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Design of Foundations for the Storebælt East Bridge

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Mr. Pedersen has been involved in design of major bridges and offshore concrete structures for more than 20 years. He has specialised in design of large pylon and anchor block structures in high quality concrete for cable stayed and suspension bridges utilising advanced finite element design models. Mr. Pedersen has been in charge of the concept development, tender design and detailed design for the main bridge substructures of the Storebælt East Bridge.

SUMMARY

The fixed link across the 18 km wide Storebælt Strait in Denmark consists of three major projects; an 8 km bored railway tunnel and a 6.8 km high level motorway bridge across the eastern channel of the Strait and a 6.6 km low level bridge for combined railway and motorway traffic across the western channel. Navigational considerations have been very much influential in the design of the East Bridge as the main navigation route between the North Sea and the Baltic Sea passes the Eastern Channel of Storebælt. Other important design considerations have been environmental aspects like maintaining unrestricted flow of water to the Baltic Sea and not least the aim to establish an elegant bridge perfectly incorporated in the landscape. The paper describes the concept development and final design of the major structural components of the East Bridge substructures in view of these considerations. In particular, the design of the huge pre-fabricated foundation caissons placed on stone beds compacted under water is addressed. The structural calculations comprised advanced models for determination of effects from ship impacts as well as finite element models capable of handling built-in stresses in the structures during all construction phases.

1 INTRODUCTION

The Storebælt East Bridge comprises a major suspension bridge with a free span of 1624 m, the second longest in the world, and approach bridges with spans of 193 m. The total length of the bridge is 6790 m and the navigational clearance in the main span is 65 m, see Fig. 1.

The East Bridge substructures consists of three major structural components

- the pylons,
- the anchor blocks and
- the approach bridge piers.

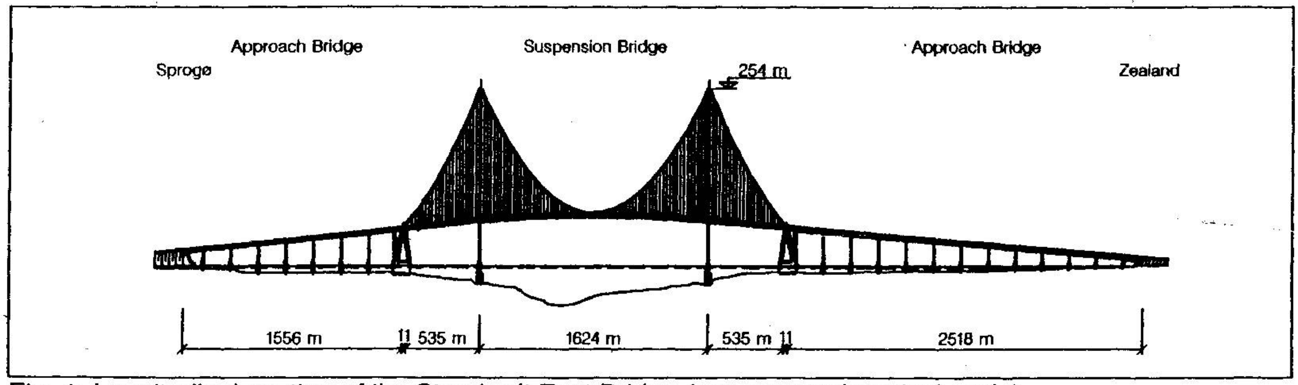

Fig. 1: Longitudinal section of the Storebælt East Bridge (exaggerated vertical scale)

The water depth in the deep part of the channel reaches more than 50 m, but at the positions of the pylons it is reduced considerably to about 21 m. At the anchor blocks and the approach span piers the maximum water depth is less than 12 m.

Generally, the geology in the bridge alignment consists of clay till on top of a marl layer and below that limestone. The clay till has a thickness of 20 to 30 m except at the pylons, where it is reduced to 8-10m . In the deep channel the till formation is missing.

Throughout the initial project phases, it was anticipated that the structures would be founded directly on this upper pre-consolidated clay till strata after excavating a thin top layer of unsuitable late or post glacial deposits.

2 PYLON FOUNDATIONS

As in situ casting of the pylon foundations within de-watered construction pits would obviously not be feasible due to the large water depth and the nearby navigation channel, a construction method was conceived based on placing pre-fabricated concrete caissons on compacted crushed stone beds. Initially these stone beds were assumed to be 1.5 m thick, but as the detailed soil investigations showed considerably lower and more variable strength and deformation parameters of the 8-10 m layer of clay till than expected, it was decided to excavate the clay till and place 5m thick stone beds on top of the stronger and more uniform marl strata.

2.1 Foundation Stone Beds

The stone material used is well-graded 5-90 mm crushed hyperite quarried from Kragerø, Norway. Compaction of the stone beds to the specified 96% of the maximum dry density was performed in two layers of approximately 2m thickness each, leaving a top layer of about 0.4 m which was loosely placed and screeded to provide a levelled surface for placing the caissons. The vibrator plates used for the compaction measured 25 m^2 weighing 75 tonnes. As it was not possible to device proper methods for in situ compaction control, a prescription system was adopted verified by full-scale underwater compaction trials in a test pit established onshore. Large scale triaxial tests on material excavated from the test pit after compaction showed very high values of the triaxial secant angle of friction , phi>50°. In all the tests the material dilated at failure, but the rate of dilatancy was lower than expected. For further information see Steenfelt and Foged [1].

2.2 Foundation Caissons

The foundation caissons, each covering an area of 2,770 m^2 , are 78 m long and 35 m wide, see Fig. 2. However, the corners and the edges are rounded in order to reduce the water blocking effect and to limit damage to possible colliding ships by avoiding sharp corners and edges. They are divided into 60 cells by internal walls and are 20 m high, including a 3 m thick plinth cast in situ on top of the cellular caissons within a temporary steel cofferdam. This was required because the top of the caisson is kept 3.5 m below water level in order to avoid the undesirable visual effect of the slender pylon legs and the voluminous caisson standing one on top of the other.

The bottom slab of the caissons has a variable thickness between 0.95 m and 1.1 m, creating a roof-shaped underside to the slab. Around the periphery of the slab and below some of the internal walls 0.5 m deep skirts are provided, designed to penetrate about 0.3 m into the uncompacted screeded top layer of the stone beds, see Figure 2. The voids thus formed between the roof-shaped underside of the slab and the stone bed were subsequently filled with sand/cement grout through pre-installed pipes, the skirts serving to confine the grout. For each of the caissons the quantity of grout was some 725 m^3, placed in about 40 hours applying a pressure of 8 bar at the inlet pipes.

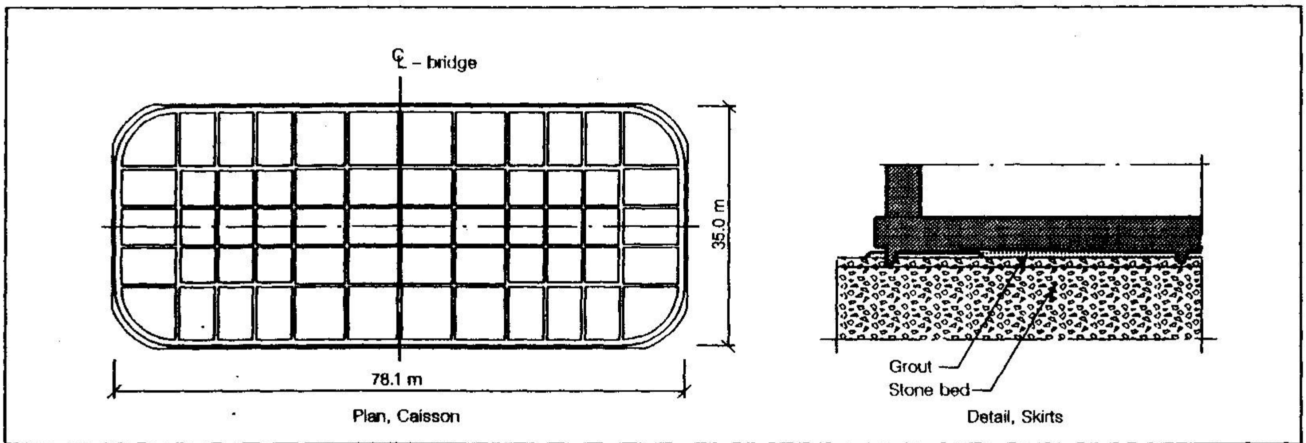

Fig. 2: Plan and Detail of Foundation Caisson

2.3 Design for Ship Collision

Due to the large water depth at the pylon locations, it was not feasible to arrange protective artificial islands around the foundation caissons as this would restrict the free opening for the navigation route too much. Therefore, the pylon foundation had to be designed to resist the impact from a fully loaded 250,000 DWT tanker travelling at a speed of 10 knots (5.1 m/s). This design vessel was selected being the maximum size which can pass the bridge considering the draft limitation of 17.5 m for the navigation route from the Baltic Sea.

The load-time curve for the impact against the non-moving pier was established based on theoretical considerations of the energy dissipated by adding up the contributions during plastic deformation of all basic structural elements of the ships hull. The procedure developed for bow crushing analysis is based on an upper-bound plasticity theory which can also handle rigid body motions of crushed and non-crushed parts of the structure. Validation of the numerically predicted crushing loads has been done by comparison with experimental crushing results of small scale bow models.

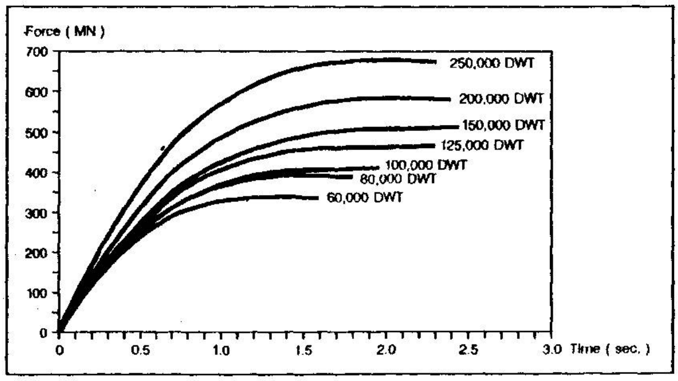

Fig. 3: Idealised load-time curves for ship impacts

In order to resist an impact load of 673 MN without permanent movement a very large effective vertical load on the foundation area of around 2,000 MN is required. This has been achieved partly by filling each caisson with 37,500 m^3 of hydraulically pumped sand being the cheapest weight material available. In this respect the inherently large dead weight of the concrete pylon itself -around 950 MN- is an advantage compared to a steel pylon solution, the lower weight of which would have had to be compensated by a considerably larger caisson with increased ballast material weight.

3 ANCHOR BLOCKS

Most existing major suspension bridges have their anchorages on land in rock formations, which makes the design fairly easy. However, for the East Bridge the challenge for the designers was much greater because the anchorages -subjected to a horizontal force of about 600 MN- had to be constructed on clay more than 2 km from the coast at water depths of approximately 12 m.

It was clear from the beginning that gravity-type anchorage structures would have to be adopted, constructed either by placing pre-fabricated caissons on stone beds or by casting the structures in situ within huge de-watered construction pits. Both concepts were developed for tendering and closely equally priced by the contractors at tendering. However, the pre-fabricated caisson solution was selected.

3.1 Foundation Stone Beds

The soil conditions at the anchor block locations consist of 20 m of clay till on top of a 40 m thick layer of marl. Excavating the clay till under water would disturb its surface and the anchor block could slide along this thin disturbed zone. Comprehensive large-scale field tests and studies to overcome this problem were carried out in the initial project phases. Different solutions for penetrating the disturbed zone by short steel sheet pile walls or large diameter steel piles were proposed as well as several configurations of wedge shaped stone beds. Based on these studies a solution with two individual wedge shaped stone beds without support of the middle section of the caisson was selected for tendering, see Fig. 4. This option was considered to give more well-defined contact pressures at the two separate wedges as compared to solutions with contact over the entire base area. For further discussion of this aspect, see Mortensen [2]. In addition a solution with large diameter steel dowels was included as a variant in the tendering, but this was priced considerable higher than the basic stone bed solution. The stone materials used as well as method for compaction, underbase grouting etc. are the same as described for the pylon foundations.

3.2 Foundation Caissons

The foundation caissons, each covering an area of 6,100 m^2, are 121.5 m long and 54.5 m wide and divided in three parts. Only the front and rear parts are in contact with the supporting stone wedges as explained above. The height of the pre-fabricated part of the caissons is 15 m having a weight of 50,000 tonnes when towed to the bridge site whereas the remaining parts of the anchor blocks above level +3.0 m were cast in-situ.

The rear part of the caissons contains the anchorage massifs comprising 18,000 m^3 of concrete which was cast in-situ partly inside some of the cells in the pre-fabricated caisson. Other cells in the rear part were ballasted with olivine (heavy sand) and iron ore.

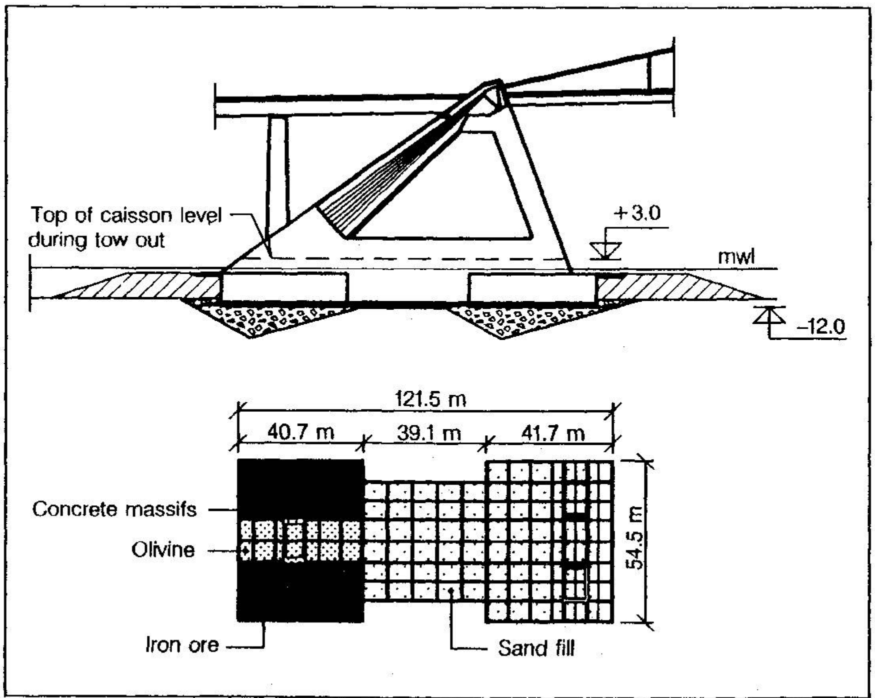

Fig. 4: Elevation and Plan of Anchor Block Caisson.

The middle part of the caisson, which has no contact with the underlying ground, is partly sand-filled; the longitudinal walls are post-tensioned with vertical bars; and the bottom slab is heavily post-tensioned longitudinally with a total force of 363 MN. The front part of the caisson is filled with hydraulically pumped sand.

The anchor block caissons are protected against ship impacts by artificial islands constructed from soil materials dredged nearby. This dredging -known as compensation dredging- was done in order to open up for increased water flow to compensate for the blocking effect of the bridge substructures. The protection islands are elliptical shaped with the long axis in the direction of the predominant water flow in order to reduce the blocking effect.

3.3 Structural Analyses

The structural calculations for the anchor blocks were mainly based on a linear elastic finite element model using the IBDAS programme, whilst the geotechnical calculations and the determination of soil pressure distributions were carried out by separate analyses models comprising use of the non-linear finite element program ABAQUS. For further information see [3]. The soil pressure distributions were then applied on the IBDAS model.

Due to the unique concept of the anchor blocks involving application of heavy post-tensioning and ballast material in various construction phases as well as the construction of the inclined upper elements by free cantilevering from the caisson deck, it was necessary to analyse a substantial number of construction phases and keep track of the built-in stresses and their re-distribution due to creep and shrinkage in subsequent phases up to the final condition. The main construction phases included:

- Construction of the caisson in the dry dock including tensioning of tendons in the bottom slab and vertical bars in the longitudinal walls of the mid-part.
- Tow-out and installation of the caisson including effects of ballasting for trimming and wave loads.
- Ballasting of the caisson with olivine/iron ore and sand before construction of the upper inclined elements.
- Construction of the upper inclined elements by free cantilevering.
- Final condition after connection of the inclined elements by the top cross beam.

The calculations showed that the stresses built-in during the construction phases were very important and required substantial strengthening in particular for the lower part of the caisson structure. However, the model was very useful for optimisation of the entire construction process including adjustment of post-tensioning levels and ballasting procedures at various stages.

4 APPROACH BRIDGE PIERS

The decisive criterion for the design of the approach bridge piers is ship impact loads, which was established based on a comprehensive collision risk model. The model estimates the risk of collapse as the sum of four risk scenarios, each representing a certain ship track pattern categorised as follows:

- Ships following the ordinary, direct route at normal speed. Accidents mainly due to human error or unexpected problems with propulsion/steering system when approaching the bridge.
- Ships failing to change course at the turning points of the navigation route
- Ships taking evasive action for other vessels when approaching the bridge.
- All other track patterns, e.g. off-course ships and drifting ships.

Based on the collision risk model the design basis for the ship impact loads was established, see Fig. 5.

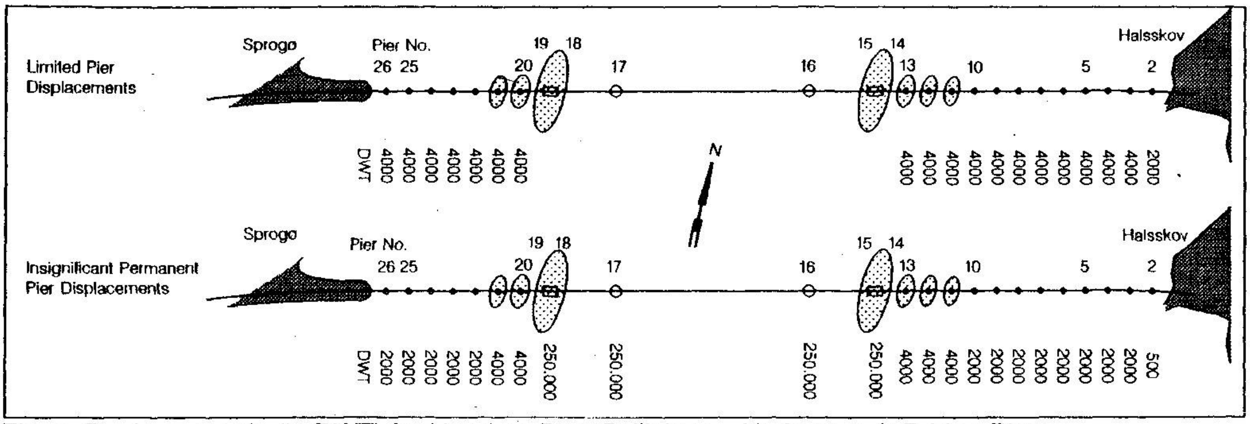

Fig. 5: Design vessels (in DWT) for Head on Bow Collisions with Approach Bridge Piers

In order to achieve the same overall base dimensions, 23 m x 19 m, of all the approach bridge piers, it was decided to construct artificial protection islands around some of the piers close to the navigation channel, see Fig. 5. The remaining piers were designed to resist an impact load of 69 MN from a 4,000 DWT ship allowing limited displacements at the pier top of 200 mm and 500 mm in the transverse and longitudinal directions respectively. These displacements will require repairs, however, without closing the bridge for more than about one month. In addition, the piers shall be able to resist an impact load of 46 MN from a 2,000 DWT ship without insignificant permanent displacements which will not require closing for traffic.

A simplified but still advanced computer model was developed for the ship impact analyses, see Fig. 6. The model comprises: a non-linear elasto-plastic model of the soil-structure interaction, a stiff foundation caisson, elastic beam elements for the pier shaft and a concentrated mass and spring at the pier top representing the bridge girder. To verify the simplified model, independent 2D finite element analyses were carried out using two different programs ABAQUS and FENRIS. In addition the effects of eccentric impacts were studied by 3D models with a more refined modelling of the soil, including local week areas etc. in FENRIS.

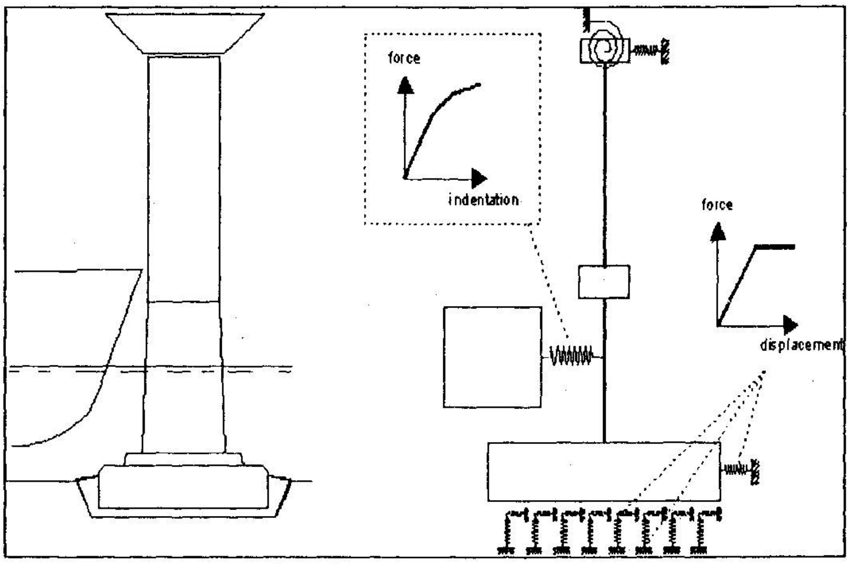

Fig. 6: Idealised model of the Ship Collision.

The comparison of results from the simplified model and the finite element analyses showed a good agreement with differences in ultimate bearing capacity being some 10% only. However, the finite element models give a better description of the behaviour of the soil especially when failure is dominated by overturning and twisting moments. For further information, see Feld and Gravgaard, [4].

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