Reliability based reassessment of an existing pile foundation

Réévaluation basée sur l'assurance de qualité d'une fondation sur pieu existante

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ABSTRACT: A rational framework for reliability based assessment of the capacity of piles with an existing pile foundation is presented. In this case where traditional approaches for reassessment indicated that a strengthening of the pile foundation with an estimated cost of MECU 5 was necessary, the use of the suggested reassessment framework revealed that no strengthening of the pile foundation was necessary at all.

RESUME: Dans un cadre rationel, l'évaluation basée sur l'assurance de qualité de la capacité des pieux d'une fondation existante est présentée. Dans ce cas, alors que les approches traditionnelles de réévaluation indiquent qu'un renforcement du pont avec un coût estimatif de 5 MECU est nécessaire, l'utilisation de la méthode de réévaluation suggérée révèle qu'aucun renforcement de la fondation sur pieu n'est nécessaire.

1 INTRODUCTION

The bearing capacity of a large concrete motorway bridge, opened in 1969, on the Danish part of the motorway system E45 was reassessed in 1994. The aim was to evaluate the technical and economical consequence of a traffic load increase from load class 40 to 100. The superstructure of the bridge was supported by driven concrete piles with enlarged base. The bearing capacity of these piles was originally determined by a deterministic bearing capacity model, based on the Danish Driving Formula (DDF). This model was calibrated from measured bearing capacities on four insitu pile load tests. As a part of the reassessment, three of the four originally tested piles were loaded with the aim to determine the possible bearing capacity increase of these piles. The load tests showed that the bearing capacity of these piles was increased by 70, 90 and 150%. Based on this new information and all the other information from 1969, new bearing capacities of all the other piles were assessed using the probabilistic methods. First a probabilistic model was established for the bearing capacity of the piles based on the original pile driving records and for the four pile load tests performed one month after the piles were installed. Using this model, the probability distribution functions of the bearing capacities were established. Secondly, the probabilistic model was applied to incorporate the results of the additional three pile load tests - thus updating the probability distribution functions based on the observed pile capacities. Finally it was shown how the use of reliability methods for utilisation of the available information concerning the state of the bridge resulted in the re-classification of the bridge without strengthening or without other modifications of the bridge.

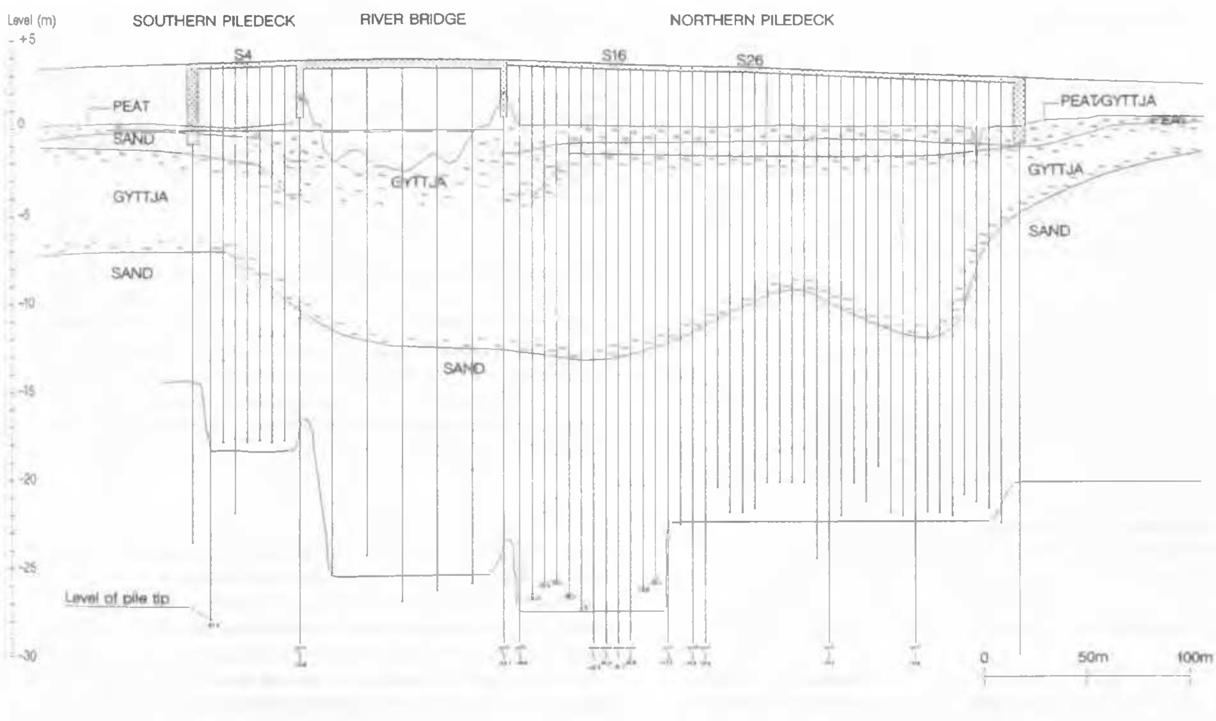

Figure 1. Bridge elevation and geotechnical conditions

2 GEOTECHNICAL INVESTIGATIONS

In 1968, before the bridge was built, geotechnical investigations were carried out to determine the ground conditions including the strength parameters of the soil. In 1994, a 30 m deep boring was carried out as a supplement.

The oldest deposits, found during the geotechnical investigations, are lateglacial meltwater sands. Above these, postglacial deposits of gyttja and sand were found. The transition zone between lateglacial and postglacial layers was difficult to determine. A part of the bridge's elevation and the geotechnical conditions in the area are shown in Figure 1.

Field vane tests in the gyttja show almost constant shear strengths down to level -8 to -9 with an average value of 13 kPa. Underneath, the strength increases slightly, but locally up to 60 - 80 kPa. The results of Swedish weight soundings show that the upper 2-4 m sand layers are very loose. Underneath and further down the sand is medium dense. The characteristic internal angle of friction of the sand is estimated to $\phi = 33^0$ to level -16 m and $\phi = 36^0$ further down. The specific weights of the soil layers is determined to: γ_{gyttja}=12 kN/m^3, γ_{sand}=19 kN/m^3 and $\gamma_{silt/clay}$=21 kN/m^3.

3 INITIAL ASSESSMENT OF BEARING CAPACITIES

In 1969 dynamic driving tests with 37 piles were carried out to determine the bearing capacity of the piles. The piles have variable cross sections and lengths. However, all piles were equipped with a 450 x 450 mm base on their lowest 2m. The bearing capacity of the piles was calculated on the basis of the driving tests, in accordance with the DDF. The driving tests showed greater irregularities in the soil conditions than first assumed, and there was hardly any increase with depth in the bearing capacity below level -15 to -20.

As a supplement to the driving tests, four pile loading tests were carried out. The aim was to check that the DDF could be used directly to determine the bearing capacity. The results from the loading tests were in good agreement with geostatic calculations and the DDF. The results of the pile loading tests are shown in Table 1.

Table 1 Pile bearing capacities in 1969

Pile no	Load test	DDF	Geostatic calculation
S4b	1400-1500 kN	1440 kN	1500 kN
S13b	1100-1200 kN	760 kN	1360 kN
S16c	1200 kN	1220 kN	
S26a	1500-1600kN	1400 kN	1100 kN

The shaft resistance from the extended base on the piles to the ground was not included in the geostatic calculations. As the loading tests were in good agreement with these calculations it could be concluded that the piles were predominantly point load bearing.

On the basis of these calculations the production piles were carried out as 300 x 300 mm piles with a 450 x 450 mm times 2 m base. The piles had to be driven to a characteristic bearing capacity of 1600 kN, determined according to the DDF. Since a part of the piles did not reach sufficient bearing capacity during driving, six piles were subjected to redriving, which showed an increase of the bearing capacity between 7 and 14% only four days after the driving. This increase could be caused by several conditions, such as:

- regeneration of the strength of the gyttja
- penetration of the sand around the pile shaft
- dissipation of the excess pore pressure around the pile base.

4 PROBABILISTIC MODEL OF INITIAL CAPACITY

On the basis of these observations it was reasonable to believe that the piles had gained a higher bearing capacity 26 years after the driving. Now the task was to propose and to validate a deterministic model for the bearing capacity increase of each pile. The bearing capacity model was based on the base resistance, calculated by use of the DDF, and the shaft resistance which were supposed to have increased since the driving as a consequence of regeneration of the soil around the shaft of the piles. The model was based on well-known bearing capacity models which were related to the actual conditions, such as:

- the shape of the piles
- the displacement of the pile in relation to the number of driving blows
- the effective insitu stresses in the soil
- the strength parameters of the soil.

The model was established on the basis of the geostatic formula for the bearing capacity of the piles in the Danish Code of Practice for Foundation Engineering, DS 415, section 6.2, as it was assumed that the formulas in this code for the shaft resistance for ordinary piles also were valid for piles with enlarged base. The used coefficients were however presumed to be lower. For the actual piles the bearing capacity Q can be expressed as:

$$Q_P = Q_1 + Q_2 + Q_3 + Q_4 \tag{1}$$

where Q_{1-3} is the shaft resistances in gyttja, in sand on the shaft part of the pile and in the sand on the base part of the pile, respectively. Q_4 is the pile base resistance. The applied model is illustrated in Figure 3.

The resistances can be expressed as:

$$Q_1 = m\, r\, c_u\, A_{m1} \tag{2}$$
$$Q_2 = q'_m\, N_m\, A_{m2} \tag{3}$$
$$Q_3 = q'_m\, N_m\, A_{m3} \tag{4}$$
$$Q_4 = q'\, A_p \tag{5}$$

where:

- m = material factor, characterising the roughness of the pile
- r = regeneration factor
- c_u = undrained shear strength
- A_m = surface area of the pile in the layer
- q'_m = average vertical effective stress in the soil
- N_m = 0.6, skin friction coefficient
- q' = pile toe unit bearing capacity
- A_p = pile toe area

The shaft and the base area of the piles are known, whereas the strength of the soil along the pile and at the pile base is uncertain. The weight of the soil, which enters into formula (3) and (4), can be determined reasonably exact on the basis of results from the original soil investigations, supplemented with the results from the new boring.

It may be assumed that the base resistance at the time of the pile driving can be estimated from the pile driving records and by application of the DDF. However, by comparison of pile load bearing capacities determined by the DDF and static pile capacity tests performed immediately after pile driving, a discrepancy is observed. This discrepancy may appropriately be described by a systematic term (bias) and a random term (noise).

The relation between the capacity of the piles estimated from the pile driving expressions Q_{DDF} and as obtained from static compression tests Q_P can therefore be given by

$$Q_P = Q_4 = K\, Q_{DDF} + \Sigma \tag{6}$$

where the bias factor K and the noise term Σ are model parameters estimated by the Maximum Likelihood Method.

One month after the piles were installed four static pile compression tests were performed and the results of these tests were used to estimate K and Σ for the present pile capacities. At the time of the pile tests it could be assumed that full resistance is established on the shaft area on the pile base. The relation between the base bearing capacity estimated by DDF and the static test results

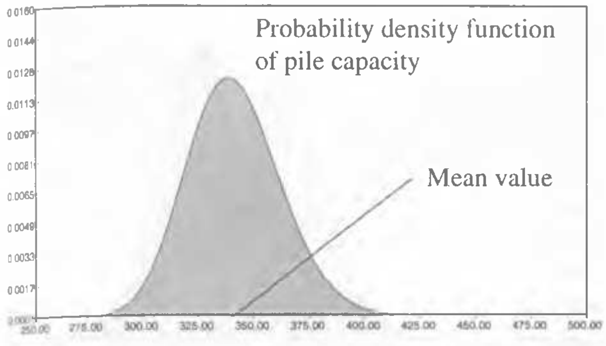

Figure 2. Illustration of the probabilistic density function of the bearing capacity for one of the analyzed piles.

could therefore be given by

$$Q_P = \quad Q_3 + Q_4 \;=\; Q_3 + K\,Q_{DDF} + \Sigma \qquad (7)$$

During the period following the static pile load tests in 1969 it is assumed that all possible pile shaft resistances have been established and the present pile bearing capacity may therefore be written as

$$Q_P = \quad Q_1 + Q_2 + K\,Q_{DDF} + \Sigma \qquad (8)$$

By modelling m, r, c_u, N_m, q´, K and Σ as random variables and fitting the parameters of K and Σ by use of the Maximum Likelihood Method, the probability distribution function $F_{Qp}(x)$ for the piles may readily be determined from

$$F_{Qp}(q_P) = P(q_P \geq Q_P) \qquad (9)$$

which represents the probability that the uncertain pile capacity Q_p is lower than a certain value q_P. The probability density function for one of the piles is illustrated in Figure 2.

According to normal practice, the characteristic pile capacities to be used with the deterministic safety formats, in the classification of a bridge foundation shall be assessed as the 50 % percentile value (indicated in Figure 2) i.e. the mean value of the pile capacity. Within this consideration there is no benefit gained by having an estimated pile capacity with a low coefficient of variation in comparison to an estimated pile capacity with a high coefficient of variation.

In order to support and validate the model it was necessary to carry out loading tests on selected piles. Three of the four piles which had been subjected to loading tests in 1969 were selected. This made a direct comparison with the earlier tests possible. Simultaneously, disturbances from driving of new piles could be avoided.

5 UPDATED STATIC PILE CAPACITY

The results of the load tests were planned in order to verify and update the probabilistic model of the bearing capacity of the piles. The updated probability distribution function for the bearing capacity, i.e. the distribution function of the bearing capacity conditional on the outcome of the experiments **X** may be determined by

$$F_{Qp}(q_P|\mathbf{x}) = P(q_P > Q_P|x_1,x_2,x_3) \qquad (10)$$

where the right hand side represents the probability that the uncertain pile capacity Q_P is lower than a certain value q_P conditional on the observed results from the pile compression tests x_1, x_2 and x_3.

As the increase in the pile capacity was quite significant it was not possible to reach the ultimate pile capacity of all three piles in the tests. In order to gain as much information as possible, also from the tests where the ultimate pile capacity was not reached, a model was established linking the pile head load displacement curve to the ultimate pile capacity. The principle is illustrated in Figure 3 where it is also seen that a random variable was introduced in order to take into account uncertainties in the proposed model.

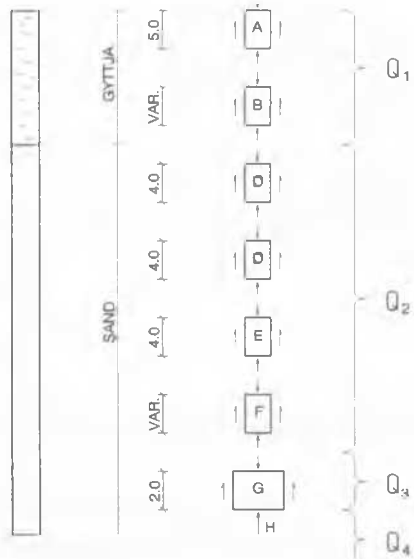

Figure 3. Bearing capacity model for the test piles

Since two of the piles were not loaded to failure, it was necessary to develope a bearing capacity model which could determine the ultimate bearing capacity of the piles on the basis of the incomplete load-displacement curves.

In the model the pile is divided in a number of elements as shown in Figure 3. The model is an iterative calculation model with the following variables: m, r, c_u, N_m and $\delta_{failure}$. The maximum resistance of the piles is calculated from the formulae (2) to (4), inclusive. As shown in Figure 4 it is assumed that the load-displacement curve for the skin friction is linear until yielding occurs at the deformation $\delta_{failure}$.

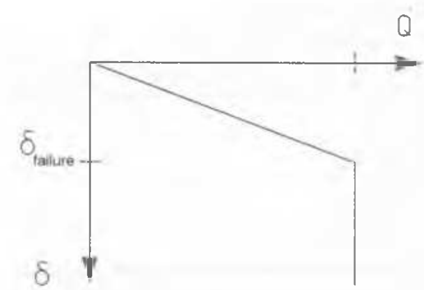

Figure 4. Load-displacement curve for the skin friction

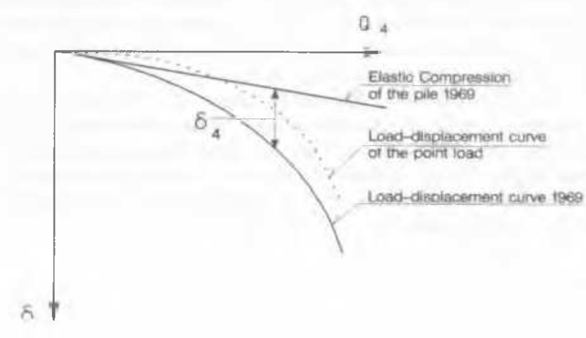

Figure 5. Load-displacement curve for the toe bearing capacity

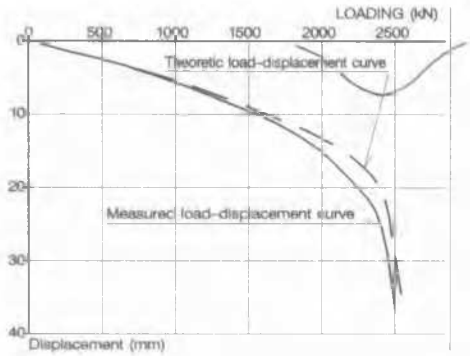

Figure 6. Theoretical and measured load-displacement curves for pile S26a.

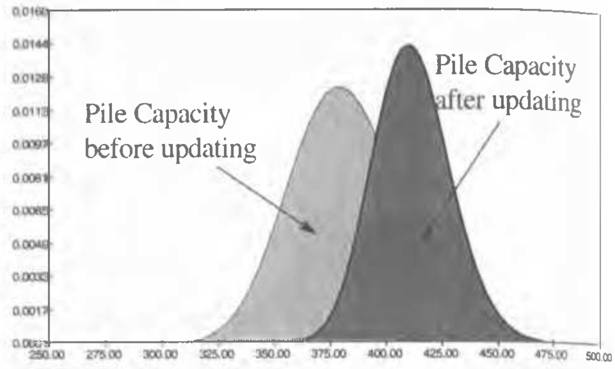

Figure 7. Illustration of the updated probablity density functions for the bearing capacity of the piles.

The measured load-displacement curve from 1969 corrected for elastic deformation in the piles is used as load-displacement curve for the pile toe.

A theoretical load displacement curve for the pile can now be determined. First, suitable values of the three variables are assumed. Then a displacement of the top of the pile is selected. This displacement is distributed on the single elements. The normal force in the pile and the corresponding displacement can now be calculated. The distribution can be controlled by the assumed and calculated displacement which must be the same. Then a new displacement is selected, and the procedure is repeated until the form of the theoretical load-displacement curve can be determined. Then a comparison with the measured curve is carried out. If the theoretical curve does not match the measured one, a new set of values for the three variables must be estimated. Values found by this method are:

$m\, r\, c_u$ = 20 kPa
N_m = 0.45
$\delta_{failure}$ = 5 mm

In Figure 6 both the theoretical determined and the measured load-displacement curves for pile S 26a are shown.

The above mentioned procedure was repeated for the two other test piles which were not loaded to failure. Their failure capacities were determined to

Pile S4b = 3600 kN
Pile S16c = 2400 kN
Pile S26a = 2600 kN

Ahead of each test the probabilistic model of the bearing capacity of the piles was used in order to predict the result of the next experiment by using the equation and the model illustrated in Figure 3. It is worth noting that all the predicted mean values of the pile strengths were within 10% of the test results.

Based on the updated probabilistic model for the bearing capacity of the piles, updated probability distribution functions were established as illustrated in Figure 7. In general the characteristic mean values of the bearing capacities were increased significantly (10 - 20 %) on the basis of the test results.

Using the updated characteristic values (50 %) for the pile compression capacity in the reassessment of the substructure it was found that only 10 of the piles did not meet the requirements for upgrading the bridge to class 100. For this reason it was decided not to use the deterministic safety format but to use the reliability analysis directly instead.

6 CONCLUSION

Applying reliability analyses and new pile load tests to quantify of the pile foundation revealed that the safety was fully sufficient without strengthening any of the piles. The bridge could thus be requalified for use as a class 100 bridge.