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[104]

Foundation of the Øresund Bridge
Fondation du pont Øresund

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Keywords: Bridge, ground investigation, calculation, design, ship impact

ABSTRACT: The Øresund Bridge is one of the major components of the fixed link across Øresund, between Denmark and Sweden. The fixed link will carry rail and road traffic and comprises a 3510m long immersed tunnel, a 4055m long artificial island and a high bridge and approach bridges of a total length of 7845m. The cable-stayed high bridge has a free span of 490m, with a navigational clearance of 57m. The paper presents an overview of the Øresund bridge, the ground investigation strategy employed and the geotechnical design of the bridge foundations. Main features are the extensive utilization of geophysical borehole logging as a tool for ground investigation in limestone, the design of heavily loaded caisson foundations exposed to large ship impact forces and the extensive use of the finite element method in the analyses of soil-structure interaction. Eurocodes (ENV's) are adopted throughout as a basis for the design.

RESUME: Le pont Øresund est l'une des composantes majeures du lien fixe traversant le détroit Øresund, joignant le Danemark et la Suède. Le lien fixe permettra le trafic ferroviaire et autoroutier et comprend un tunnel immergé de 3510m de longueur, une île artificielle d'une longueur de 4055m, et enfin un pont à grand gabarit ainsi que des ponts d'approche sur une longueur totale de 7845m. Le pont haubanné possède une portée libre de 490m, avec une ouverture navigable de 57m de hauteur. L'article présente les caractéristiques générales du pont Øresund, la stratégie d'investigation (géotechnique) employée et le dimensionnement des fondations du pont. L'attention est portée sur la vaste utilisation de forages géophysiques en tant qu'outil d'investigation du sol calcaire, le dimensionnement des fondations sur caissons à haute résistance exposées aux sollicitations dues aux impacts des bateaux, et une utilisation étendue de la méthode des éléments finis pour l'analyse de l'interaction sol-structure. Le dimensionnement est basé sur les Eurocodes (ENV's).

1 ØRESUND LINK PROJECT

The fixed motorway and railway link across Øresund extends just under 16km between Kastrup on the Danish coast and Lernacken on the Swedish coast. The link's key elements are:
- An artificial peninsula extending 430m from the Danish coast at Kastrup.
- A 3510m long immersed tunnel under the Drogden navigational channel.
- A 4055m long artificial island south of the island of Saltholm.
- A 7845m long bridge between the artificial island and the Swedish coast at Lernacken.

The location of the Øresund link appears from Figure 1.

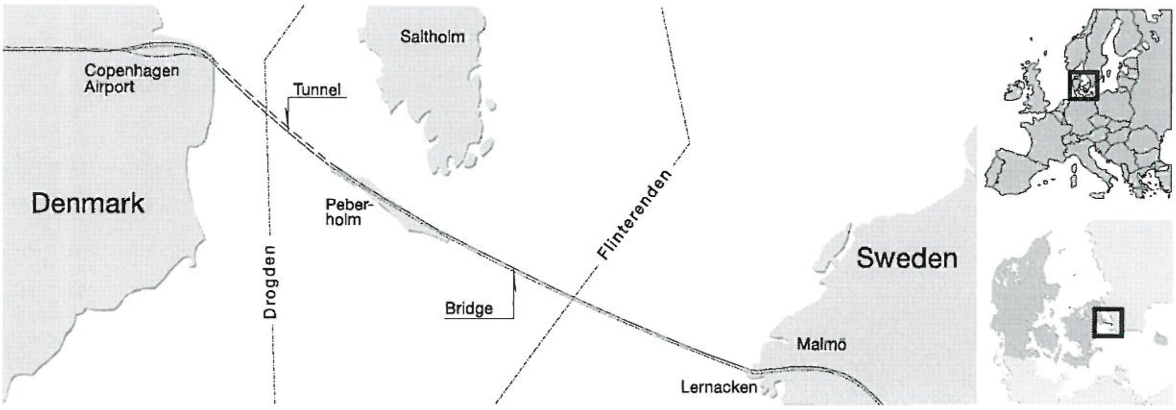

Figure 1: Location plan of Øresund Link

2 ØRESUND BRIDGE

The Øresund bridge is divided into three main components:

- A western approach bridge 3014m long leading up to the high bridge from the artificial island.
- A cable stayed high bridge 1092m long, with a main span of 490m crossing the Flinte navigational channel. The navigational clearance of the high bridge is 57m.
- An eastern approach bridge 3739m long leading down from the high bridge to the Swedish coast at Lernacken.

The bridge superstructure is a composite steel-concrete structure with truss girders. The upper deck contains a four-lane motorway while a two-track railway is on the lower deck. The cable stayed bridge is the largest of its kind in the world carrying both train and motorway traffic. The completion of the bridge is scheduled for year 2000.

Extensive use of prefabrication of bridge elements is adopted for the construction of the bridge. The bridge piers and pylons are founded on prefabricated reinforced concrete caissons placed onto the limestone in up to 9m deep dredged pits, as the caissons are not allowed to protrude above the sea bed. The caissons for the bridge piers are cast in a prefabrication yard in Malmø North Harbour, and lifted out and placed by the floating crane Svanen, which is previously used for the construction of the West Bridge across Storebælt in Denmark and for the Confederation Bridge in Canada. The caissons for the pylons are cast in an existing drydock in Malmø, and floated out to their final position assisted by two large pontoons.

The bridge pier and pylon caissons are initially supported by temporary foundations in the dredged pits, and thereafter the voids between the caisson base slabs and the limestone are filled with cement grout. The principle of the foundation of a typical bridge pier is shown in Figure 2.

3 GEOLOGY

3.1 *Pre-tender ground investigation*

The Øresund is shallow in the bridge alignment, with water depths of generally less than 8m. The soil strata forming the sea bed consists of postglacial sands, generally less than 1m thick, and at a few locations of thin layers of peat or organic mud. These deposits are underlain by 0 to 5m of glacial clay till and glacial meltwater sands.

Prequaternary deposit consisting of limestone of Danian age forms the bedrock in the bridge alignment. The Danian limestone in the Øresund area can be divided into an upper, middle and lower Copenhagen limestone, underlain by Bryozoan limestone, but in the bridge alignment the upper Copenhagen limestone is absent. All bridge pier and pylon caissons are founded in the Danian limestone.

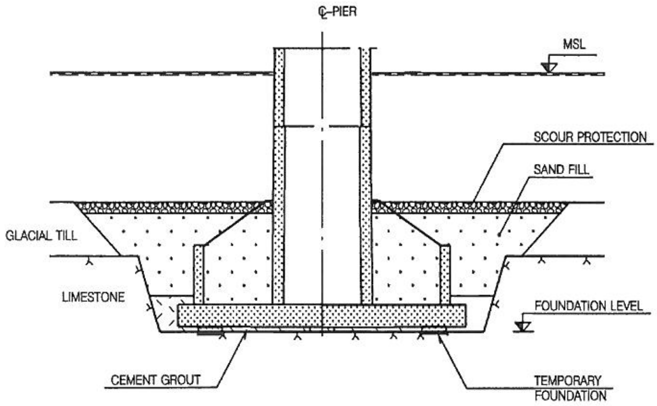

Figure 2: Foundation of Bridge Pier

Prior to tender the Owner carried out geotechnical boreholes in the bridge alignment, supplemented by large scale site trials in the Danian limestone formation at Limhamn in Sweden. To illustrate the properties of the limestone formation the Owner also excavated a test pit in the limestone at Lernacken and made this pit available for inspection by the Contractors during the tender period.

On the basis of these investigations the Owner defined the geotechnical design basis to be adopted for the tender, i.e. soil stratification and soil properties. The results of the pre-tender were entered into a geotechnical database (Geomodel) by the Owner.

A summary of the geology in the Øresund area is given by Knudsen et al. (1995). A geological profile in the bridge alignment, with indication of pier and pylon caisson foundation levels, is shown in Figure 3.

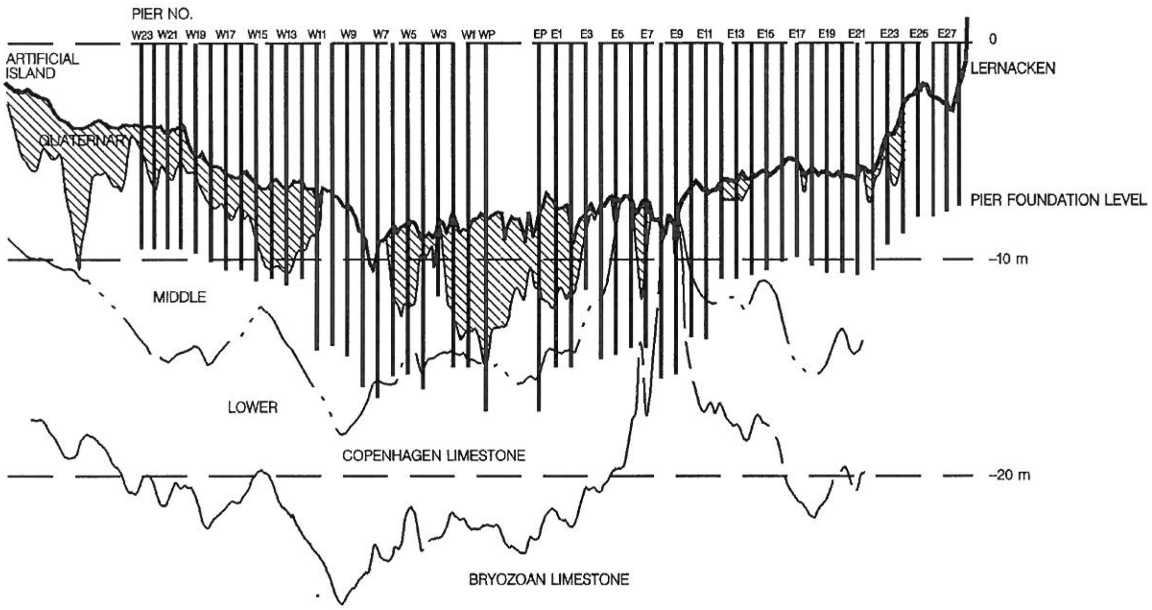

Figure 3: Longitudinal Profile in Bridge Alignment

3.2 *Detailed ground investigation*

For the detailed design of the Øresund Bridge the geotechnical design basis from the pre-tender ground investigations was verified by ground investigations at each pylon and pier caisson position. In these ground investigations particular emphasis was taken to verify that none of the following features existed, which, if present, could have presented problems in relation to the bridge foundation design:

- Highly crushed or fissured limestone.
- Extensive zones of unlithified limestone.
- Cavities due to solution of limestone (karstic limestone).
- Soft sediments deposited in depressions between mounds of bryozoas, especially in the transition between Copenhagen limestone and Bryozoan limestone.

The detailed ground investigations comprised one geotechnical borehole and four soundings at each pier and pylon foundation. The borehole and the four soundings at each pier or pylon position were carried out concurrently from the same jack-up platform.

The geotechnical boreholes were carried out using light cable percussion boring technique in the quaternary deposits and using high quality core drilling techniques (Geobor-S) in the prequaternary limestone formation. Soundings were carried out using down-the-hole hammer technique, with continuos digital recording of drilling parameters (rotation speed, weight on bit, penetration rate and torque).

A suite of geophysical logs were run in all boreholes and sounding holes. These logs comprised caliper, natural gamma radiation, neutron porosity, gamma density, and deep and shallow guard resistivity. In addition a flow log was carried out in all core drilled boreholes. The geophysical marker horizons defined in the pre-tender investigations, Klitten et al. (1995), could generally be traced and correlated in all the boreholes and soundings, thereby facilitating the interpretation of the ground stratification.

No significant occurrences of any detrimental features were found during the ground investigations.

The presentation of the ground investigations followed the requirements of EC7, i.e. a geotechnical design report (covering all pier/pylon foundations) and a number of ground investigation reports (one for each pier/pylon foundation) were prepared. All data were delivered to the Owner in digital form, who entered these into the GeoModel and provided updated versions at regular intervals.

4 STRENGTH AND DEFORMATION PROPERTIES OF THE LIMESTONE

The following deformation parameters, Table 1, and strength parameters, Table 2, for the limestone formation, given by the Owner, were used for the analyses in the serviceability limit state:

Table 1. Deformation parameters for the limestone formation (after Øresundskonsortiet, 1995)

Load Condition	Modulus of Elasticity [MPa]	Shear Modulus [MPa]	Creep Parameter. Height reduction in % per log cycle of time (days)
Unloading	50-100	-	-
Loading	100-500	-	0-0.02
Dynamic Loading	-	500-1500	-

The strength parameters for sliding and passive earth pressure were taken directly from the given values. The reaction from the limestone on the caisson base depends on the strength and on the failure mechanism in the limestone. The presence of possible embedded horizontal layers of thin weak limestone would effect the appearance of this mechanism. A preliminary analysis using the upper bound method showed that the appearance of the failure mechanism corresponded to a normal mechanism for homogeneous soil with an internal friction angle of 45 degrees. This was later confirmed by an ABAQUS analysis.

Table 2. Strength parameters for the limestone formation (after Øresundskonsortiet, 1995)

Load Condition	Peak		Residual	
	φ [°]	c' [kPa]	φ [°]	c' [kPa]
Active Loading	45	100	30	0
Passive Loading	45	0	30	0
Horizontal Shear	33	50	27	0

5 FOUNDATION PRINCIPLES

The main geotechnical challenge was the size of the project and the design of the bridge piers and pylons for substantial ship impact loads. This fact led to very thorough evaluations of the feasibility of different calculation tools for the design. The main issues to be considered were the effect of weak horizontal layers embedded in the limestone and the effect of the limestone's peak/residual strength on the bearing capacity of the pier and pylon foundations.

Shallow direct foundation was found to be the most economical foundation principle due to the very competent limestone in Øresund.

6 DESIGN CALCULATIONS

6.1 *Ultimate limit state*

The ultimate limit state load situation was not governing for the piers, therefore only a simple bearing capacity model was used as verification check. This model disregarded the passive earth pressure on the caisson walls.

6.2 *Accidental limit state*

The accidental limit state load situation (ship impact) was decisive for the design of the piers, except near the coast where the serviceability limit state design was governing.

Due to the large ship impact loads, the effective width of the footings was only 3-4m even with high mobilisation of the passive resistance. Therefore it was decided that the design should be based on more than one calculation tool. The following tools were examined to make a reliable basis for determination of the bearing capacity of the footings:

- Theory of plasticity, kinematically (upper bound) admissible solutions.
- Limit Equilibrium Analysis (BEAST).
- Finite Element Analysis (ABAQUS).

These methods were also used for the design of the Storebælt Anchor Blocks. The result of a comparative study of these were presented in Sørensen et al (1993). Here it was concluded that: *"as long as the bearing capacity is governed by the clay till, the differences between the selected analysis methods will be small. 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."* This warning caused further fundamental calculations as the bearing capacity for the actual footings primarily was dictated by the frictional strength parameters of the limestone. The assessment of the bearing capacity therefore implied a reassessment of the applicability of the three methods for a case with frictional material and strongly eccentric loading.

The theory of plasticity was used to calculate the bearing capacity of the failure mechanism for the foundations. Further this theory was used to evaluate whether it was the peak or the residual strength parameter should be used to calculate the peak bearing capacity of the footing. It was concluded that the peak strength should be used and that the friction angles should be corrected for the influence of dilatancy, Hansen (1996).

The Limit Equilibrium Analysis was used with the same purpose. Two limit equilibrium computer programs were used. The first program, BEAST, was based upon the method of slices. The second program, WEDGE3, was developed as a part of the studies carried out. The results for both programs were comparable with the other methods, Clausen (1997).

The following calculation procedure was used for the ABAQUS analyses:

1. Establish a conceptual model for the limestone based on triaxial laboratory test results.
2. Calibrate and validate the constitutive model using large scale shear tests.
3. Apply the constitutive model to the ship collision problem.

The Owner provided a large number of test results that could be used to calibrate a constitutive model for the limestone. Even though that the limestone was essentially anisotropic due to horizontal layering and fissuring the Owner acknowledged the fact that the available numerical models were isotropic. Therefore an isotropic conceptual model was proposed, see Figure 4.

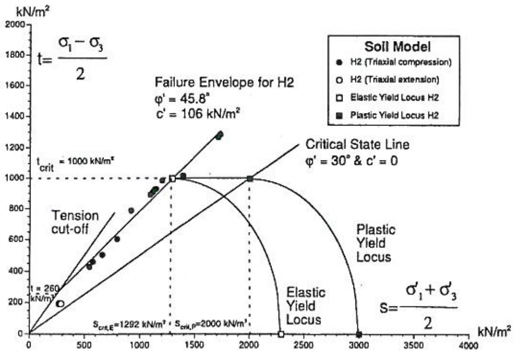

Figure 4: Conceptual model for Copenhagen Limestone (after DGI/SGI 1994)

The most important features of the limestone behavior can be captured by a Drucker-Prager model with cap in ABAQUS, Hibbitt et al. (1996). The Drucker-Prager surface controls the frictional behavior and the cap ensures that the shear stresses do not exceed the maximum shear strength.

The material parameters were calibrated to match large scale shear tests performed in test pits at Lernacken. The calibration was carried out on several large scale shear tests, representing direct shear, passive shear and active shear. The calibration aimed at satisfying a horizontal shear failure mode because the resistance towards ship impact was assumed to be governed by this failure mode, see Figure 5. The calibration showed that the finite element model would yield conservative results when using the strength values given in the Contract.

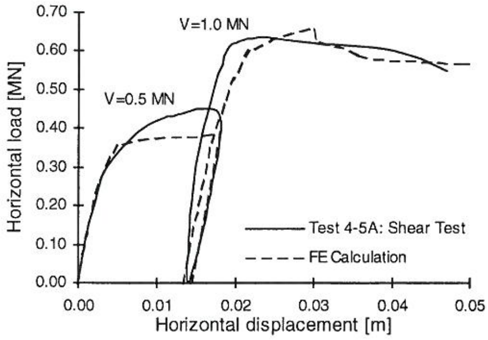

Figure 5: Calibration result for direct shear

The bearing and displacement capacities were verified by quasi-static push-over analysis using a 2-dimensional elasto-plastic finite element model, Hauge et al (1998) and Hededal & Sørensen (1999). The results from the model were used to define line springs and dashpot dampers that represented soil-structure interaction in a dynamic analysis with a global model containing the entire high bridge. An iterative process was carried out in order to obtain consistency between the local and the global model. The failure pattern for the west pylon is shown in Figure 6.

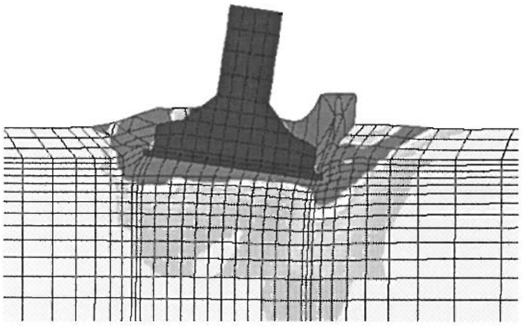

Figure 6 Plastic zones developed due to ship impact

The three methods confirmed that it was possible to utilize very high foundation pressures (3-5 MPa in ALS).

A simple calculation model for the bearing capacity was established based on the results from the three methods. The starting point for this model was the peak capacities of the passive earth pressure (rough wall) and the reaction from the foundation base. The passive pressure was then reduced by 60%. The reason for this large reduction was partly the large difference between the peak and the residual strength of the limestone and that the passive earth pressure was developed later than the maximum reaction force between the base and the limestone.

6.3 *Serviceability limit state*

The geotechnical design criteria in the serviceability limit state was that the stresses in the limestone should remain inside an elastic stress space, to avoid yielding and collapsible tendencies outside the Elastic Stress Space, see Figure 7.

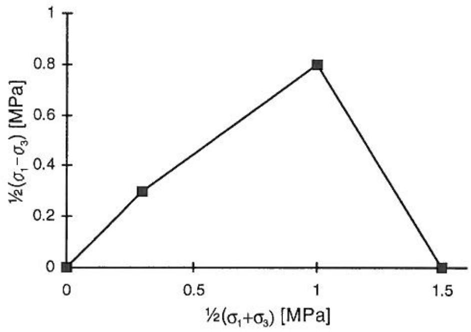

Figure 7: Elastic Stress Space

The Elastic Stress Space defines the stress combinations for which no excessive plastic deformation will occur. All serviceability loads had to fulfill this condition.

The stresses in the limestone immediately below the caisson were calculated using a linear elastic 3D finite element model of the caisson and the subsoil. The model comprised a detailed shell model for the caisson cells in order to obtain the correct stiffness distribution over the bottom slab. The shell model was connected to 3D solid elements, which modeled the foundation soil to a depth of about 100m below the bottom slab.

The condition was satisfied for all interior points under the bottom slab for all load combinations. However, the condition was violated along the edges where the elastic finite element model had singularities. The extent of the zone affected by the singularities was negligible and the violations could therefore be accepted.

7 ACKNOWLEDGEMENTS

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8 LESSONS LEARNED

The experience form the project may be summarized as follows:

- The ground investigations carried out for the tender design and for the detailed design have demonstrated the Danian limestone to be a highly competent limestone.
- A comprehensive geological model is necessary for a proper correlation of even the most detailed and advanced ground investigation programme.
- Modern data handling and statistical methods are required.
- Large scale laboratory tests and in situ tests are absolutely necessary.
- Completely independent geotechnical analyses must be performed for such important and complicated structures, especially when applying modern "black box" computer systems. This is considered the most important quality control measure.

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