Soil/structure interaction study for a piled concrete platform

Interaction sol-structure pour une plateforme en béton sur pieux

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SYNOPSIS The development of the large Troll gas field in the Norwegian sector of the North Sea will, as a result of deep water and very soft soils, require fixed platform solutions different from those used previously in the northern North Sea. The paper describes one of the candidate fixed platforms being considered, the CONDEEP T300. Details are presented related to static and dynamic analysis of soil/structure interaction.

INTRODUCTION

The Troll offshore oil and gas field was discovered by Norske Shell A/S in September 1979. This field, located some 100 km north-west of the city of Bergen in Norway, will be producing by 1995, provided a market can be found for the huge gas quantities. With its estimated $1.6 \cdot 10^{12}$ cubic metres of gas, Troll could for example meet the present UK gas consumption for a period of 30 years.

The possible development of the Troll field represents a major technical challenge as a result of

- Shallow reservoir, located only 1600 m below the surface.
- Water depth of 330 m, close to twice as deep as any North Sea field developed so far.
- Presence of soft, normally consolidated clay from seabed to 24 m depth.

Norske Shell A/S has since the discovery studied a number of fixed and floating platform concepts (Offshore Engineer, 1983; Norwegian Oil Review, 1984) to check their ability to meet the demanding requirements. The present paper describes some of the design studies carried out for one of these concepts, the CONDEEP T300 platform proposed by Norwegian Contractors.

Two alternative T300 platforms have been studied for the Troll field, with and without piles. Only the piled version is addressed below.

SOIL CONDITIONS

Extensive soil investigations have been carried out at the Troll field (Cuckson, 1983; Moeyes and Hackley, 1983). They include undisturbed sampling with X-ray checking of quality, samples maintained under high ambient pressure, and in situ vane and cone penetrometer tests. A typical soil profile is summarized in Fig. 1 (Eide and Andersen, 1984).

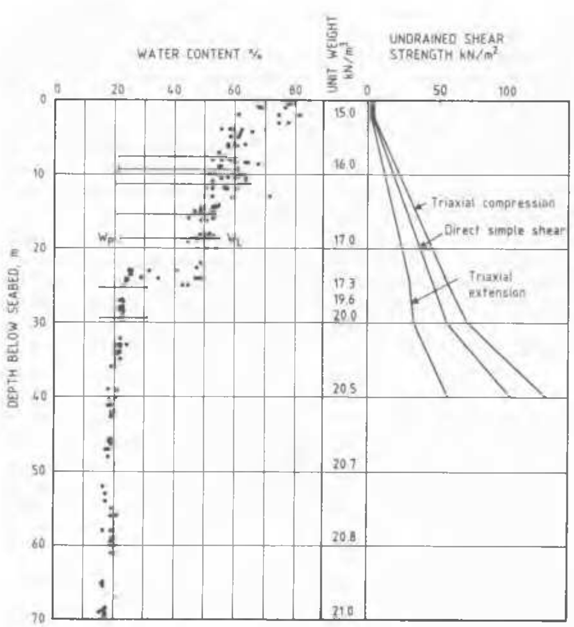


Fig. 1: Summary of Troll Soil Conditions (Eide and Andersen, 1984)

THE CONDEEP T300 CONCEPT

The base of the T300 platform will be built in a dry-dock, then towed to a protected deepwater site for slipforming of the concrete superstructure and mating with the deck structure. On tow-out the draught will be 225 m and the pay load some 45 000 tons (Schjetlein, 1983). Figure 2 shows a sketch of this structure after installation at the field.

The piled version of T300 will have circular steel skirts penetrated 12 m into the soft upper clays to give sufficient stability until the foundation piles have been installed. The concept studied has 75 piles of outer diameter 3.0 m, wall thickness 75 mm and penetration to 80 m below mudline. The piles are placed in 3 groups, 25 piles underneath each foundation pod.

The structural and dynamic analysis and design of T300 is done by Norwegian Contractors with Norwegian Offshore Consultants (NOC) as consultants. The Norwegian Geotechnical Institute (NGI) is the geotechnical consultant. Figure 3 shows in principle the two structural models of the T300 that were analysed as a part of the design studies, a space frame model and a detailed finite element model.

It is the responsibility of the geotechnical consultant to present the boundary conditions for these two models to the designers, such as displacements and stresses at any point of the structure/foundation interface for given loading acting upon the superstructure.

PRINCIPLES FOR INTERACTION ANALYSIS

An intimate interaction between the T300 platform and the foundation will govern both the distribution and the magnitude of the stresses in the platform. The problems associated with reliable analyses of all aspects of this interaction are formidable. There appeared to be no procedure available that was able to handle adequately all of these aspects simultaneously, and it was decided to carry out two separate interaction analyses:

- A static analysis, which aims at giving a detailed picture of the stress distribution in the foundation pods, the piles, the girders between the pods, the lower parts of the legs and the area around the riegel (Fig. 2).
- A dynamic analysis, which aims at giving an overall picture of the dynamic behaviour of the structure, including dynamic amplification factors by which the statically computed stresses are multiplied.

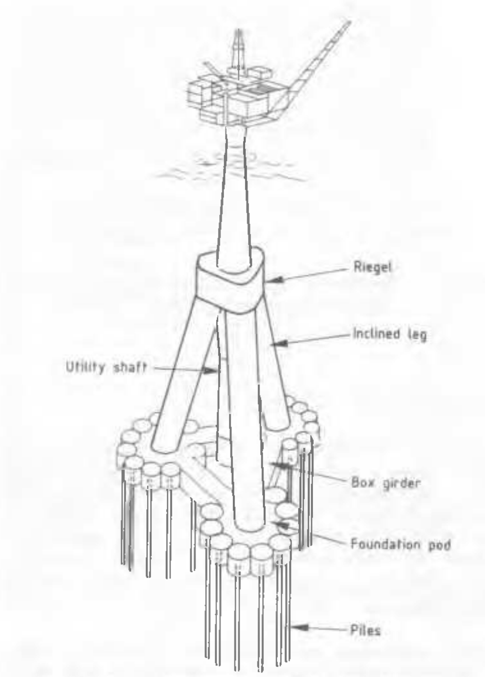


Fig. 2: CONDEEP T300 Platform.

ANALYSIS FOR STATIC INTERACTION

Basic principles

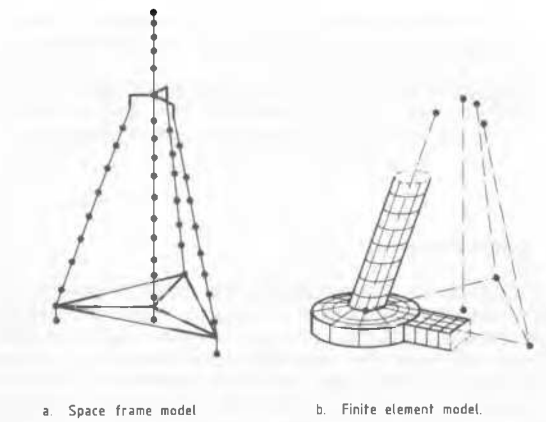
The forces acting upon the structure must be transferred into the supporting soils, partly through the piles, and partly through the platform base areas in direct contact with the soil. The distribution of the support reactions, and hence the stresses in the structure, will depend upon the relative stiffnesses of the different parts of the foundation system and the structure.

It was decided to carry out a foundation analysis that included the following points:

- A fully coupled solution that allows for the stiffness of both superstructure and foundation system.
- Three-dimensional geometry.
- Interaction effects between the various parts of the foundation system (for example pile group effects).
- Non-linear load/displacement behaviour of individual foundation elements (for example local soil yielding along the edge of the platform base).

For a complex system of this type it is not possible to give a single set of conservative soil properties that will govern the required dimensions of all parts of the structure. As an example, one may expect that a "soft" soil will give the highest displacements and thus the highest bending stresses in the piles, whereas a "strong" soil will allow high base edge loads and thus give high bending stresses in the pod.

The analyses of the combined structure/foundation system were therefore carried out for two soil profiles, for simplicity referred to as "soft" and "strong". These two profiles were obtained from one set of characteristic soil stiffnesses and strengths, determined taking the cyclic nature of the wave loading into account.



a. Space frame model b. Finite element model.

Fig. 3: Structural Models of T300

The "soft" soil profile was obtained by dividing the strengths by the partial coefficient of 1.30 required by the Norwegian code (Norwegian Petroleum Directorate, 1977). This code does not give any guide on how to develop the "strong" soil profile. It was therefore decided to simply mulitply the strengths by 1.30 for the "strong" soils.

In addition to uncertainties in soil behaviour, the foundation analysis must also include the variations in platform support conditions that occur with time. The following three support conditions have been considered:

1. The platform has just been placed and the piles not yet installed. Loading is a low submerged platform weight combined with summer storm conditions.

2. The piles have been installed and the platform ballasted to its final weight. This weight is partly carried by the piles and partly by the base. The design 100 year wave then occurs.

3. After some time the submerged weight may be carried by the piles only. The design 100 year wave then occurs.

<u>Linear elastic methods</u>
The structural designers selected 16 interface joints where the space frame model was to be connected to the foundation system, Figure 4. Each of the three pods were assumed to be rigid in the foundation analysis, i.e., the displacements of any point on a pod can be expressed in terms of the six displacements at the pod centre.

The foundation piles were modelled as individual 6 by 6 stiffness matrices representing the relation between pile head forces and displacements. The foundation base and skirts were modelled as a number of plate elements with 3 degrees of freedom at their centre. For this elastic foundation system there exists a linear relationship between displacements **v** and forces **S**:

$$\mathbf{v} = \mathbf{F}\mathbf{S} \tag{1}$$

where **F** is a flexibility matrix. The size of this system of equations will be 3 times the number of plate elements plus 6 times the number of piles. Procedures used to find the different elements in the **F** matrix are explained below.

The elastic relationship between interface joint displacements **p** and sum of foundation element forces w.r.t. the joints $\mathbf{R}_f$, is given by:

$$\mathbf{R}_f = \mathbf{K}_f \cdot \mathbf{p} \tag{2}$$

where $\mathbf{K}_f$ is the stiffness matrix of the foundation system referred to the interface joints. For the superstructure the same type of relationship exists:

$$\mathbf{R}_s = \mathbf{K}_s\ \mathbf{p} \tag{3}$$

where $\mathbf{R}_s$ are superstructure forces referred to the interface joints and $\mathbf{K}_s$ the superstructure condensed stiffness matrix.

Equilibrium of the interface joints require that:

$$\mathbf{R} - \mathbf{R}_f - \mathbf{R}_s = 0 \tag{4}$$
$$\mathbf{R} = (\mathbf{K}_f + \mathbf{K}_s) \cdot \mathbf{p} \tag{5}$$

where **R** is the given load vector acting upon the interface joints when displacements are prevented. Equation (5) is the governing equation for the combined elastic system of superstructure and foundation. With 16 interface joints this system contains 16 • 6 = 96 equations only, compared to say 1500 for Equation (1).

The stiffness matrix $\mathbf{K}_f$ is determined from Equation (1) by a condensation process where 96 unit displacements are assumed for the interface joints, the corresponding vectors **v** computed, and Equation (1) solved for **S**. This results in the matrix $\mathbf{K}_f$ and two other matrices that relate interface joint displacements to pile head and base element forces.

The above are straight forward matrix operations that only require matrix **F** to be known. It consists of the following submatrices:

$\mathbf{F}_{BB}$ = Base element displacements due to base element unit forces.

$\mathbf{F}_{PP}$ = Pile head displacements due to pile head unit forces.

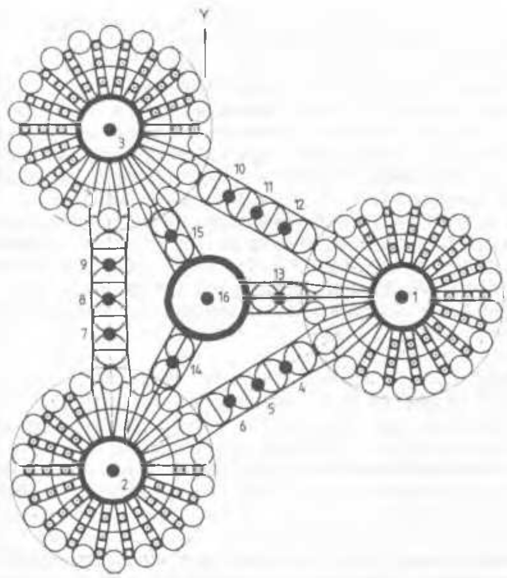


Fig. 4: Coupling Joints between Superstructure and Foundation

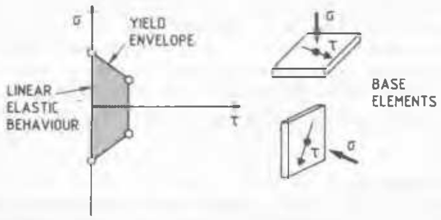


Fig. 5: Base Elements with Yield Envelope

F_{BP} = Base element displacements due to pile head unit forces.

The submatrix F_{BB} is formed by the computer program SPRINT (Clausen, 1983), based upon a numerical integration of Mindlin's point load solutions (Mindlin, 1936). These solutions have been adjusted to approximately account for the effect of increasing soil stiffness with depth.

The submatrices F_{PP} and F_{BP} are computed with the computer program SPLICE (Clausen et al., 1982). The pile is modelled as beam elements embedded in an elastic half space with a soil modulus that increases linearly with depth.

Pile/soil/pile interaction values were actually computed for distances between two piles of 5, 10 and 20 metres. For other distances an interpolation formula for the interaction value I was used:

$$I = A \cdot e^{-B \cdot x} \tag{6}$$

where A and B are constants determined from the SPLICE results, and x is the distance between the two piles. Similar procedures were used for pile/soil/base interaction values.

Non-linear corrections

The above matrix **F** assumes a linear elastic behaviour of the foundation system. After an elastic solution has been found, one could in theory compute a new **F** matrix with adjusted stiffnesses and repeat the condensation process. However, this would be highly impractical and very costly in terms of computer time, as one elastic solution requires of the order 5 hours CPU time on a Prime 750 computer.

The CONDEEP T300 studies were therefore carried out with a basic elastic solution subjected to correction forces to account for non-linear effects. For each base element a yield envelope as indicated on Figure 5 was defined. The maximum value of normal stress was determined from bearing capacity considerations (Lauritzsen and Schjetne, 1976). For each pile an axial and a lateral load/displacement diagram as indicated on Figure 6 was defined. An approximate non-linear solution could then be found by the following steps:

1. Solve the linear elastic system.
2. For each base element and each pile compute the unbalanced forces caused by the element being unable to exceed the yield or load/displacement curve.
3. Form the sum of these unbalanced forces at the interface joints and add to the given vector **R**.
4. Solve the system once more and repeat until convergence has been obtained.

This procedure may tend to overestimate the interaction effects between the different foundation elements, but it will ensure that all elements have resulting forces that are compatible with the strength of the supporting soils, and in equilibrium with the external forces.

Example results

The above numerical model was analysed with 53 different load combinations, 2 soil types and 3 support conditions. This resulted in 189 complete solutions for a detailed structural study. A few (10-15) of these cases were found to be of particular interest, and therefore analysed further by the large finite element model of the structure.

Table 1 gives a summary of some computed results for a "soft" soil case. Values given are displacements and forces for the most heavily loaded pod when the structure is subjected to self weight, and 100 year wave loading in the +X-direction (Fig. 4) multiplied by a load factor of 1.30.

TABLE 1

Pod 1 Computed Results	Linear analysis	Non-linear analysis
Displacements		
δ_x, mm	36	51
δ_z, mm	45	74
θ_{yy}, mrad	0.22	0.38
Vertical Forces		
Piles, MN	780	994
Base Elements, MN	510	284
Horizontal Forces		
Piles, MN	93	100
Base Elements, MN	96	119

The somewhat surprising increase in horizontal shear taken at the pod, is due to local overstressing underneath other parts of the base, with lower bearing capacity than the large leading pod.

The axial pile forces were found to vary between 23 to 48 MN for the linear case, and 37 to 42 MN for the non-linear case.

ANALYSIS FOR DYNAMIC INTERACTION

The dynamic analysis of the T300 platform is based upon a space frame model, where the foundation pile/skirt/soil system is represented by 3 axial and 3 rotational springs under each of the three foundation pods. Experience from performance monitoring of gravity platforms installed in the North Sea shows that even such a simplified model is capable of giving a satisfactory prediction of the overall dynamic behaviour, provided the stiffness in the various parts of the structure is adequately determined (Hansteen, 1979).

Soil model

The regulations of the Norwegian Petroleum Directorate allow use of a linear model in the dynamic analysis of the structure. If non-linear effects have a significant influence on the safety of the structure, they shall be considered.

The dynamic analysis carried out for the T300 platform was a stochastic analysis of the design storm, assuming stationary conditions during the 6-hour storm duration. In such an analysis, the

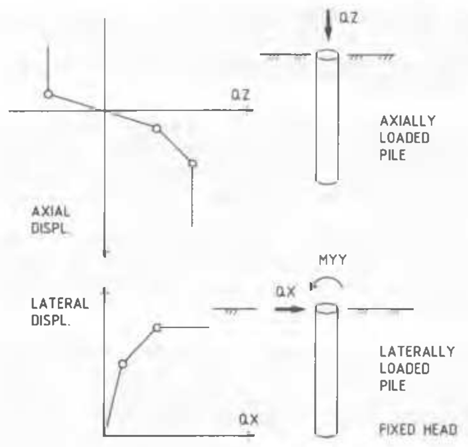


Fig. 6: Axial and Lateral Non-Linear Behaviour of Piles

power density spectrum $S_r(\omega)$ of the response quantity r (r is, say, a stress component) and its standard deviation σ_r is computed. The expected maximum value r_{max} is found as

$$r_{max} = f \cdot \sigma_r \tag{7}$$

where f is computed from the power density spectrum, and is normally in the range 3.5-4.0.

In determining the stiffnesses of the foundation springs, it is the cyclic stiffness (secant modulus in hysteresis loop) that is of interest. This modulus is strongly dependent upon the cyclic stress amplitude. When a large number of stress cycles occurs, it is also necessary to consider the decrease in modulus with increase in number of stress cycles, due to pore pressure build-up. A method to predict the development of strain amplitudes in clays subjected to a large number of stress cycles of variable amplitude (Andersen, 1976; Andersen, 1983) was used to compute the appropriate secant shear modulus for the soil elements. In principle, the procedure implies that each soil element is subjected to stress cycles corresponding to the stresses during the design storm. In the present case, the design storm consisted of altogether 1800 load cycles, ranging from 900 cycles with 20% of the maximum amplitude to 1 cycle with 100% of the maximum amplitude. The maximum amplitude varies from element to element. The equivalent linear secant modulus was then determined as the ratio between the standard deviation of the stress and the standard deviation of the corresponding strain during the storm. This definition of the equivalent modulus ensures that the total energy of the computed displacements are reasonable, even if the frequency distribution (shape of the power spectrum) may be somewhat inaccurate due to non-linearities.

Computer implementation

The soil model outlined above has been built into NGI's non-linear finite element program FEAST84, which was used to compute the foundation spring stiffnesses. This program can only handle a two-dimensional plane strain model. The computational model used had the same base area and the same moment of inertia as the complete T300 foundation, and also contained the axial effect of piles with the same moment of inertia about the platform centreline as the T300 piles.

Horizontal loads and moments corresponding to the standard deviation of the loads during the design storm were applied in steps, and one iteration was carried out for each step. An equivalent horizontal spring stiffness for the whole platform was then found as the ratio between applied horizontal load and the resulting horizontal displacement, and a similar rotational spring as the ratio between moment and rotation.

The total rotational stiffness of the T300 foundation results partly from the vertical axial springs under the pods, and partly from the rotational springs under each pod. From static elastic halfspace solutions, it was estimated that the first of the above contributions accounted for 70% of the total rotational stiffness. From this assumption, the stiffness of all the axial springs and the rotational springs about horizontal axes could be computed. The stiffnesses of the rotational springs about the vertical axes have a negligible influence on the computed dynamic response.

Results

The computed dynamic axial spring stiffnesses under each pod were 6500 MN/m horizontally, and 43000 MN/m vertically. For comparison, the non-linear results from the static analysis given above correspond to an apparent horizontal spring stiffness of 4295 MN/m. The static equivalent vertical stiffness, which also includes the immediate settlement due to the submerged weight of the platform, is 17300 MN/m.

The difference in equivalent stiffnesses between the two procedures is significant. Note that the "static" values represent a much higher load level than the "dynamic" values. Also, the static analysis was primarily aimed at obtaining the distribution of soil reactions under the platform, while the dynamic analysis aimed at obtaining representative displacements of the platform during the design storm.

In the subsequent dynamic analysis, the first natural period of the platform was found to be 4.0 sec. If the softer static spring stiffnesses had been used, a significantly higher natural period would have been predicted, giving rise to higher dynamic amplification of the wave forces.

CONCLUSIONS

The paper has summarized the procedures used to analyse the interaction between superstructure and foundation for the piled version of the CONDEEP T300 proposed for installation at the Troll field.

It is the opinion of the authors that for complex foundations of this type, it is necessary to undertake separate interaction analyses, each focusing on interaction effects of particular significance for the subsequent analyses to be carried out. It is also necessary to analyse a wide range of possible soil behaviour and support conditions, as it is highly unlikely that one critical combination will govern the design of all superstructure members.

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