# INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING

XIII ICSMFE, 1994, New Delhi, India / XIII CIMSTF, 1994, New Delhi, Inde

# ANALYSIS OF AN ECCENTRIC SHIP IMPACT
# ANALYSE SUR UN IMPACT EXCENTRIQUE DE NAVIRE

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SYNOPSIS : As part of the detailed design of the piers for the Storebælt link's East Bridge the critical load scheme, an eccentric ship impact, is yielding the final dimensions of each pier footing. Subsoil conditions may include inclined soil interfaces and sand lenses. A boundary element model, including girder, pier shaft and soil was used. To verify this simplified model a finite element analysis was carried out, modelling pier shaft and soil. Hand calculations were not deemed possible due to the 3D nature of the problem. The process of evaluation of soil parameters is described.
2D analyses were carried out for different soil and element mesh conditions. The results were evaluated and compared with those from the 3D analysis - the need of executing 3D analyses is discussed. The outcome of the FEM and BEM analyses are compared.

## INTRODUCTION

In 1986 the Danish government decided to connect the island of Funen with the island of Zealand. For the client, A/S Storebæltsforbindelsen, RH&H and Cowiconsult are presently preparing the detailed design of the motor way bridge across the Eastern Channel. The project includes the world's at present longest suspension bridge with a free span of 1624 m.

Generally, the geology consists of two layers of Clay Till, the lower stiffer than the upper, underlain by marl, and below that, limestone.

Shallow foundation is used throughout the project. As part of the detailed design of the approach piers ship impact is analysed. The most critical design ship impact will, in most cases, occur eccentrically. This paper focuses on the problems encountered when applying advanced analysis method to real life problems. The preparation of soil model parameters and a realistic finite element mesh are the main issues addressed in this paper.

Generally the stress-strain behaviour was very well documented. Several triaxial tests were available. For the fill material no tests had been performed, since the material had not yet been selected by the contractor.

Creating a realistic mesh for the model calculation, yet limiting calculation time to a minimum was a challenge. The problem investigated was in no way symmetrical, calling for a complete model: On one side sand with a relatively hard clay layer. And on the other a soft clay layer embedded in harder ones.

## SHIP IMPACT ANALYSIS

A simplified, but still advanced computer program, SIAS62, has been used generally for the analysis of ship impact. The program performs an analysis of the total construction: Pier, bridge girder, neighbouring piers and soil, thus permitting many construction parts to absorb the impact. The soil is modelled by simple linear elastic, ideally plastic springs for deflections in three directions and torsion. These springs only reflect the soil in relatively close contact with the foundation, whereas more distant soil layers are not modelled. The project Q/A procedure requires, that all calculations be subjected to quality control. For the soil part of the problem an independent analysis, by finite element modelling, was decided upon. A hand calculation was considered impossible. Two non-linear finite element computer programs: ABAQUS and FENRIS were used. Initially planar investigations were performed as control. But when it became apparent, that an eccentric ship impact, in some cases, was the dimensioning load scheme for a pier, a 3D analysis was decided upon. FENRIS, part of the SESAM analysis system, was used for this analysis.

## SOIL INVESTIGATIONS

For each pier the project investigations were planned to consist of two geotechnical borings and eight PCPT soundings. The structures for the main span have been closer investigated. The complete investigations for the East Bridge have approximately comprised:

- 350 PCPT soundings (3500 m).
- 100 Geotechnical borings (3000 m) with vane shear tests, SPT tests, undisturbed and disturbed samples.
- Menard pressuremeter and elastmeter tests.
- Pumping tests.
- 200 Triaxial tests, distributed on Clay Till, upper and lower, marl, limestone, sand and lately gravel fill material. Samples have been selected throughout the sample material to cover a wide range of strengths and locations.
- 70 Oedometer tests with the same distribution as mentioned above.
- Direct, simple shear tests.
- Brazilian tests on marl and limestone.
- Unconfined compression tests.
- A wide range of classification tests: Moisture content, Atterberg limits, lime content, sieve analyses etc.

### Design Soil Profile Elaboration

For each pier a soil profile with characteristic parameters is prepared, based upon the project investigations: PCPT soundings, Borings with vane shear tests and the various classification tests. In cases with very varying conditions more than one such profile is elaborated. PCPT soundings are treated with a moving average filter, removing spikes from stones in the clay till soil matrix and correcting cone resistance for pore pressure and overburden. The soundings are aggregated into one characteristic profile, or more in cases as mentioned previously. The resulting profile is plotted together with data from the in situ tests. An example of such a profile is shown on Figure 1. From the boring a profile is constructed with characteristic parameters. An example is shown on Figure 2. The design soil profile with characteristic soil parameters forms the parametric basis for all analyses: ULS, SLS and ALS.

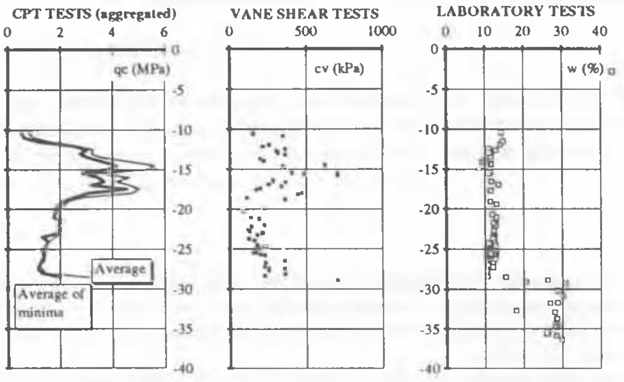
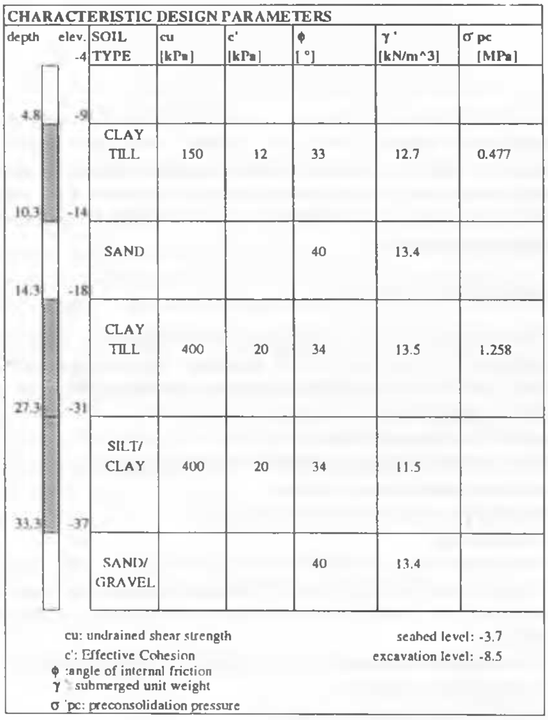


**Fig. 1.** Soil parameters for design soil profile preparation.



| CHARACTERISTIC DESIGN PARAMETERS | | | | | | | |
|---|---|---|---|---|---|---|---|
| depth | elev. -4 | SOIL TYPE | cu [kPa] | c' [kPa] | ϕ [ °] | γ ' [kN/m^3] | σ pc [MPa] |
| 4.8 | -9 | | | | | | |
| | | CLAY TILL | 150 | 12 | 33 | 12.7 | 0.477 |
| 10.3 | -14 | | | | | | |
| | | SAND | | | 40 | 13.4 | |
| 14.3 | -18 | | | | | | |
| | | CLAY TILL | 400 | 20 | 34 | 13.5 | 1.258 |
| 27.3 | -31 | | | | | | |
| | | SILT/ CLAY | 400 | 20 | 34 | 11.5 | |
| 33.3 | -37 | | | | | | |
| | | SAND/ GRAVEL | | | 40 | 13.4 | |

cu: undrained shear strength
c': Effective Cohesion
ϕ :angle of internal friction
γ ': submerged unit weight
σ 'pc: preconsolidation pressure

seabed level: -3.7
excavation level: -8.5

**Fig. 2.** Characteristic design soil profile

## THE FINITE ELEMENT SOIL MODEL

The non-linear finite element program FENRIS uses the Mobilised Friction Model (MFM) developed by SINTEF.

The model is based on effective stresses. It may be applied for both drained and undrained conditions, as well as a combined analysis. It is based on an elasto-plastic formulation with two yield surfaces, a coulombian cone and an end closing cap. The yield surface is an extension of the simple Drucker - Prager surface. Isotropic hardening, controlled by means of the mobilised friction, is adopted. Non-associated plastic flow allows for flexible dilatancy and contractancy control. During undrained loading in a triaxial test only plastic shear strains on the cone occur, and during oedometer tests it is the cap which is active.

The elastic bulk modulus, K, and the shear modulus, G, are both functions of the current preconsolidation stress level. The current preconsolidation stress level is the hardening parameter for the cap yield surface. It relates to permanent volumetric strains.

## INTERPRETATION FOR SOIL MODEL INPUT PARAMETERS

Generally, the interpretation can be divided into three actions for each type of soil:
- Interpretation of triaxial tests.
- Interpretation of oedometer tests.
- Adaptation to in situ conditions.

Five soil materials have been modelled for the ship impact analysis:
- Clay Till. The type generally found in the area.
- Clay Till. A local softer layer.
- Sand. A melt water deposit.
- Gravel bed and backfill material.
- Remoulded clay till at excavation boundaries.

### Clay Till.

As previously mentioned a large number of triaxial tests have been performed on this material, mainly multiple-stage anisotropically consolidated undrained tests (MACU). Samples for testing were selected to provide an array of tests covering a spectrum of strengths of most materials. When an analysis of a particular design aspect is to be carried out, there will not necessarily be a specimen well tested in that particular area. However, tests will probably be available from the array prepared, with similar characteristics. The MACU tests are performed with the first stage at the in situ vertical stress, the second in the area of $\sigma'_{pc}$ and the last increment significantly above $\sigma'_{pc}$.

Stress paths from three such tests are displayed on Figure 3. The similarity in behaviour is clear. One test was picked out as typical for normal clay till behaviour. On this sample both an oedometer and a triaxial test have been performed on adjacent parts. The interpretation has followed the path indicated on Figure 4. The local oedometer test is used for local determination of elastic and elasto-plastic parameters.

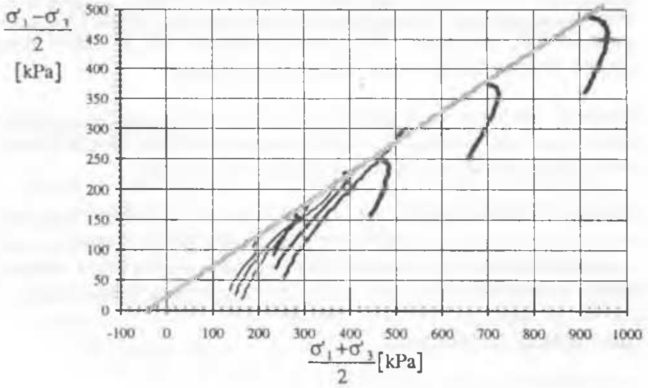


**Fig. 3** Stress paths, triaxial tests on clay till

### Clay Till, local softer layer.

Both an oedometer and a triaxial test had been performed on this particular layer due to its deviation from general conditions. The same procedure as demonstrated above was used.

**Oedometer and triaxial test on same sample**

**Oedometer**

$$k_0 = \frac{\Delta\sigma'_v}{\Delta\varepsilon\,(\sigma'_{vur} + a)}$$

elastic bulk modulus number

$$g_0 = k_0 \frac{3(1 - ARC)}{(2 + 4ARC)}$$

shear modulus number
where ARC is the coefficient of earth pressure at rest

**Triaxial**

a attraction
RN stress-dependency exponent
$\Delta\alpha_Q$ dilatancy parameter
$\left(\sum \Delta\lambda_{cone}, \sin(\rho), \alpha_{Qspec}\right)$ Stress/strain path

where $$\Delta\lambda_{cone} = \sqrt{\frac{2}{3}}\left(\frac{3}{2}\varepsilon_1 - \frac{\Delta(\sigma_1 - \sigma_3)}{2G}\right)$$

plastic strain

$$\alpha_{Qspec} = \frac{\alpha_{Qi}}{\sqrt{\frac{\sigma'_{mi}}{\sigma'_{ci}}} \cdot \left[1 - \Delta\alpha_Q\left(\frac{\sigma'_{ci} - \sigma'_{mi}}{\sigma'_{ci}}\right)\right]}$$

volumetric plastic strain ratio

where $$\alpha_{Qi} = \frac{\Delta\sigma'_{mi}}{3K_i\Delta\lambda_{cone}}$$

where

$$K_i = k_0(\sigma'_{ci} + a)$$

$$\sin(\rho) = \frac{\sigma'_1 - \sigma'_3}{\sigma'_1 + \sigma'_3 + 2a}$$

mobilized friction

$$g_0 = \frac{G}{100\left(\frac{\sigma'_c + a}{100}\right)}$$

where $\sigma'_c$ is the preconsolidation pressure in mean stress terms

$$G = \frac{\Delta(\sigma'_1 - \sigma'_3)}{3\Delta\varepsilon_1^e}$$ initial loading ,or unloading

**Oedometer test on local sample**

$$k_0 = \frac{\Delta\sigma'_v}{\Delta\varepsilon\,(\sigma'_{vur} + a)}$$ elastic bulk modulus number

$$RATMOD = \frac{K^{ep}}{K}$$ elasto-plastic ratio

**In Situ Parameters**

$$\sigma'_c = \frac{\sigma'_{pc} + 2ARC\sigma'_v}{3}$$

preconsolidation pressure in mean stress terms

where $$\sigma'_{pc} = \sigma'_v\left(\frac{c_u}{0.4\sigma'_v}\right)^{\frac{1}{0.85}}$$ Mayne (1988)

**Fig. 4.** Interpretation procedure for clay till.

### Sand and Gravel.

These materials were difficult to model the traditional way due to a lack of available tests. The gravel for backfill around the caisson and gravel pad had not been selected by the contractor at that point of time.

For the natural deposit of melt water sand the angle of internal friction, $\phi'$, could be determined from the PCPT tests, and for the gravel material from project specifications. Since no triaxial test were available it was decided to artificially create the simplest possible stress path, with no dilation. An oedometer test from a melt water sand deposit in another area was incorporated. A young's modulus, E, for initial loading could be established as well, and from this value the shear modulus, G, could also be determined.

The resulting stress-strain curve and stress path is shown on Figure 5.

## GEOMETRY OF REPRESENTATIVE PIER

A typical Pier was chosen as representative for the approach piers. Figure 6 shows the chosen pier including the soil conditions modelled.

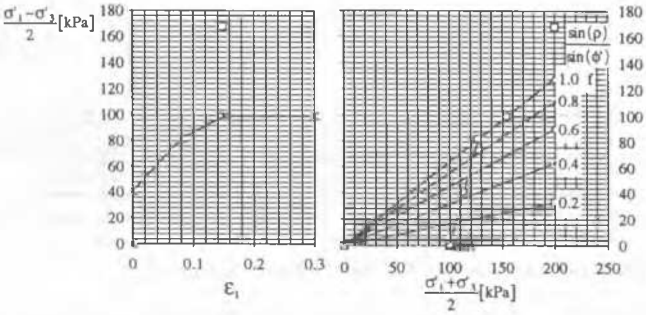


**Fig. 5.** Stress-strain and stress path

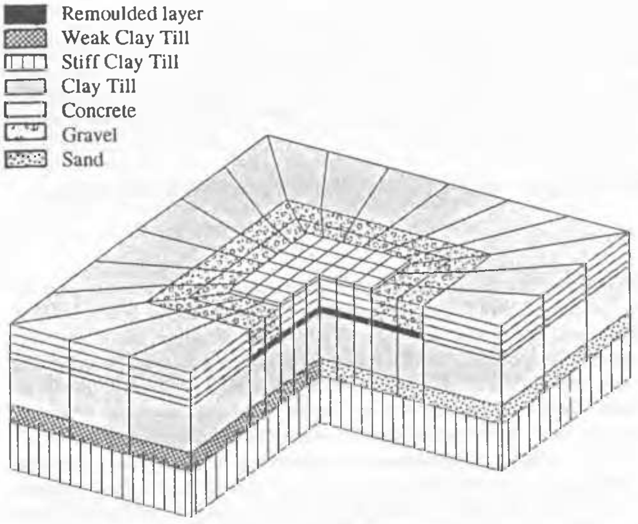


**Fig. 6.** Typical substructure .

## CONSTRUCTION OF A REALISTIC FINITE ELEMENT MODEL

In order to establish a suitable 3D model to examine the case of an eccentric ship impact, the authours chose to construct the element mesh based on findings from 2D analyses ( simulating a central ship impact ). A number of 2D analyses were carried out for different element and soil conditions.

### Element configurations

In the case of a central ship impact to a pier, a 2D-model of the problem is sufficient. Thus it is possible to examine the influence of element refinement, special meshing techniques etc.

A 2D model of a representative Pier and the soil beneath it was established utilizing 8-node membrane elements. Gravel wedges as well as a remoulded clay layer have been modelled. The different soils are modelled according to the actual material properties using the mobilized friction material model (MFM) described earlier. The pier is assumed infinitely stiff.

Different element coarsenesses were examined to determine the needed mesh refinement to obtain a resonable result. Figure no.7 depicts the deformed mesh at failure caused by a central ship impact.

A feature in FENRIS makes it possible to create a FEM mesh *shaped as a flownet*, this exersize was performed in order to get the aigorithm to converge as rapidly as possible. The flownet shaped mesh is shown in Figure 8. For all the different mesh configurations it applies that the area where failure is expected to occur, the element mesh is very detailed whereas the coarseness increates as one moves away from the point of action and closer to the boundary conditions.

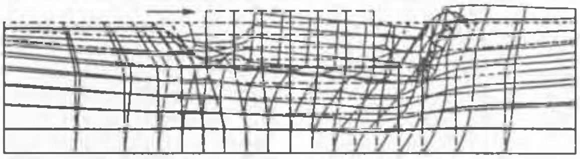


**Fig. 7.** Deformed mesh at failure due to a central ship impact

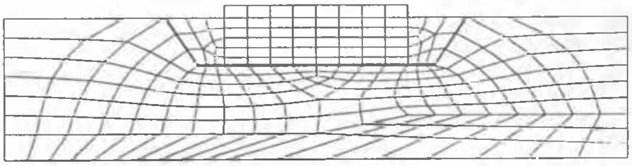


**Fig. 8.** *Flownet - shaped* FEM mesh to obtain an accelerated convergence.

**Soil conditions**

The soil conditions were also examined preliminarily in 2D - this was done for two reasons : First of all to examine the effect of modelling gravel wedges, sand lenses, the remoulded clay layer, local week areas etc. and secondly to determine, how detailed the 3D analysis needed to be in order to achieve a results which could be used as part of the verification of SIAS62. Ideally all checks should be carried out as general non-linear dynamic analyses. However, due to computation time this was not practically possible for the 3D analysis - Hence it was chosen to perform static analyses only, when calculating the undrained failure. Actual loads were applied for the permanent load ( self weight of pier ) while the ship impact was increased until failure occured. Maximum ship impact load and maximum displacements from FENRIS were calculated and compared with the results from SIAS62.

Due to the eccentric ship impact and the different subsoil conditions, no plane of symmetry could be used in the 3D-analysis. The FEM mesh is made relatively coarse to limit the calculation time for the 3D analysis. Based on the tests performed in 2D involving mesh coarseness - it has been tried to obtain the same numerical error in 2D and 3D, hence achieving a reasonable result in 3D. All elements used are 20 nodes solid IHEX elements each with 3DOF, except for boundary nodes.

## COMPARISON BETWEEN FINITE ELEMENT ANALYSES AND BOUNDARY ELEMENT ANALYSES

The Load - Displacement curve for both FENRIS and SIAS62 is depicted in Figure 9. The difference in ultimate bearing capacity is found to be 13%. This difference is evaluated to be caused by the more refined approach in the Finite Element Analysis, where the soil is modelled with the actual properties. When comparing the failure curves of the FEM and BEM analyses, it is noted 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 servicebility state. A BEM analysis seems to make the soil stronger than it is in reality. However, the roughness of mesh does include a possible numerical error which could result in part of the difference.

It is observed that for an eccentric ship impact the pier tilts more and as a result rotates less in the FEM calculations compared to the BEM analysis. This is a natural concequence of the more refined modelling of the soil - including local week areas, lenses etc. in FENRIS. Figure 10 shows the deformed 3D mesh at failure.

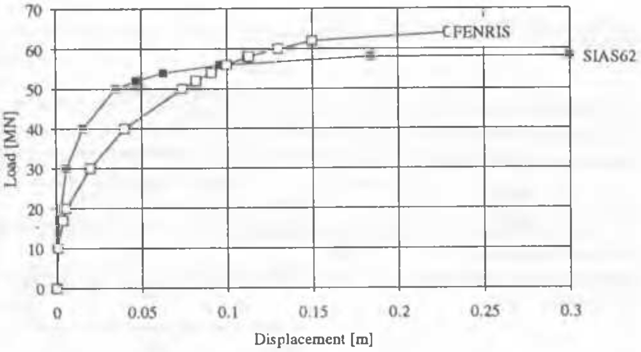


**Fig. 9.** Load - displacement cuve for an eccentric ship impact Analysed both in FEM and BEM.

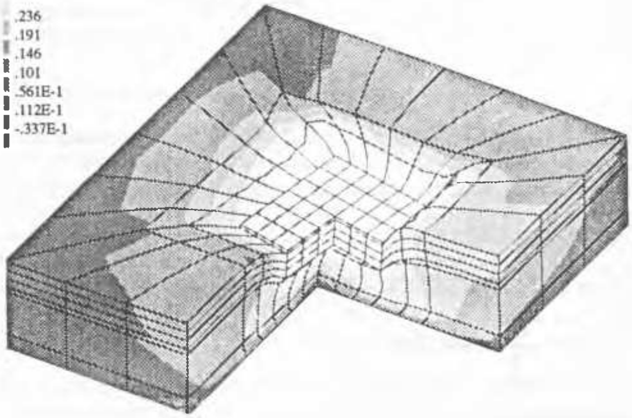


**Fig. 10.** Deformed mesh due to an eccentric ship impact.

When comparing the ultimate bearing capacity found in 2D and 3D respectively, it is noted that the ultimate capacity in the 3D analysis is greater than the corresponding value found in 2D. At first this might seem unreasonable; since one would expect a lower capacity for the eccentric (3D) ship impact due to rotation and torsion. However, in the 3D analysis the weak area is only incorporated in one corner ( to simulate reality as close as possible ) where in 2D it is a generel layer.

In light of the present problem it was verified 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 a reasonable capacity in cases where failure is dominated by sliding.

## ACKNOWLEDGEMENTS

The authors are greateful to the Client, A/S Storebæltsforbindelsen, for permission to publish this paper. Futhermore we wish to thank our joint venture partner COWIconsult. We also wish to thank the Danish Geotechnical Institute, DGI.

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