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Autor(en): **Hededal, Ole / Sørensen, Carsten S.**

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Elasto-plastic foundation analysis of ship collision to the Øresund High Bridge

Ole HEDEDAL
Head of Geotechnical Analysis Group
COWI Consulting Engineers & Planners AS
Lyngby, Denmark

Dr. Hededal has extensive experience with numerical analysis using the finite element method. In particular, Dr. Hededal has been involved in development of several linear and non-linear finite element codes for research and educational purposes. He has recently been involved in the design of the Øresund Bridge and in the design evaluation of the Metro in Copenhagen.

Carsten S. SØRENSEN
Chief Engineer, Geotechnics and Foundation Engineering
COWI Consulting Engineers & Planners AS
Lyngby, Denmark

and

Associate Professor
Geotechnical Engineering Group
Aalborg University, Denmark

Mr. Sørensen has been involved in research, teaching and consulting projects for more than 20 years. The work has involved field and laboratory testing on soft and stiff soils as well as design of large bridges and offshore foundations. Recently he has been in charge of the foundation of the Øresund Bridge. He is chairman of the committee responsible for the Danish Code of Practice for Foundation Engineering.

SUMMARY

One of the governing loads on the foundation of the Øresund Link High Bridge is ship collision. In order to assess the the foundation design it was necessary to employ numerical analysis using a 2D finite element model with an elasto-plastic model for behaviour of the soil. This paper shortly presents the limestone material on which the pylons are founded and the constitutive model assumed for the limestone. The applicability and conservatism of the chosen material model was assessed by calibration to medium scale shear plate test. Finally, 2D finite element models were defined to calculate the foundation bearing capacities of pylons subjected to ship collision.

1 INTRODUCTION

The Øresund Bridge is one of the major components in the fixed link between Denmark and Sweden. The fixed link will carry railway and road traffic. It comprises a 3.5 km immersed tunnel, a 4 km artificial island and a 7.9 km bridge consisting of approach bridges and a cable-stayed high bridge. The bridge girders are composite steel-concrete truss girders. The upper deck carries a four-lane motorway and the lower deck carries a dual-track railway. The 1.1 km long cable-stayed high bridge has a free span of 490 m and a navigational clearance of 57 m.

1.1 The pylon foundation

The two high bridge pylons each consist of a cellular concrete caisson of 35 x 37 m^2 and two legs extending to level +155 m (above sea level). The caissons are founded directly on Copenhagen Limestone at level -17 m and -18 m. In order to reach the dimensions of the caisson base a simple model was established on basis of results obtained by use of the finite difference code FLAC, [4]. The model combined vertical bearing capacity of a foundation with a partial mobilisation of the passive pressure in front of the caisson. The preliminary design was then assessed to be in accordance with Eurocode 7, [1], for the ultimate limit state load cases. The accidental load case of ship collision was analysed in a later stage. The geometry and foundation conditions are shown in Fig. 1.

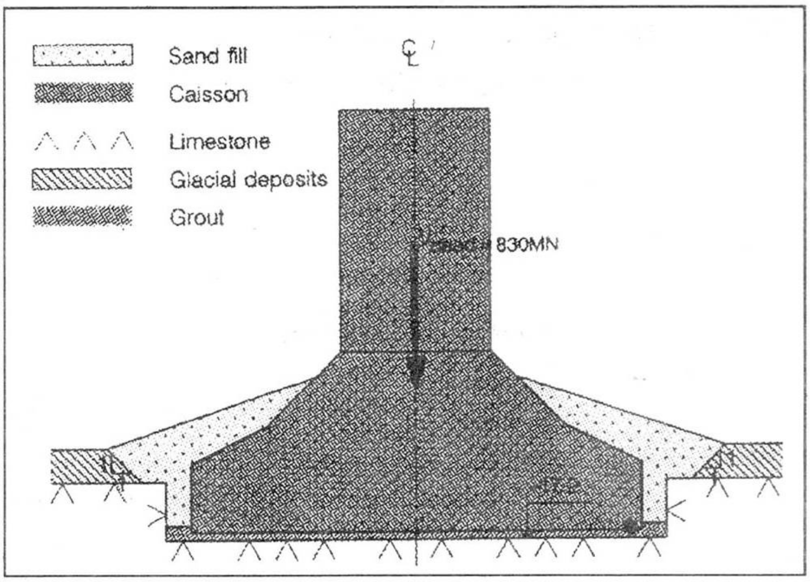

Fig. 1: Geometry and foundation conditions for pylons

1.2 Accidental ship collision

One of the governing loads on the foundation of the Øresund Link High Bridge is ship collision. The design criterion for ship collision consists of both a bearing capacity requirement and a maximum permanent displacement. This, together with the fact that the base plate dimensions were fixed, imposed constraints on the type of model necessary to carry out the assessment. Therefore it was chosen to carry out numerical analyses using a 2D finite element model with an elasto-plastic model for behaviour of the soil. This paper describes the material model used for describing the limestone behaviour, the calibration of the model and finally the quasi-static push-over analysis used to verify the bearing capacities.

2 COPENHAGEN LIMESTONE

The bridge is founded directly on Copenhagen Limestone. The experience with these foundation conditions was sparse and several series of tests (in the laboratory and in test pits) were carried out in order to obtain a better understanding of the limestone behaviour, see e.g. [3]. The test data were synthesised into a principle soil model for the limestone.

The Copenhagen Limestone is a horizontally layered deposit with fissures and layers of flint. Even though the limestone is highly anisotropic, it was chosen to apply an isostropic elasto-plastic model to the limestone. The Owner acknowledged the fact that the available computational models are essentially isotropic. Consequently, he proposed a principle model for the behaviour of the limestone which assumed isotropy.

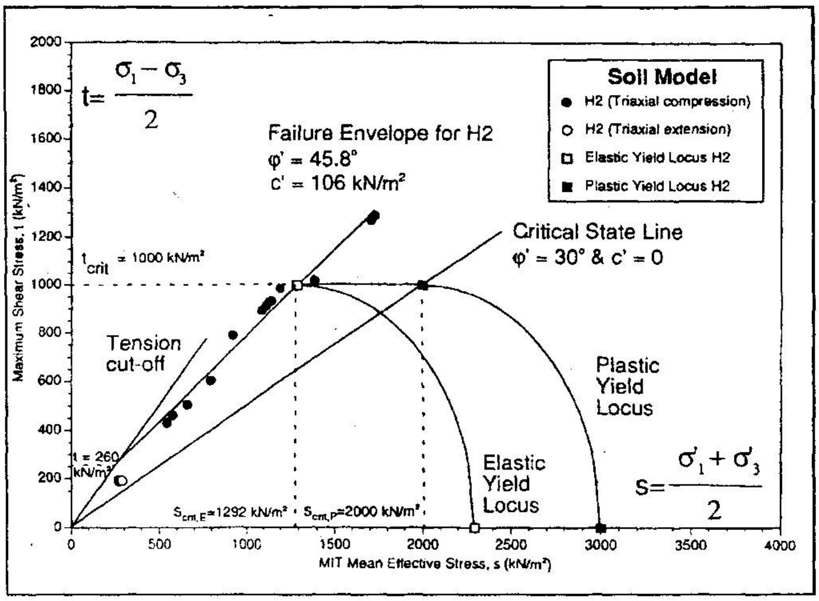

Fig. 2: Principle soil model for Copenhagen Limestone, after [3]

The principle model basically consists of 3 parts: a failure envelope, a maximum shear strength and a critical state line, which marks the transition from compacting to dilating behaviour and defines a friction angle for large deformations, i.e. the residual strength.

2.1 The constitutive model

The most important features of the limestone can be captured by the Drucker-Prager model with cap as defined by ABAQUS, [2], see Fig. 3. The Drucker-Prager yield surface represents the frictional behaviour. The cap ensures that the shear stresses can not exceed the maximum shear strength. The cap was stretched to fit the plastic yield locus shown in Fig. 2. The critical state line corresponds to a reduction of the effective friction angle from a maximum value to a residual value. At the CSL the limit on the shear stresses will in principle disappear. This feature is not captured by the DP model. Still, it was observed during the analyses that the soil did not reach the residual state, so it was not important to model this feature.

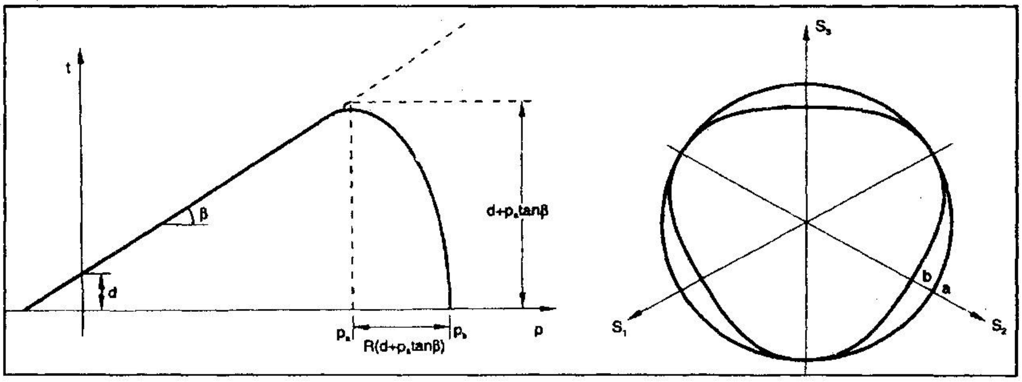

Fig. 3: Drucker-Prager model with cap as defined in ABAQUS, [2]

2.2 Representation of triaxial stress states

A characteristic of the limestone - like most soils - is that the triaxial compression strength is higher than the triaxial tension strength. In an isotropic elasto-plastic model this is reflected by a triangular shape of the deviatoric contours of the yield surface. In ABAQUS a shape correction factor is multiplied to the circular contour, see Fig. 3. For the present problem this formulation was not numerically stable, so it was decided only to use the circular contour.

This decision made it vital to calibrate the Drucker-Prager surface to match the dominating failure mode. For the horizontal ship collision, shear failure was assumed to be the governing case. Calibrating to direct shear failure lead to the following relations between Mohr-Coulomb parameters (c',ϕ') and Drucker-Prager parameters (d,β):

$$d / c' = \sqrt{3} \cos \phi' \qquad \text{and} \qquad \tan \beta = \sqrt{3} \sin \phi'$$

These relations yields highly conservative results for triaxial extension zones, whereas it is only slightly unconservative for triaxial compression, as might be experienced during passive shear failure mode.

3 ASSESSMENT AND CALIBRATION OF THE MATERIAL MODEL

Three types of medium scale tests were carried out in a test pit at Lernacken, Sweden, [3]. The test setups were modelled with finite elements. The three tests represented direct shear, passive shear and active shear. Thus, if the model could capture these 3 modes sufficiently well, the model could with confidence be applied to the large scale problem of ship collision to a bridge pylon.

The calibration strategy was as follows:

- Calibrate to direct shear test
- Verify that the calibration is conservative with respect to other load conditions
- Determine the corresponding Mohr-Coulomb strength parameters

The result of the calibration is shown in Fig. 4. It is seen that the model can represent different preloadings correctly, thus capturing the dependency of a geotechnical material on in-situ stresses.

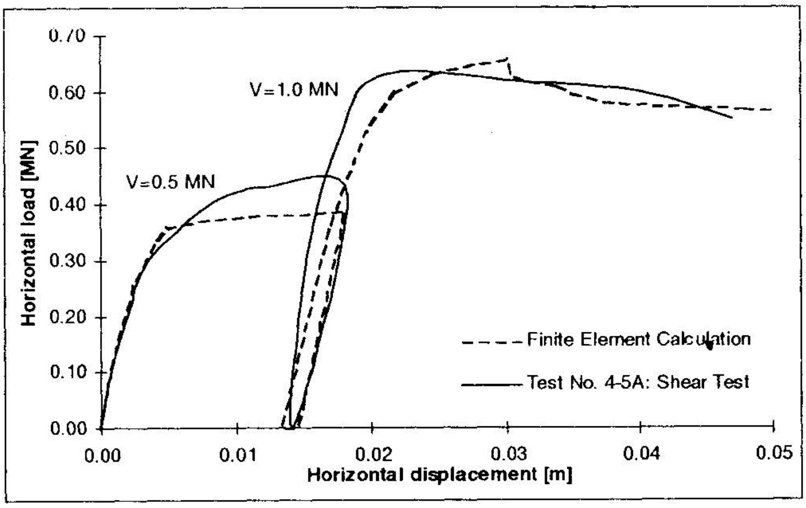

Fig. 4: Calibration to medium scale shear plate test, [3]

The calibrated model was then applied to tests representing passive shear and active shear. The passive failure mode is governed by failure in the weakest horizontal layers of the limestone. Therefore, it was assumed that the calibration would yield appropriate results for the passive failure mode. The assumption was confirmed by the calculation. The active shear failure is more dominated by the crossing of the layers, i.e. the strength of both the weaker and the stronger layers. Therefore an isotropic model should give much lower failure loads than measured in the test. The finite element model actually gave a failure load of only 20% of the measured value.

The final part of the calibration process consisted of showing that the model would yield conservative results, if the prescribed Mohr-Coulomb values were to be applied. The calibrated Drucker-Prager parameters were transformed into Mohr-Coulomb friction angle and cohesion. It appeared that the strength values prescribed in the Design Basis were about 20% lower than the results obtained by direct calibration.

The conclusions of the calibration process are:

- The Drucker-Prager soil model with cap can describe the behaviour of the limestone for load conditions dominated by horizontal shear
- Applying the Design Basis strength parameters to the computational model will yield conservative estimates of the ultimate capacity.

4 SHIP COLLISION ANALYSIS

The calibrated model was employed for a quasi-static push-over analysis of the ship collision problem. A 2D plane strain finite element model was defined. The soil behaviour was modelled using elasto-plastic models. The limestone was defined in terms of the Drucker-Prager with cap, when the calibration had proven that the results would be conservative when using the design values of Mohr-Coulomb friction parameters. The sand and the glacial deposits were modelled by a traditional Drucker-Prager yield condition. The concrete was defined as linear elastic. The stiffness of the caisson was reduced to account for the cellular structure of the caisson. Still, the stiffness of the caisson was much larger than that of the subsoils, so exact determination of the caisson stiffness was not essential. The material parameters used in the analyses are summarized in

Table 1.

Table 1: Material properties used in FE-analysis.

Material	E [MPa]	ν	γ' [kN/m^3]	c' [kPa]	φ' [°]	d [kPa]	β [°]
Concrete	6000	0.2	---	---	---	---	---
Glacial deposits	50	0.3	12.5	35	34	50	44.1
Sand	50	0.3	12.5	0	35	0	44.8
Limestone I	400	0.3	12.0	0	45	0	50.8
Limestone II	400	0.3	12.0	14	33	20	43.3

Limestone II is used for the direct shear zone immediately below the caisson. Limestone I is used for other zones.

The resulting ship collision force acts at a level some 2-6 m above sea level. The position of the resultant is determined by a dynamic analysis of the entire bridge. Thereby it was possible to take into account the dynamic amplification of the maximum static load. The position of the resultant was adjusted to correspond with the overturning moment acting at maximum shear force.

Differences in foundation depth and conditions, and hence in stiffness, implied different failure loads for the two pylons, see Fig. 5. The failure mode due to ship collision to the east pylon mainly consisted of a rotation of the caisson due to a rather high position of the resultant. The bearing capacity is therefore mainly governed by passive shear failure behind the caisson, see Fig. 6. For the west pylon with a deeper foundation level and a shallower resultant, the failure mode was a combination of translation and rotation, see Fig. 7. It is seen that shear bands extend into the limestone in front of the pylon. This gives a larger zone for dissipation of energy, which is partly the reason for the higher bearing capacity.

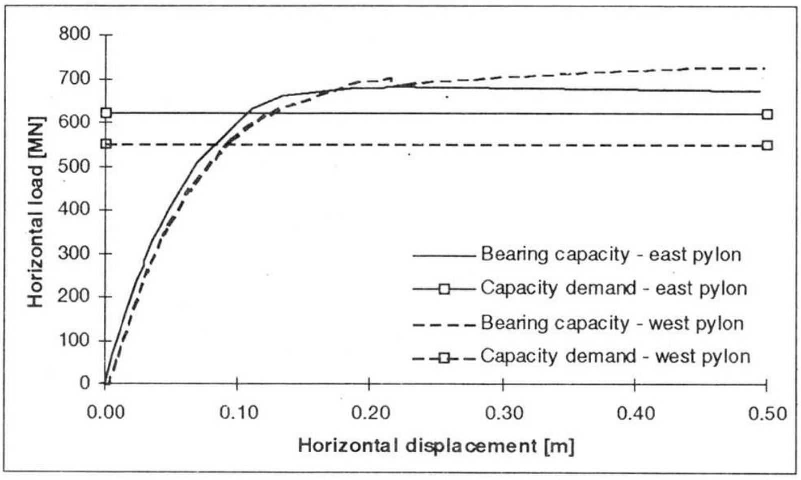

Fig. 5: Load-displacement curve for east and west pylon

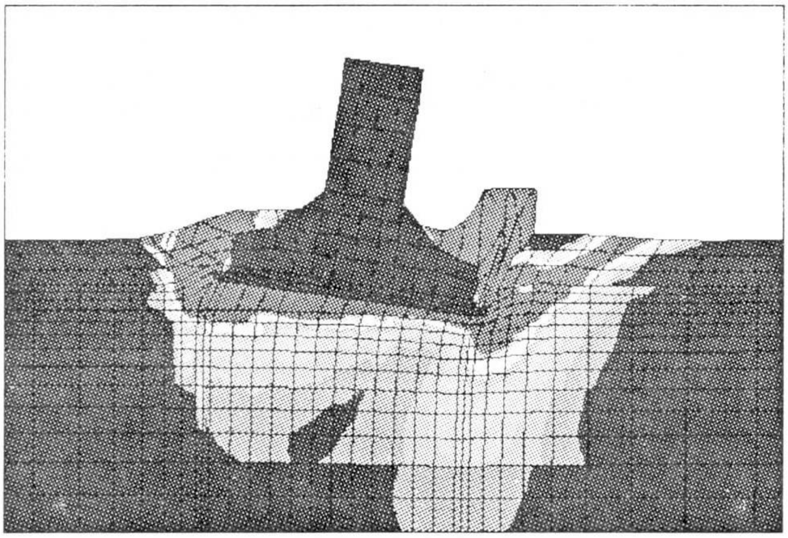

Fig. 6: Failure of east pylon

5 CONCLUSIONS

Ship collision to the High Bridge pylons of the Øresund Bridge has been analysed by means of elasto-plastic finite element models.

The results were presented in terms of non-linear load-displacement curves. They showed that the bearing capacities exceeded the maximum ship collision forces for both pylons.

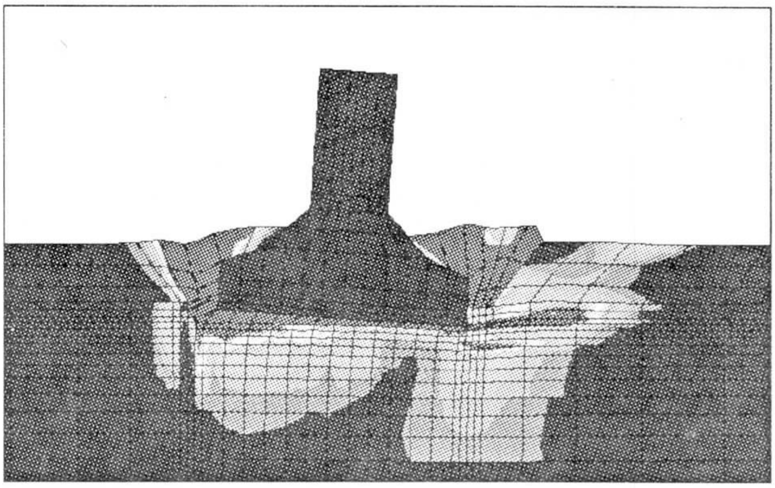

Fig. 7: Ship collision to west pylon

The application of finite element models to geotechnical problems give a number of advantages. Firstly, it is possible to determine both load capacity and displacement capacity in a consistent way - a feature that has become an essential part of modern design practise for large bridges. Secondly, careful calibration of the soil model and demonstration of an appropriate conservatism enables the engineer to design closer to the limit, thus to obtain more economic designs.

The use of constitutive modelling for soils must however still be done with care. Most of models available to the practicing engineer will generally be able to model the bearing capacity with proper precision. The deformation properties are unfortunately not as well described. Using the finite element model to determine the displacements associated with e.g. ship collision can give only a rough estimate. Therefore, there is still much work to be done on the modelling of deformations close to failure.

Studying the dynamic problem of ship impact using a quasi-static model is somewhat dubious. Especially in light of the fact that codes like ABAQUS include a fully coupled porewater-soil skeleton analysis which - in principle - allows for dynamic failure analysis. Still, the limitations in the material model's abilities in describing the volumetric strains close to failure can not justify use of such a complex analysis.

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