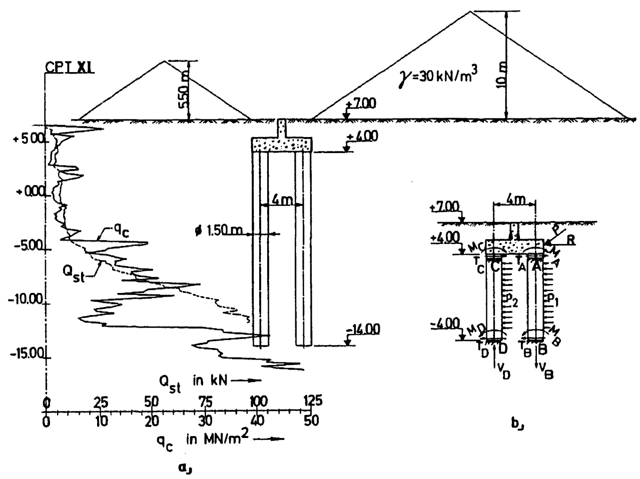

Fig. 7a and 7b.

Under the action of the ore surcharges, not only settlements of the surface layer are expected at the location of the support beam but also lateral deformation of this layer, particularly when the surcharge is acting only on one side of the support beam. Hence, raking piles were excluded. Because of the lateral displacement of the soil, the piles and the support beam work as a frame, and large bending moments are expected. Hence, only two 1,50 m diameter bored piles were used for each support.

Per pile, the maximum live load is only 2.33 MN and the dead load 0.8 MN. The arrangement of the loading bridge is such that no horizontal load is transmitted to the rear support beam.

By assuming that the negative friction on the piles develops down to the lower level of the upper soft soil layers, its value was assessed in accordance with the first author's method (De Beer, 1966) from 1.23 to 1.86 MN per pile, according to the spacing between the support beam and the foot of the stockpiles.

In a first stage, the bending of the piles was estimated by considering the two-dimensional case and assuming that the surcharge was acting on only one side of the support beam. No passive lateral pressure was considered on the downward pile ($p_2 = 0$, Fig. 7b) because this one is sheltered by the upward one. The passive lateral load on the upward pile was assessed to $p_1 = 196$ kN/m from the 1968 authors'method (De Beer, Wallays, 1968). The stresses in the piles were determined by considering that they are embedded at the top level of the sand layer (Fig. 7b). The most unfavourable stresses correspond to the following cases :

in point A, lowest or highest vertical load,

$$V_a = -0.55 \text{ or } +3.64 \text{ MN} \qquad M_A = 4.0 \text{ MNm}$$

in point D, highest vertical load,

$$V_D = 6.44 \text{ MN} \qquad M_D = 2.94 \text{ MNm.}$$

A longitudinal reinforcement of 64 deformed bars, 1.41 inch diameter (area ratio 3.62 per cent), was used in order to limit to 10 N/mm^2 the compressive stress in the concrete and to 240 N/mm^2 the tensile stress in the steel. The pile base level was fixed to el. - 14.0.

In a second stage, the bending of the piles was verified by considering in the tri-dimensional case, successive locations of the stockpile half cone front. In these cases, passive lateral loads are acting on each of both piles of the considered trestle ; moreover, they have components in the transversal and longitudinal directions of the support. The resulting bending moments and maximum stresses in the piles do not differ very much from these obtained in the two-dimensional case, so that no modification was brought to the reinforcement and length of the piles.

In 1970-71, the support beam was constructed and the loading bridge installed. Since then, this equipment is used at the satisfaction of the owner Société Générale des Minerais.

In the design of the support beam trestles, the type, diameter and reinforcement of the piles resulted mainly from the passive lateral loading (e.g. bending moment $M_A = 4$ MNm) and much less from the dead and live loads and also the negative friction. The negative friction had only an effect on the length of the piles. It is clear that a construction such as this support beam can hardly be successful when the passive lateral loading and negative friction on the piles are overlooked.

CONCLUSIONS

The damages suffered by the bridge over the Dender river were mainly due to the passive lateral loading of the piles supporting the Ostend side abutment, which was not accounted for in the design. The negative friction on the piles has had a much lower influence on the behaviour of the bridge. The critical situation did not appear during construction but progressively developed during the first years afterwards. This confirms the time delayed character of such influences.

The design of the rear support beam of a loading bridge in a heavy ore stockpiling area in Antwerp, in which these effects were taken into account, confirms that the passive lateral loading on the supporting piles is usually a much more determining factor than the negative friction. In general, the negative friction on the piles has a detrimental effect only when it results in large differential settlements or in the crushing or yielding of the piles.

Both examples underline how large can be the risks when negative friction and particularly passive lateral loading on the supporting piles are overlooked.

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