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**GEOTECHNICAL ENGINEERING GROUP
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Structure-Foundation Interaction on the Storebælt Link East Bridge

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Summary

Interaction between structure and foundation in different limit states are described by:

- A verification of an eccentric ship impact - comparing a BEM model including girder, piershaft and soil with a FEM analysis of the pier and the soil beneath it (ALS)
- Three different independent calculation methods to determine the bearing capacity of the anchor blocks (ULS)
- Different methods to predict the directions and size of the settlements (SLS) for both approach piers and anchor blocks are presented.

1. East Bridge Project

In 1986 the Danish government established a political agreement for connecting the island Fyn with the island Sjælland.

The Storebælt Link project entails three major civil engineering contracts (see Figure 1):

- the West Bridge (rail and motorway)
- the bored tunnel (railway), and
- the East Bridge (motorway) being the world's at present longest suspension bridge with a free span of 1624 m.

This paper looks at the interaction between bridge structure and foundation for the East bridge in the different limit states from a geotechnical point of view.

The East Bridge substructure consist of tree major component (see Figure 2):

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

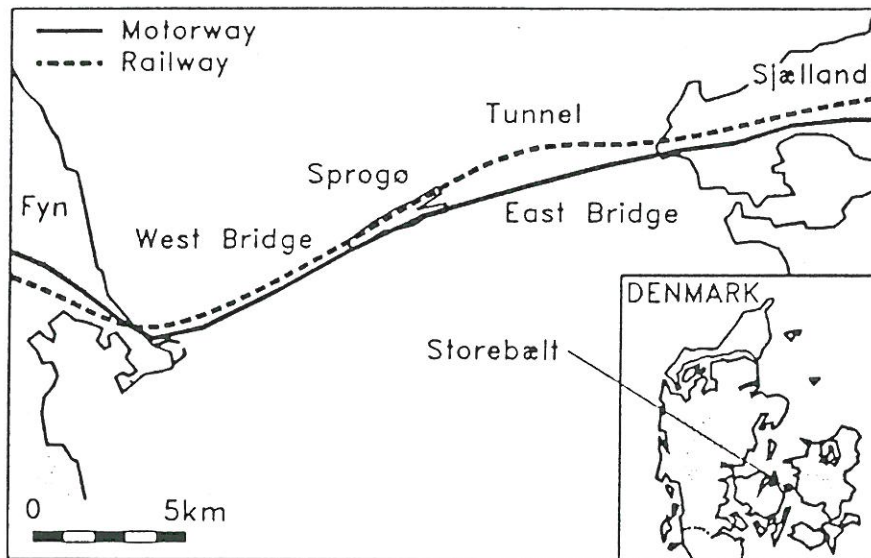


Fig. 1 Location and components of the Storebælt Link

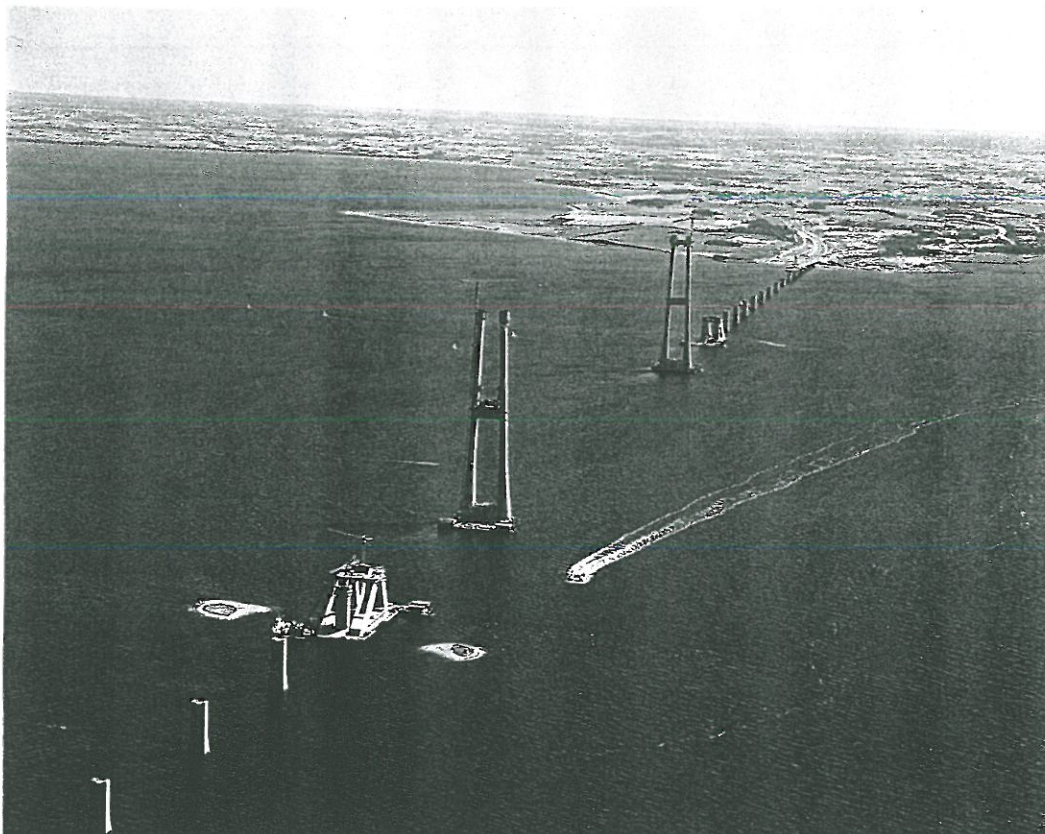


Fig. 2. Birds View of East Bridge, Denmark late 1994.

Generally, the geology consist of two layers of Clay till, with stiffness increasing with depth, underlain by a marl layer and below that, limestone.

Shallow foundation is chosen throughout the project. During the design phase the different limit states lead to a number of challenges in applying advanced analysis method to real life problems

2. Accidental Limit State (ALS)

Based on probabilistic analyses a load scheme for each structure was determined in terms of size and velocity of the colliding ship. The consequences of an impact were determined using a computer program, SIAS62, generating results for inclusion in the overall probabilistic analysis for the entire East Bridge.

2.1 Ship Impact Analysis - Approach Pier

The detailed design of the approach piers included a ship impact analysis, yielding the final dimensions of each pier. In most cases, the critical ship collision with the pier will occur eccentrically, requiring a complete 3D model consisting of sand, relatively hard clay layer, inclined soil interfaces and a local occurrence of soft clay embedded in harder ones under a pier corner.

2.1.1 Boundary Element Model

Generally a simplified, but still advanced computer model, SIAS62, has been used for the analysis of ship impact, ice loading and other accidental loads. The program performs an analysis of the total structure: Pier, bridge girder, neighboring piers and soil, thus including several structural elements in absorbing the impact. In the Boundary Element Model the soil is modelled by simple linear elastic, ideally plastic springs for deflection in three directions and torsion. However these springs reflect only the soil relatively close to the foundation, whereas soil layers more distant are not taken into consideration.

2.1.2 Finite Element Soil Model - Soil Response

To verify the simplified model, independent finite element analyses were carried out. Two independent non-linear finite element programs: ABAQUS and FENRIS were used. Hand calculations were not deemed possible due to the 3D nature of the problem.

2.1.3 Establishment of a Realistic Finite Element Model

Creating a realistic mesh, yet limiting the calculation time was a challenge. The establishment of a suitable 3D mesh to analyze the eccentric ship impact was based on findings from 2D analyses. (simulating a central ship impact.) In the case of a central ship impact, a 2D model is sufficient, thus it is possible to examine the influence of element refinement, special meshing techniques etc.

2.1.4 Comparison between Finite Element Analysis and Boundary Element Model

Figure 3 presents the Load-Displacement curve for both FENRIS and SIAS62. The difference in ultimate bearing capacity is found to be some 10% only.

A comparison of the failure curves of the FEM and BEM analyses, yields that the BEM is more conservative with respect to ultimate bearing capacity - and probably less able to describe the actual behavior of the soil during the serviceability state.

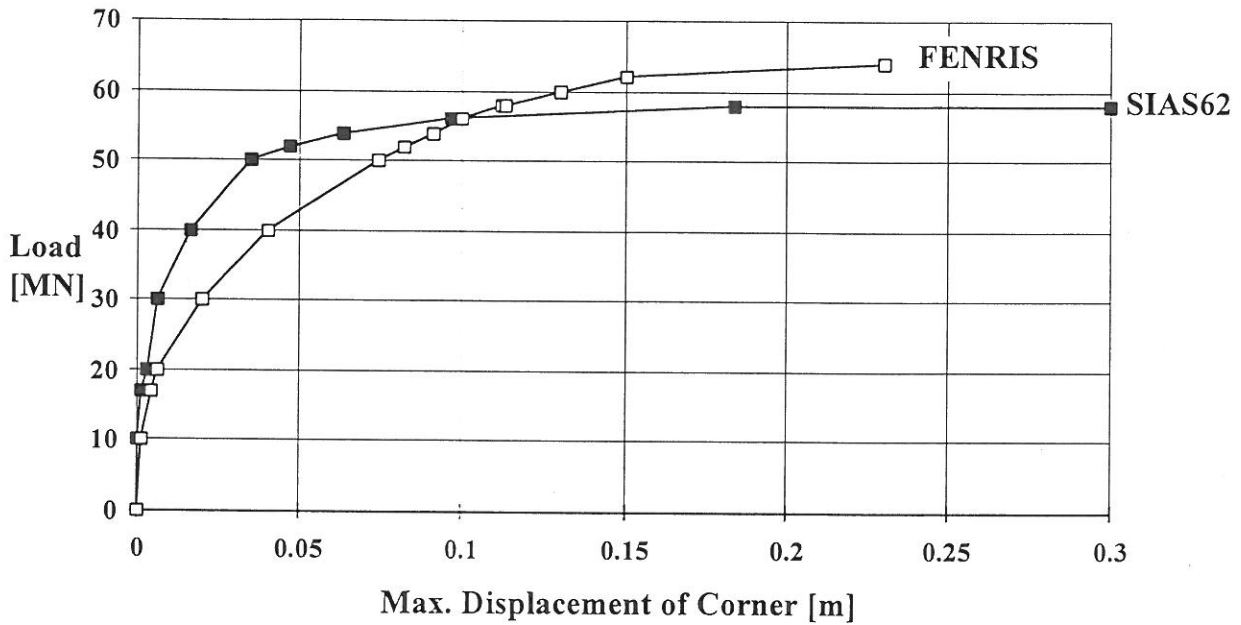


Figure 3 Load-displacement curve for an eccentric ship impact analyzed both in FEM and BEM

The FEM calculation for the eccentric ship impact resulted in larger overturning effects .e.i the pier tilts more and as a result rotates less in the FEM analysis compared to the BEM analysis. This is a natural consequence of the more refined modeling of the soil, including local weak areas, lenses etc. in FENRIS. The deformed 3D mesh at failure is depicted in Figure 4

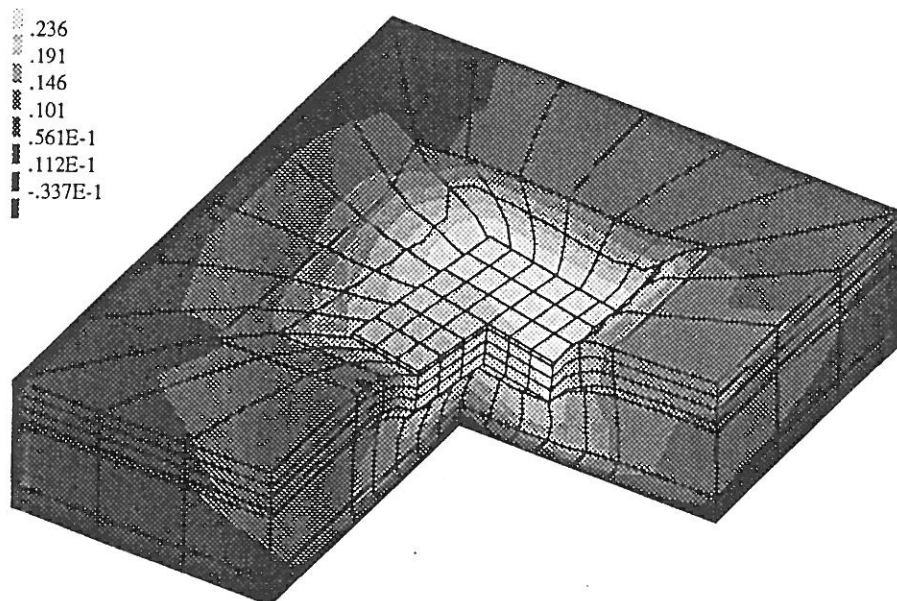


Figure 4 Deformed mesh due to an eccentric ship impact

From the results it was concluded, Feld and Gravgard (1994), that the soil model in SIAS62 describing the ultimate capacity of a foundation subjected to vertical forces, horizontal forces, overturning moment and twisting moment, results in reasonable capacity prediction in cases where failure is dominated by sliding.

2.2 Pylons

Unlike the approach piers the dimensions of the base (78m by 35m) were primarily dictated by the requirements for structural integrity in the base and lower part of the pylon legs. Requirements to bearings capacity and settlement criteria's were not governing. However FEM analyses were carried out similarly to the approach piers.

2.3 Anchor Blocks

The risk assessment resulted in protective islands in front of each anchor block, diminishing the possibility of a ship collision to negligible ship sizes. Leading the ultimate limit state to impose the dimensions of the foundation of the anchor blocks.

3. Ultimate Limit State (ULS)

The loads from the main cable are carried by two anchor blocks located at 10 m water depth.

3.1 Bearing Capacity - Anchor Blocks

3.1.1 Anchor Blocks

Each anchor block has a rectangular form with the length of 121.5 m and the width of 54.5 m (Figure 5). This base is divided into 3 parts, a front pad of 41.7 m, a middle part of 39.1 m and a rear pad of 40.7 m. Only the front and the rear pads are in contact with the supporting soils.

Both anchor blocks are founded on very stiff to hard preconsolidated clay till. The undrained shear strengths range from 150 - 300 kPa.

As a result of excavation, the top part of the clay till was included in the design with a reduced sliding resistance. This was compensated for by introducing a wedge shaped fill of compacted crushed stone below each of the two pads.

3.1.2 Anchor Block Loads

The loading situation on Figure 5(a) leads to a resulting force shown in Figure 5(b).

Assuming a uniform vertical load distribution for each of the two pads, the two vertical reaction forces are statically determined.

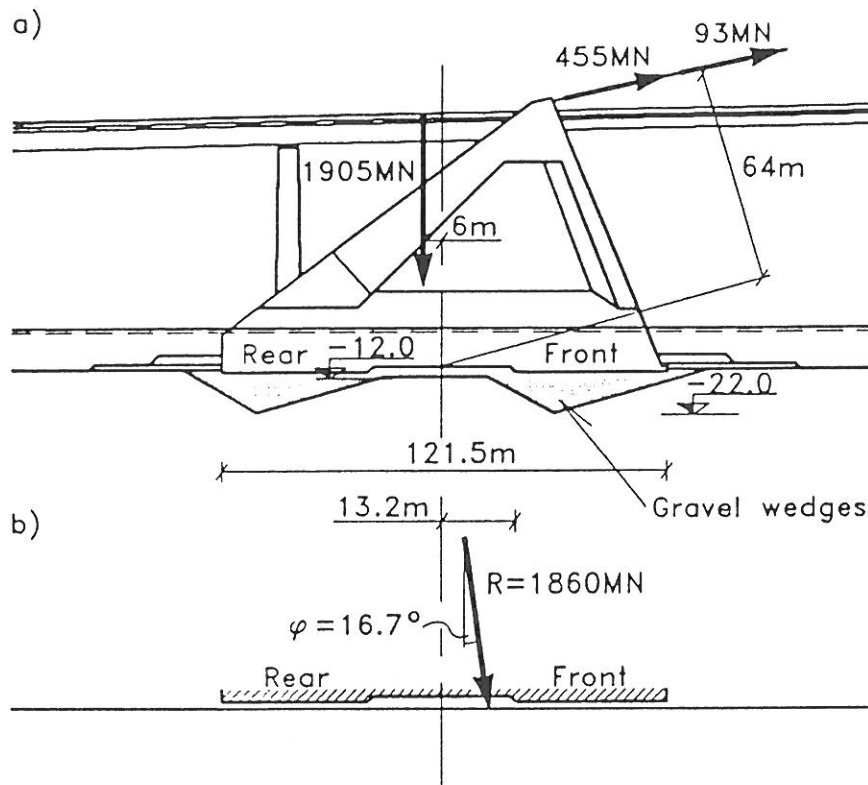


Figure 5 (a) Section of Anchor Block;
(b) Load Resultant acting at Foundation Level

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

3.1.3 Bearing Capacity Analysis

The bearing capacity analysis was performed both as a traditional deterministic and as an advanced probabilistic ultimate limit state analysis.

Three principally different types of failure modes are possible for each foundation pad depending on load inclination, see Figure 6.

The 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). A correct solution for the bearing capacity has to be both statically and kinematically admissible.

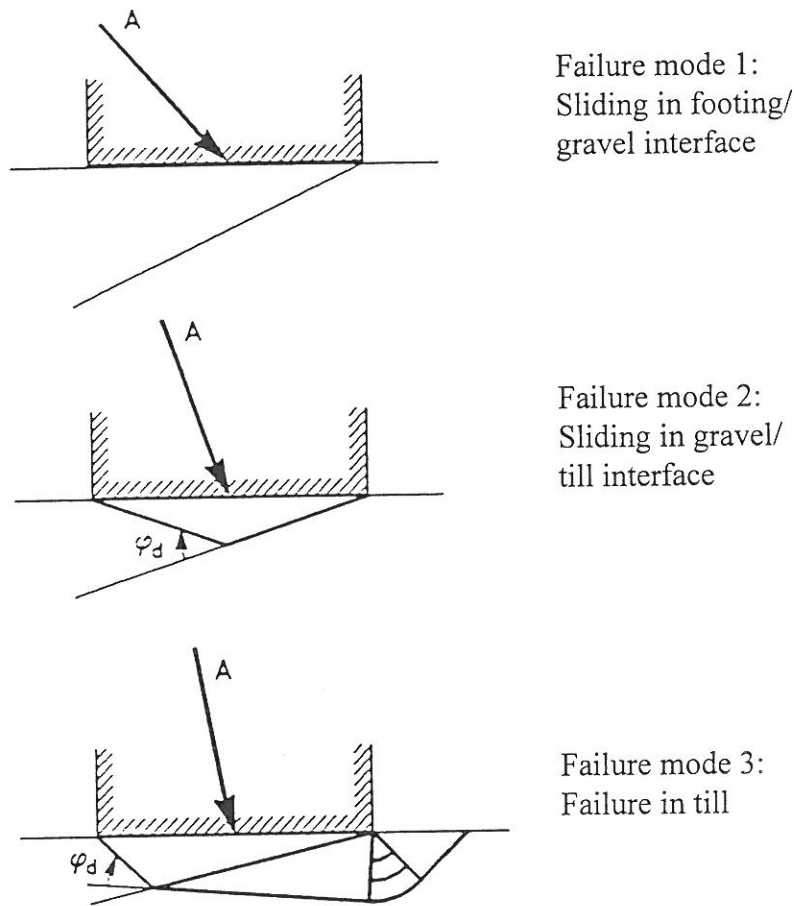


Figure 6 Foundation Pad Failure Modes

It is difficult to find solutions which fulfill both conditions. Therefore, it was decided to use three different and independent calculation methods to determine the bearing capacity of the anchor blocks. The selected methods, all in 2D, included:

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

Prior to calculation of the bearing capacity of the anchor blocks, the three calculation methods were tested through five bench mark cases. From the results it was concluded, Sørensen et al (1993), that all three methods could be considered as relevant and usable tools for the design procedure. The method which resulted in the lowest value of safety should be the decisive one.

One of the tests was called *The Anchor Block Case* with a geometry and a soil strength nearly identical to those of the real anchor block. The calculation resulting bearing capacities were found to range from 32.3 MN/m to 33.8 MN/m (corresponding to the 2D simplification of

strip footings). The rupture figures for the upper bound and ABAQUS analyses are shown in Figure 7 and 8.

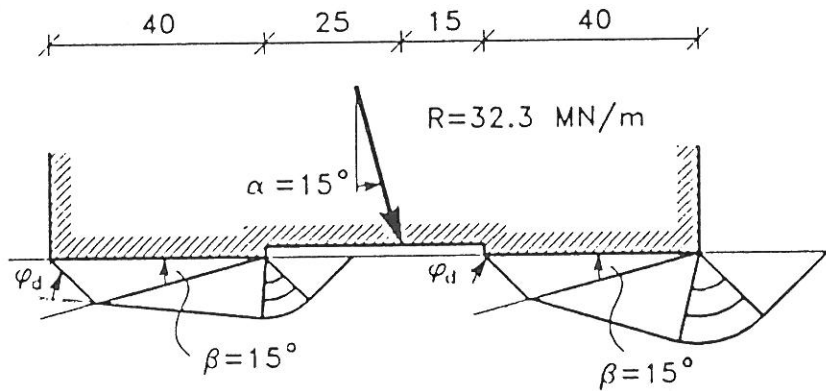


Figure 7 Rupture Figure, Upper Bound Analysis

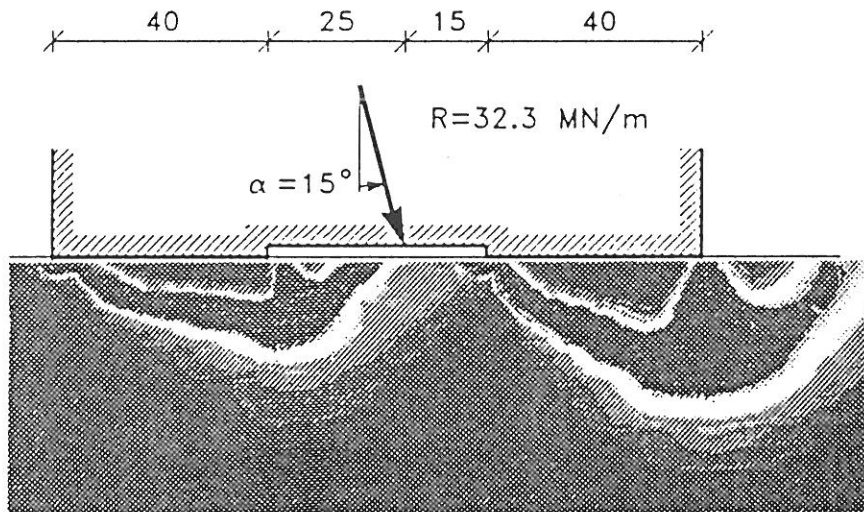


Figure 8 Rupture Figure, ABAQUS Analysis

In addition to these 2D analyses, supplementary 3D analyses were performed for verification of the adequacy of the 2D analyses.

The probabilistic analysis indicated a coefficient of reliability of $\beta = 4.9$. This corresponds to high safety class according to the Danish codes.

4. Serviceability Limit State (SLS)

4.1 Anchor Blocks

The size and complexity of the anchor block foundation called for the use of advanced calculation methods. A hand calculation was considered impossible, thus the finite element program ABAQUS was utilized. The following effects were included in the determination of the settlements :

- the complex geometry of the anchor blocks
- the geometry of the gravel wedges
- the variability of the soil conditions, and
- the detailed loading sequence of the anchor block.

The deformation parameters for the soil were determined by advanced laboratory tests, and the stress-strain behavior was well documented.

4.2 Approach Piers

The difficulty in the SLS analysis was applying an analysis true to the nature of the clay. A method based on SHANSEP strength and stress model combined with Bjerrum's theory of primary and secondary consolidation was established. Sørensen et al (1995) describe this method in more detail.

This model applied the different loading and unloading schemes during the different construction stages while it at all times kept track of consolidation stress, preconsolidation stress and displacements. The model can be updated and changes made, when comparing to the real project allowing for delays etc.

5. Monitoring

5.1 Anchor Blocks

Porepressure gauges and inclinometers have been installed in the gravel wedges and in the underlying clay till. On-line monitoring of the porepressure and inclination has not begun at this stage. However, when registration starts, the readings will continuously be compared with the design assumptions.

5.2 Settlements

5.2.1 Approach Ramp

An unique possibility to follow in full scale the settlement and creep model established for the clay till occurred during construction of the embankment at Sprogø. The settlement model is based on laboratory tests from the East Bridge project.

Settlement gauges have been installed in the base of the embankment at Sprogø, in the underlying clay till and finally below the clay till in the marl. Monitoring of these gauges are still on-going (sixth year).

5.2.2 Substructures

From the time of installation of the caissons, settlements observations started and have been recorded ongoing using a DGPS satellite system. The system has been used on the approach piers, the pylons as well as on the anchor blocks. Upon placement of the girders, relative readings have been compiled using traditionally land base survey (trigonometric readings) This monitoring is still on-going (third year).

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