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Bearing Capacity Analyses for the Great Belt East Bridge Anchor Blocks

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ABSTRACT

This paper presents a comparison between different methods of bearing capacity analyses: Upper Bound Method. Limit Equilibrium Analysis and Finite Element Analysis. For the Great Belt East Bridge anchor blocks it was concluded that these methods of calculation agree within 5%. However, for cases where the bearing capacity is dominated by frictional materials, much higher differences should be expected.

1. INTRODUCTION

The East Bridge, which is part of the Great Belt Link Project, is a suspension bridge with a 1624 m main span, the longest in the world when completed in 1997. The loads from the main cables are carried by anchor blocks placed in a water depth of 10 m.

Foundation design for these anchor blocks included verification of the bearing capacity by means of three different methods of calculation. A pilot study was carried out to check how the selected methods would predict bearing capacity for large footings supported by different soil types. The results of this pilot study are given in this paper.

Methods selected for the bearing capacity control included:

- Upper Bound Theory
- Limit Equilibrium Analysis (BEAST)
- Finite Element Analysis (ABAQUS)

2. ANCHOR BLOCK FOUNDATION

Each anchor block has a rectangular base of length 121.5 m and width 54.5 m (Figure 1). This base is divided into 3 parts, a front part of 41.7 m, a middle part of 39.1 m and a rear part of 40.7 m. Only the front and the rear parts are in contact with the supporting soils.

Both anchor blocks are to be founded on very stiff to hard preconsolidated clay till. The undrained shear strengths range from 150 to 300 kPa. The lowest part of the

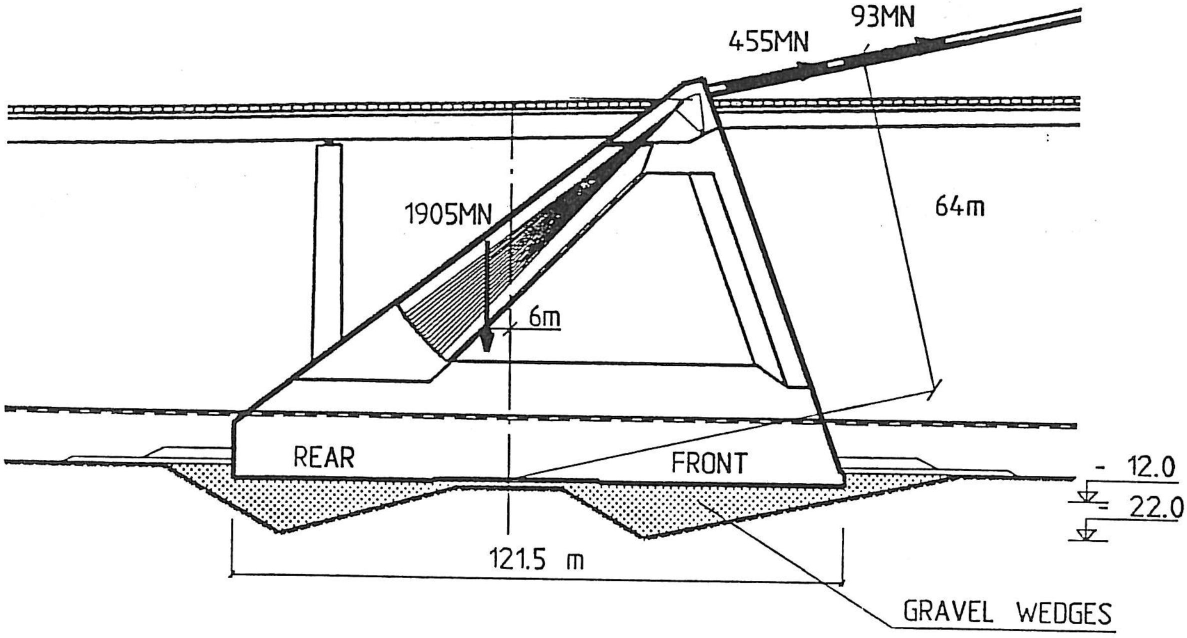

Figure 1. Section of the anchor block.

anchor block is constructed in a dry dock, floated to the position, ballasted into place and the construction completed offshore.

As a result of excavation the top part of the clay till is expected to be disturbed and to have a reduced sliding resistance. This problem is compensated for by introducing a wedge shaped fill of compacted gravel or crushed stone under each of the two pads.

3. ANCHOR BLOCK LOADS

The dead weight of the anchor block is 1905 MN, acting 6.0 m behind the base center line. The two main cables transmit a force to the anchor block of 455 MN from bridge weight and 93 MN from traffic load. This force, acting at an angle of 13°, causes a forward movement and an inclination of the resultant as shown in Figure 2.

Assuming uniform vertical stress distribution against the two pads, the two vertical reaction forces are statically determined.

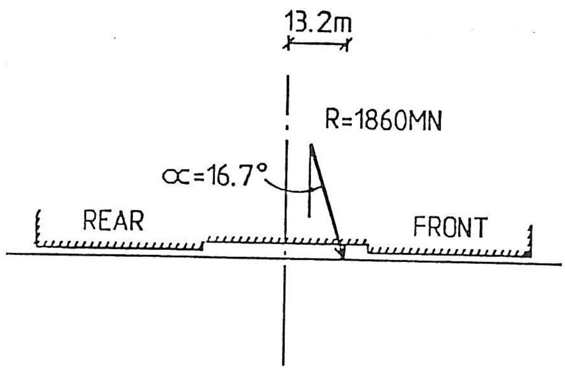

Figure 2. The load resultant acting upon the foundation.

The horizontal shear of 534 MN can be assumed distributed in such a way that the two foundation pads have the same safety against bearing capacity failure. These assumptions are not necessary with the Finite Element Analysis where the total structure can be analyzed and where the load will be distributed automatically. This of course implies that the concrete superstructure has the rigidity and the strength needed to distribute the shear in this manner.

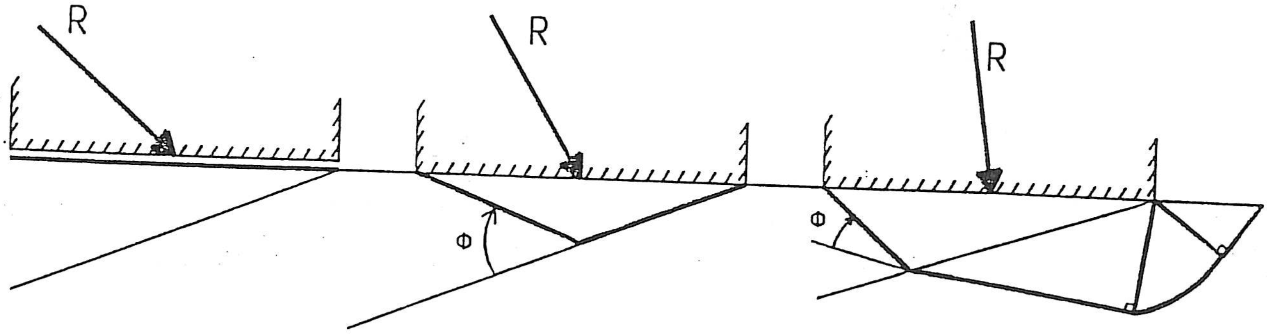

Figure 3. Foundation Pad Failure Modes.

4. FAILURE MODES

Three different failure modes are in principle possible for each foundation pad as shown in Figure 3. Critical mode for a given case will depend upon geometry, soil strength and the inclination of the resultant force. The failure mode that involves sliding along the disturbed clay till surface has been discussed earlier in some detail by Mortensen (1983).

The foundation design, finally adapted for the anchor blocks, used the two load bearing pads, combined with the selected slope of the gravel wedge.

5. CALCULATION TOOLS

5.1 Upper Bound Method

A correct solution for bearing capacity has to be both statically and kinematically admissible. But it is difficult to find solutions which fulfill both conditions. Generally, solutions which are only kinematically admissible give reasonable but non-conservative results.

A kinematically admissible solution is defined by a displacement field which satisfies the boundary conditions for displacement and the flow rule. The flow rule ensures that the work equation can be applied.

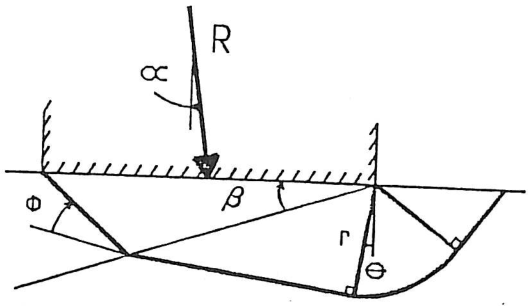

Figure 4. Kinematically admissible rupture figure.

Such a rupture figure for a footing on a gravel wedge is shown in Figure 4.

The internal work IW of this rupture figure is given by the geometry of the rupture figure described by the fixed values B, ϕ and β and the variables r and Θ and the strength c_u of the clay till.

The external work EW of the rupture figure is given by R, α and Θ.

The most critical rupture figure is where the ratio IW/EW is lowest.

The calculations are repeated for the other possible failure modes.

5.2 Limit Equilibrium Solution

The ultimate bearing capacities for the test cases were calculated by the limit equilibrium method using the computer program BEAST. Clausen (1990). This program is based upon the method of slices. Force and moment equilibrium is satisfied for each individual slice. The shear stress along the shear surface corresponds to the Mohr-Coulomb failure criterion.

Figure 5 shows a typical shear surface consisting of a line, a circular arc and a line. The geometry is described by the parameters XC, R and Ψ assuming that the circle center is located at the soil surface. By varying these parameters the location of the critical surface can be determined.

A number of different assumptions for interslice roughness distribution are tried in order to determine the solution with the *highest* safety factor that gives a reasonable position of the line of thrust. Consequently a certain amount of judgment is involved.

The calculations are repeated for the other possible failure modes.

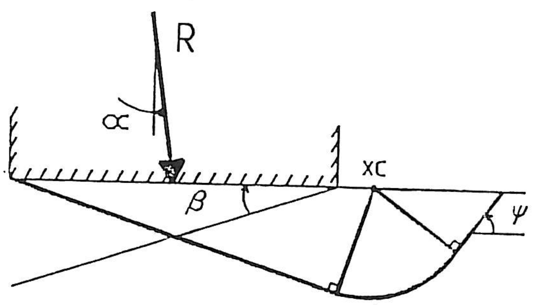

Figure 5. Shear surface used for Limit Equilibrium Analyses.

5.3 Finite Element Analysis

The general purpose non-linear Finite Element Programme ABAQUS has been used for the calculation of bearing capacities of the test cases.

In Finite Element Analysis the geometry of the problem is modelled with finite elements. Each of these elements are assigned a material property. It is therefore possible to model variations in soil strength, for example in the case of a remoulded layer.

ABAQUS provides the choice between various material models. For the test cases the classical Drucker-Prager model with associated flow has been applied. With the Drucker-Prager model it is possible to match the Mohr-Coulomb parameters ϕ and c.

The classical Drucker-Prager model will predict larger bearing capacities than a modified Drucker-Prager where non-dilatant flow is applied. HKS (1992).

Boundary conditions and loads are applied to the model. In the case of a bearing capacity analysis the load is increased gradually until failure occurs. The result of the analysis is hence not only the bearing capacity but also a load-deflection curve showing the behaviour of the footing before failure occurs.

At failure a rupture figure has been developed. It is possible to present the rupture figure by means of plots of the plastic strains in the soil. The calculated rupture figure can be compared with the ones assumed for the Upper Bound Method or in more complicated cases the rupture figure can be a guideline for the Upper Bound Method.

6. TEST CASES

The ultimate bearing capacity was determined for a 40 m wide strip footing subjected to either vertical loading or loading with 15° inclination. For both cases the resulting force was assumed to pass through the footing center, i.e. there are no moments. This footing was supported by the following soil profiles:

- Clay with undrained shear strength 100 kPa.

Case	Load inclination	Soil profile	Bearing capacity, MN/m		
			U.B.M.	BEAST	ABAQUS
1	0°	Clay	20.6	20.3	21.1
2	15°	Clay	14.0	14.4	14.5
3	0°	Sand	216.0	138.0	128.8
4	15°	Sand		57.4	53.6
5	15°	Clay with sand wedge	16.4	17.1	16.8

Table 1. Summary of results from test cases 1 - 5.

- Sand with submerged unit weight 10 kN/m^3 and a friction angle of 30°.
- A combined profile consisting of clay with a 15° sand wedge directly underneath the foundation pad (Figure 6). The strength parameters were the same as given above.

The calculated bearing capacities for these cases are summarized in Table 1 and commented below.

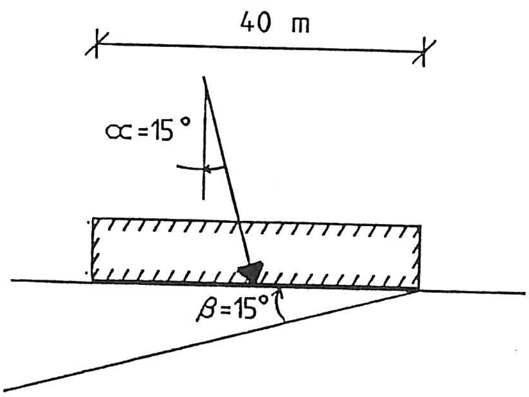

Figure 6. Test case 5, clay with sand wedge.

6.1 Clay Only Cases

For the clay only cases there is very close agreement between the solutions from the three methods. The Upper Bound Method gives the theoretical solution for these cases. The Finite Element solution is 2 - 4% higher than the theory. BEAST is within 2-3% of the theory.

6.2 Sand Only Cases

For sand only cases there is a considerable difference between the solutions. This reflects the difficulties in obtaining a generally accepted solution for a frictional material taking the weight of the material into consideration. The vertical stress at failure is often expressed as:

$$\sigma_{vf} = 0.5 \cdot \gamma' \cdot B \cdot N_\gamma \cdot i_\gamma$$

Values for N_γ and i_γ proposed by various authors, are shown in Table 2. There is a factor of more than 2 between the highest and the lowest solutions. It is believed that both the BEAST and the ABAQUS solutions tend to be on the high side of the unknown correct solution.

6.3 Clay With Sand Wedge

For this case the difference between the highest and the lowest solution is 4%. This reflects the fact that the resulting capacity is governed by the clay strength. The effect of the sand wedge is to increase the capacity by 13 - 19% only.

Source	N_γ	i_γ	$N_\gamma \cdot i_\gamma$	Ratio
BEAST	17.3	0.40	6.92	1.62
ABAQUS	16.1	0.40	6.44	1.51
Meyerhof (1963)	15.7	0.25	3.93	0.92
Brinch Hansen (1970)	15.1	0.35	5.29	1.24
Vesic (1975)	22.4	0.39	8.74	2.05
DS 415 (1984)	14.7	0.29	4.26	1.00

Table 2. Comparison of bearing capacity factors and load inclination factors for $\phi = 30°$ and $H/V = \tan 15°$.

7. ANCHOR BLOCK CASE

An example plane strain anchor block of length 3 x 40 m was analyzed by the three methods. The soil profile under each of the pads corresponds to Test Case 5. Resulting load inclination was taken as 15° with an eccentricity of 5 m towards the front pad.

The calculated resulting bearing capacities were found to range from 32.3 MN/m to 33.8 MN/m, as one would expect from the results quoted above. The corresponding rupture figures are shown in Figures 7 and 8.

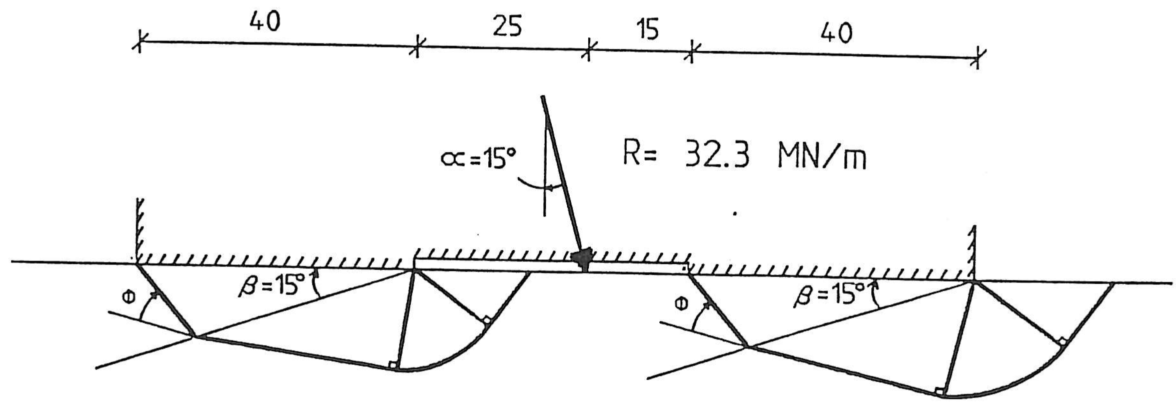

Figure 7. Rupture figure, Upper Bound analysis.

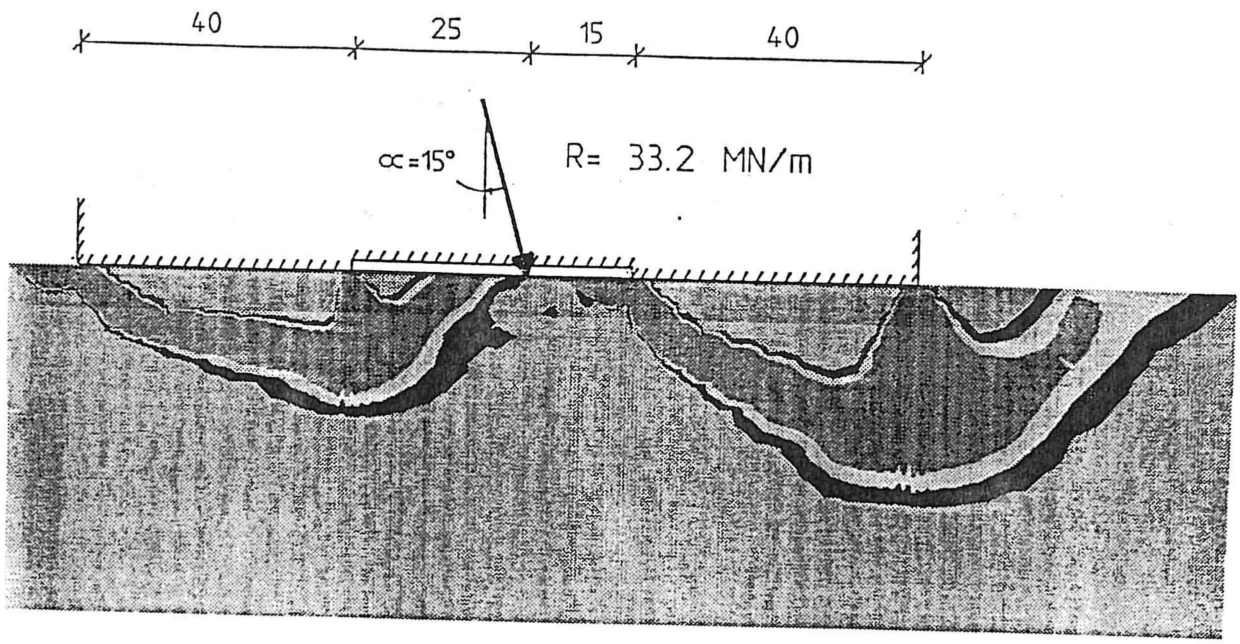

Figure 8. Rupture figure, ABAQUS analysis.

8. CONCLUDING REMARKS

The above results show that as long as the bearing capacity is governed by the undrained strength of the clay till, the differences between the selected analysis methods will be small. The main uncertainty will be linked to the determination of a representative in situ undrained strength of the clay till. This is a demanding and challenging task which falls outside the scope of the present paper.

For cases where the strength of the frictional material dominates the bearing capacity, care will be needed when deciding upon the analysis method to be used. It should, however, be realized that a change of angle of internal friction of only 15% corresponds to a factor 2.0 on the above N_γ factors.

Refined methods for bearing capacity analysis cannot replace the need for an accurate determination of the soil strength parameters applicable to actual cases.

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10. REFERENCES

Clausen, C.J.F. (1990). "BEAST, A Computer Program for Limit Equilibrium Analysis by the Method of Slices". Report 8302-2, Rev. 1, April 24, 1990.

Danish Geotechnical Institute (1985). "Translation into English of Dansk Ingeniørforening's Code of Practice for Foundation Engineering". DS 415. DGI Bulletin No 36, Copenhagen, 1985.

Hansen, J. Brinch (1970). "A revised and Extended Formula for Bearing Capacity". Danish Geotechnical Institute, Bulletin 28, Copenhagen, 1970.

HKS (1992). "ABAQUS. User's Manual" and "ABAQUS. Example problems". Hibbitt, Karlsson & Sorensen, Inc. Rhode Island, 1992.

Meyerhof, G.G. (1963). "Some Recent Research on the Bearing Capacity of Foundations". Canadian Geotechnical Journal, Vol. 1, September 1963.

Mortensen, K. (1983). "Is Limit State Design a Judgment Killer?" The Sixth Laurits Bjerrum Memorial Lecture. Norwegian Geotechnical Institute, Publication No 148, Oslo 1983.

Vesic, A.S. (1975). "Bearing Capacity of Shallow Foundations". Foundation Engineering Handbook, Edited by H.F. Winterkorn and H.-Y. Fang, Van Nostrand Reinhold Company, New York, 1975.