SKIRTED FOOTINGS CAPACITY FOR COMBINED LOADS AND LAYERED SOIL CONDITIONS

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SUMMARY: Conventional and numerical, finite element soil - foundation interaction modelling is carried out for the world's largest three-leg jack-up, skirted footings resting on layered soil conditions consisting of sand overlying clay with varying strength. The footings are subjected to general combined vertical V, horizontal H and moment M loadings. Differences between the yield capacities calculated from different methods are discussed, a design yield envelope is proposed and some experience and recommendations for offshore foundation design applicable to similar soil conditions are drawn.
Keywords: Conventional analysis, combined loads, finite element (FE), jack-up structure, yields capacity, skirted spudcan,

INTRODUCTION

Offshore structures are often supported on multiple foundations. Design of such foundations is generally based on their capacity / integrity under monotonic combined vertical V, moment M and horizontal H load as limiting conditions due to environmental impacts and other factors during the operation.

In order to predict the behaviour of an offshore structure such as a jack-up rig the response of these individual foundations subjected to combined loading should be understood.

The present industry guidelines such as [1, 2] applicable to offshore foundations are generally based on the theoretical bearing capacity solution for failure of a strip /

two-dimensional (2D) footing under vertical load. Alternative foundation geometry, embedment, load inclination and eccentricity, are accounted for by various modification factors and effective / equivalent area method [3, 4].

However, the conventional approach is not always suitable for conditions when V, H and M loads act together on a three-dimensional (3D) footing geometry located on layered soil conditions, which are often encountered offshore.

Furthermore, jack-up rig foundations are in some cases equipped with outer and internal skirts, which penetrate the seabed during installation confining a soil plug. The skirt enhances additional moment capacity, which increases due to suction developed within the skirt during moment loading.

Skirted footings under combined loadings have been numerically investigated [5, 6, 7] for clay soil with constant and increasing undrained shear strength c_u with depth. Not much investigation is however, performed for footings resting on sand overlying clay.

Ultimate capacities of skirted footings under combined loadings and layered soil conditions were calculated by 2D and 3D finite element (FE) analyses [8, 9], which are applicable to jack-up rig installations in the North Sea.

It is shown in the following that the conventional approach based on the effective footing area A' and bearing capacity for inclined load applied at the centre of A', depending on the assumptions made may underestimate footing capacity when the soil profile consists of sand over clay.

Aiming for a more safe design conventional and 2D, 3D FE skirted footing-soil interaction analyses under (V, H, M) loadings are carried out for the world's largest jack up foundations in the North Sea. The differences in the failure mechanisms, ultimate capacities and nonlinear stiffness between the methods applied are discussed.

FOUNDATION ANALYSIS OF THE JACK-UP RIG

Rotational stiffness or foundation fixity of jack-up footings is an important aspect in the overall structural behaviour. Therefore, soil-foundation interaction effect is required in the structural design predicting the fixity.

In the following initial structural analyses including weight, environmental loads and soil conditions as expected at the site are carried out predicting the combined loads on the foundations. The interpretation of the soil conditions at the jack-up location is carried out.

For the rig installation, footing penetration is initially predicted. No risk for punch through / rapid penetration is expected at the site and skirted spudcan penetrations corresponding to full base contact are assumed.

Skirted spudcan geometry
The jack-up footings have approximate diameter D = 22 m and are fitted with outer and internal skirts, which divide the spudcan into 6 compartments.

The main vertical geometry of the spudcan structure is: distance from spudcan base to tip of outer skirts 2.3 m; distance from spudcan base to tip of internal skirts 1.1 m;

The spudcan itself is a flat rigid plate. A drawing is given in Fig. 1. The transverse stiffnesses of the skirts are derived from the structural FE model of the spudcan.

Based on such model equivalent wall thicknesses are calculated for the outer skirt and the internal skirts. These thicknesses are applied in the 2D and 3D FE analyses employing beam and wall structural elements, respectively.
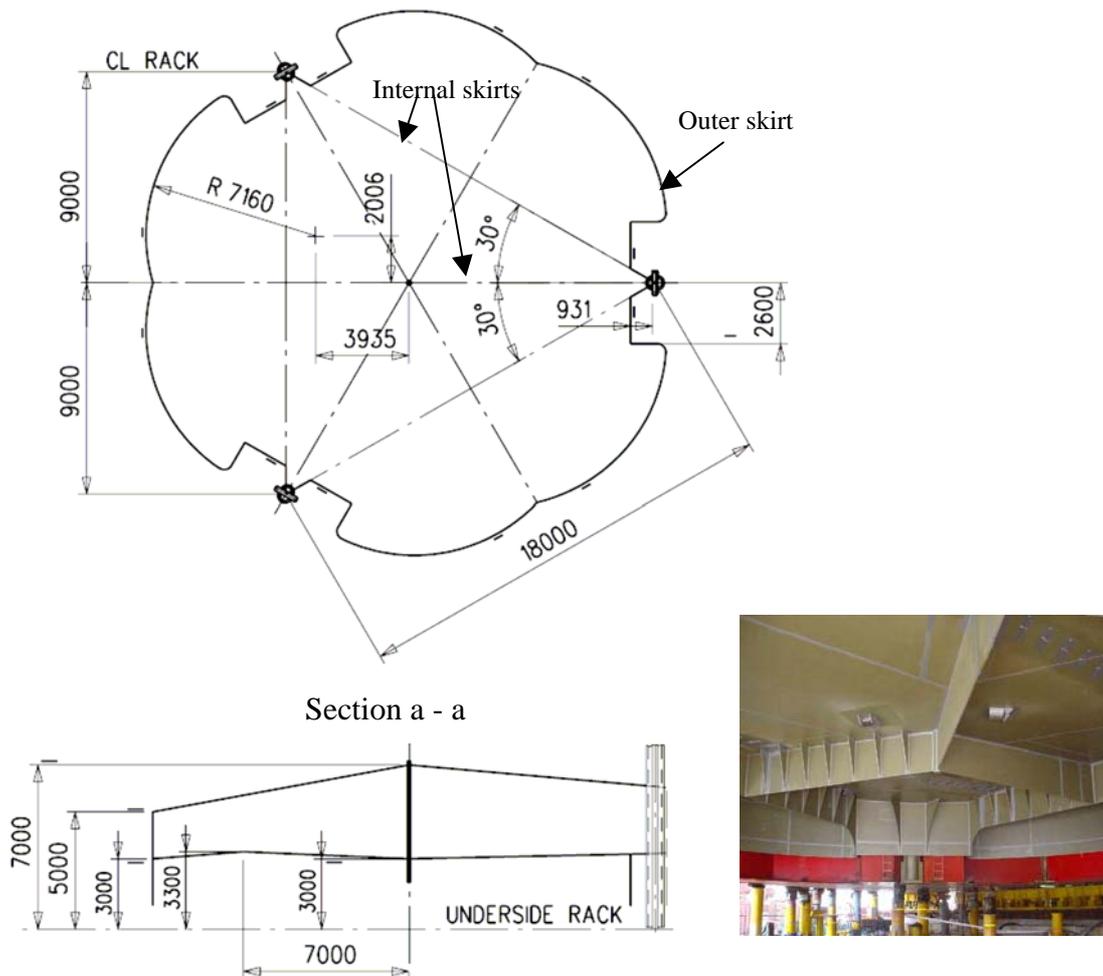
Section a - a

Fig. 1. *Skirted spudcan geometry*

Soil conditions, design soil profile

Two continuous piezocone penetration tests (PCPTs), one sampling, borehole (BH) to 30 m depth and laboratory testing are carried out at the location. The soil conditions at the three BH / CPTs appear to be generally similar. At the seabed they reveal very dense, (relative density $D_r > 80\%$), fine sand with shell fragments and organic material to (10 - 11) m depth. Firm to stiff sandy clays are found below this depth.

The interpreted design soil profile, generalized to the three-spudcan locations, with the strength and deformation parameters as applied in the 2D and 3D FE analyses, are given in Table 1. The soil strength parameters given by Kellezi et al, [9] are converted to design parameters applying the DNV [1] material coefficients $\gamma_m = 1.2$ for the tangent of the friction angle and $\gamma_m = 1.3$ for the undrained shear strength.

It should be noted that not the same safety applies to the strength parameters for the sand and the clay soil using the DNV material coefficients. Larger safety applies to sand than to clay parameters

Regarding the Young's modulus E for the clay layers a correlation $E = N*c_u$ has been applied where N the correlation factor is chosen based on the experiences in the North Sea.

The deformation parameters (the elastic stiffness) the same as recommended by Kellezi et al. [9] are applied assuming that the elastic behaviour or the path to yield is the same

for the soil layers at the location.

Table 1. *Design soil profile applied to 2D and 3D FE analyses*

Soil description	Depth of layer h (m)	Unit weight γ' (kN/m^3)	Angle of internal friction and dilation		Undrained shear strength c_u (kN/m^2)	Deformation Module E (kN / m^2)
			φ (°)	ψ(°)		
SAND, medium dense to dense.	0.0 – 2.3	10.0	35	5	-	17500+25000*z/m
SAND, medium dense to dense.	2.3 – 10.5	10.0	35	5	-	75000
CLAY, firm	10.5 – 11.2	10.0	-	-	69.2	18000
CLAY, stiff	11.2 – 15.0	11.0	-	-	96.2	25000
CLAY, stiff	15.0 – 20.0	11.0	-	-	76.9	20000
CLAY, firm	20.0 – 27.0	11.0	-	-	57.5	15000
CLAY, stiff	27.0 – 30.0	10.0	-	-	80.8	21000

Design storm loads

Based on the initial footing 3D yield capacities calculated by Kellezi et al, [9] and reducing them by a coefficient of 1.3, and applying the same elastic-plastic stiffness the soil-structure interaction of the jack-up structure is re-evaluated.

For the current jack-up the maximum preload is 194 MN / leg. With the weight and buoyancy of the legs and spudcans (no soil plug), approximate design pseudo static loads are derived from the structural model where the spudcan and the surrounding soil are modelled as elastic springs, reflecting rotation and displacements.

Final ultimate limit state design (pseudo-static storm) combined loads are defined from simple iterative structural and soil-foundation analyses as shown in Table 2.

Table 2. *Design factored quasi-static storm combined loads*

Design load combinations (LC 1 – 5)	1	2	3	4	5
V (MN)	195.0	140.0	67.0	44.0	35.5
H (MN)	15.00	15.00	13.50	13.50	12.84
M (MNm)	650	650	450	325	255
Associated static load V (no storm loading) and storm load increase δV					
V, static (MN)	145	145	90	90	90
δV, storm (MN)	50.0	-5.0	-23.0	-46.0	-54.5

The large differences in the V, M loads between the jack-up aft legs, load combination (LC 1, 2) and bowleg (LC 3, 4, 5) is due to the eccentricity of the hull's centre of gravity.

The static load 'V,static (no storm load)' is applied after preloading to 194 MN by unloading to V = V,static. So, two different values appear for this unloading stage. It must either be 145 MN or 90 MN, depending on the LC or the type of the footing leg, aft or bowleg.

The 'dV, storm' is thus the storm load increase or decrease relative to V, static to be applied together with H and M, leading to the final storm load (V, H, M) combination.

Conventional bearing capacity analyses

Conventional method is first applied based on Hansen's work [3] for calculation of the footing ultimate capacity in sliding and moment loading. The method is developed for a block embedded footing, confined by the skirt, resting on sand over clay, (with average c_u = 77 kPa,), soil profile.

The eccentricity is first calculated. Based on this, an effective footing area is defined as given in Fig. 2 [3]. The small fictive footing is then projected to the clay below the sand with an inclined spreading pressure responding to the direction of the inclined load (resultant of V and H).
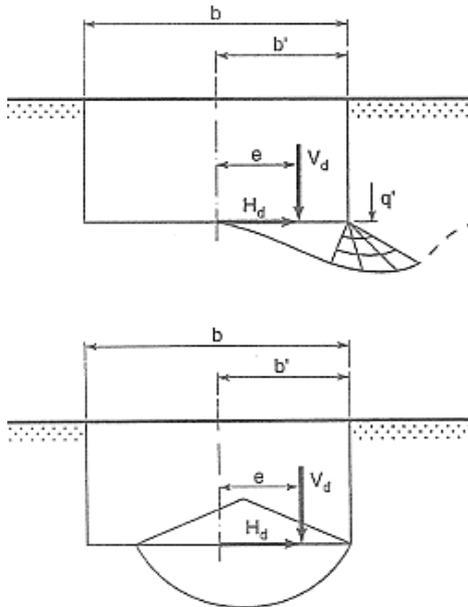

Fig. 2. *Effective area for circular footing*

Fig. 3. *Alternative failure mechanisms under footing with large eccentricity*

The results are given in Table 3. Values for H_{yield} are derived for constant V as given in Table 2 and zero eccentricity. Values for M_{yield} are derived for constant V and H as given in Table 2. However, there are limitations in the conventional analyses reflected in the simplified soil profile, footing geometry and particularly the assumption on the critical failure mechanisms of the type given in Fig. 3.

The critical failure assumes a sliding failure of the small or projected footing with effective area. This seems to be realistic when the ultimate capacity is determined from the sand layer and not realistic when deep failure occurs and the ultimate capacity is determined from the clay layers.

Table 3. *3D conventionally calculated design LB yield capacities*

Design load combinations (LCs)		1	2	3	4	5
Conventional Design Yield	H_{yield} (MN)	67	68	44	34	30
	M_{yield} (MNm)	487	654	543	381	323

2D and 3D FE modelling

Generally for layered soil profiles, such as found at the considered location FE method is

preferred, employing non-linear constitutive soil models able to seek the critical failure surfaces as part of the analyses.

2D FE modelling is carried out first. As footings showed limited strength and stiffness 3D modelling is performed as the main calculation method.

The following phases are calculated: Assessment of the initial stress conditions; Footing installation and preloading; Unloading to V,static; Application of quasi-static combined (V, H, M) storm loadings; Assessment of the ultimate H capacity in the (V–H) plane for V constant; Assessment of the ultimate M capacity in the (V-H-M) plane for V, H constant;

In a 2D plane strain FE modelling, one commonly used method [1] is transforming the footing circular area into a rectangular one with the same area and moment of inertia. The 2D FE analyses provide in general quick results but tend to underestimate capacities and stiffness due to representation of the circular spudcan by a strip footing. The beneficial effects of the skirts cannot be accurately accounted for.

2D and 3D FE analyses are carried out with the Plaxis FE programs [10, 11]. Considering the available geotechnical data, the Mohr Coulomb elastic-plastic constitutive soil model is applied assuming drained conditions for the sand and undrained conditions for the clay. Hardening soil model is also applied but not chosen for the design, as a lower bound solution was of interest for the current analyses.

Finer and coarser mesh is applied. The results for coarser mesh are used in the following to compare with the 3D FE analyses where the mesh cannot be very fine.

The 2D FE results show that aft leg footings cannot support LC1 giving an indication that this LC is probably close to the footing H or M yield capacities. Footing bearing capacity is on the other hand fully satisfied for the other LCs. Compared to the 3D results the deformations in 2D FE analyses are much larger and the failure loads differ.

In the 3D FE analyses the footing – soil interaction is modelled as shown in Fig 4. The skirted spudcan is very similar to the real footing geometry shown in Fig. 1. Wall structural elements are applied for the outer and internal skirts and plate structural elements are used for the spudcan with an average thickness allowing full base contact.
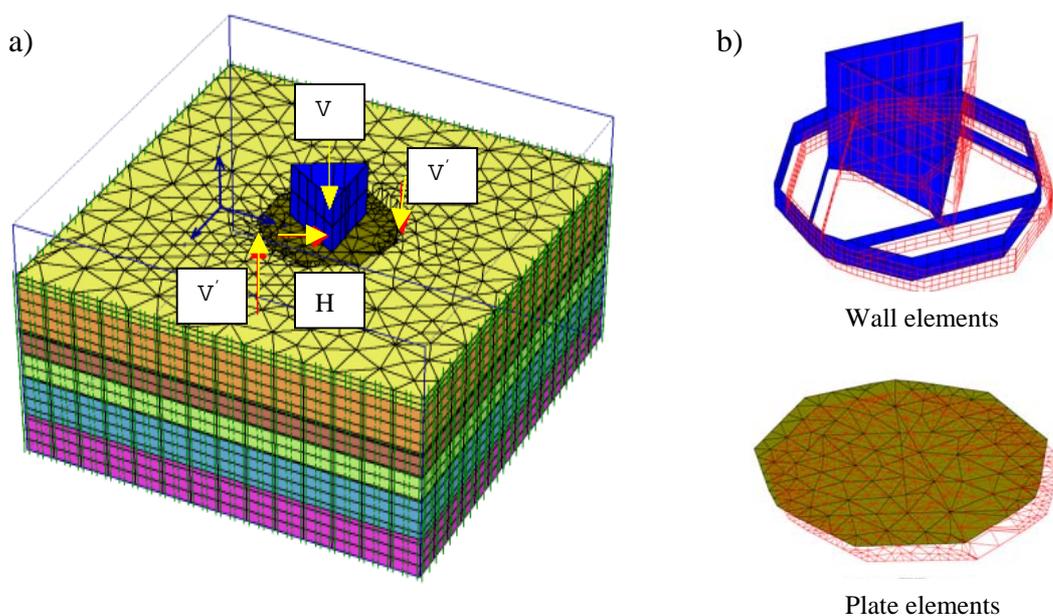

Fig. 4. *3D FE model (a) Soil-skirted spudcan system (b) Skirted spudcan, wall and plate elements*

The 3D FE model is built up in horizontal planes, whose elevations correspond to the start and the end of the footing elements in the y-direction. Tetrahedron 15-noded finite elements are used for the soil. Strength reduction is applied in the footing-soil interface.

The combined loads are applied as given in Fig. 4. Standard boundary conditions are incorporated at the far field. The size of the model in the two horizontal and the vertical directions is chosen so that the boundaries will not affect the full development of the failure mechanisms. From the 3D FE analyses the foundation bearing capacity is satisfied for all 5 LCs. Some results, (failure figures) from the 2D and 3D FE analyses for (V, H, M) working loads given in Table 2 are shown as total displacement increments in Fig. 5. The values have no physical meaning. They are only used for scaling.
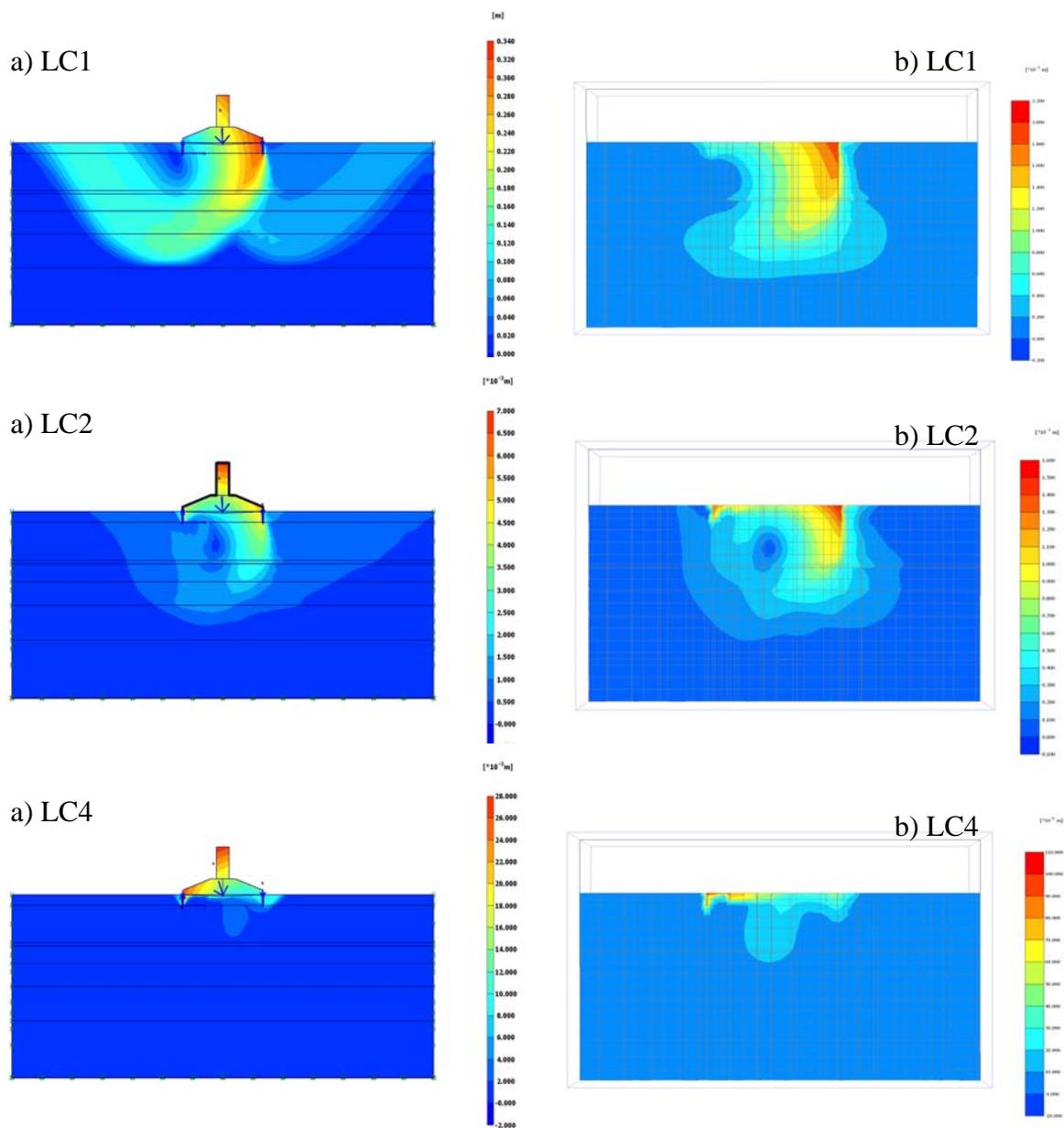

Fig. 5. *Failure figures (total displacement increments) under working loads (V-H-M) for a) 2D FE analyses b) 3D FE analyses*

The 3D FE designed yield capacities for sliding (with constant V as given in Table 2 and zero eccentricity), and moment loading (with constant V and H as in Table 2), are also calculated. The results are summarized in Table 4. However, the moment yield capacities for the LC1, corresponding to the largest vertical load, could not be assessed applying (V, H, M) load path. Some results are derived increasing (V, H, M) by a ratio.

The moment yield capacity M_{yield} for LC1 is defined for V and H as given in Table 4. The large value of the moment yield capacity provokes large footing deformations.

The failure figure, horizontal displacement increments at H yield, and total displacement increments at M yield, for two chosen load combinations is given below in Fig. 6. The values have no physical meaning. They are only used for scaling.

Table 4. *3D FE calculated design yield capacities, (No deformation criterion applied)*

Design load combinations (LCs)		1	2	3	4	5
3D FE Design Yield	H_{yield} (MN)	108	90	52	39	34
	M_{yield} (MNm)	(V_{yield} = 226 H_{yield} = 24.2) 1048*	1020	583	427	362

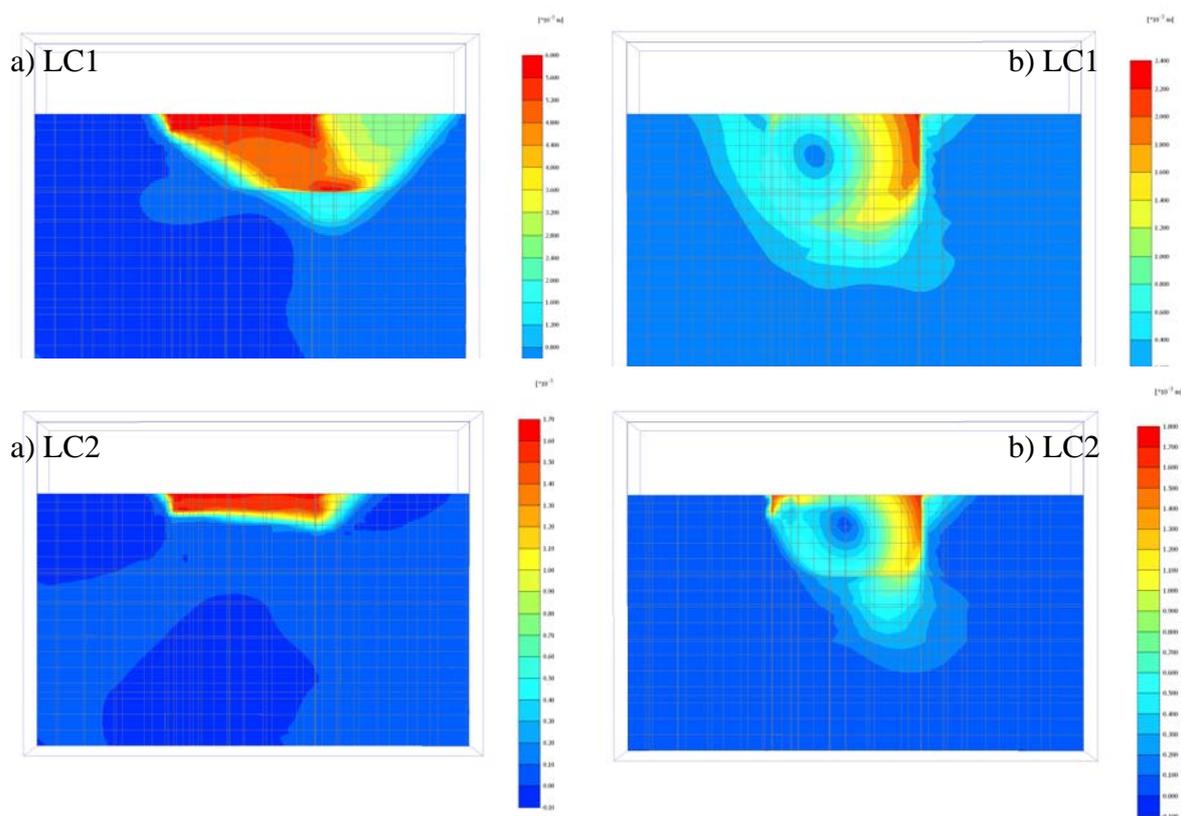*M_{yield} could be calculated only when increasing by a ration V, H and M*

Fig. 6. *3D FE failure figures at yield*
a) H yield, (horizontal displacement increments) b) M yield, (total displacement increments)

LC1 and LC2 are chosen as they develop different failure mechanisms at yield. LC3 - LC5 are not shown as their failure mechanism at H yield is similar to Fig. 6a for LC2

(sliding) and at M yield similar to Fig. 5b for LC4.

Hardening behaviour is observed in the stiffness curves, (load-displacement relationship at a point in the footing) for LC1, which means that considerable increase in the footing capacity is achieved while deformations increase. It seems that the yield envelope expands with the increasing loads, while the moving soil volume increases. A deformation criterion is applied in this case to assess the footing capacity.

The stiffness curves for a point at the footing centre are given for LC2 and LC5 in Fig. 7 where the parameter Sum-Mstage is the ratio of the load level reached during the calculation stage and the load applied. Large loads are applied to reach the yield values.
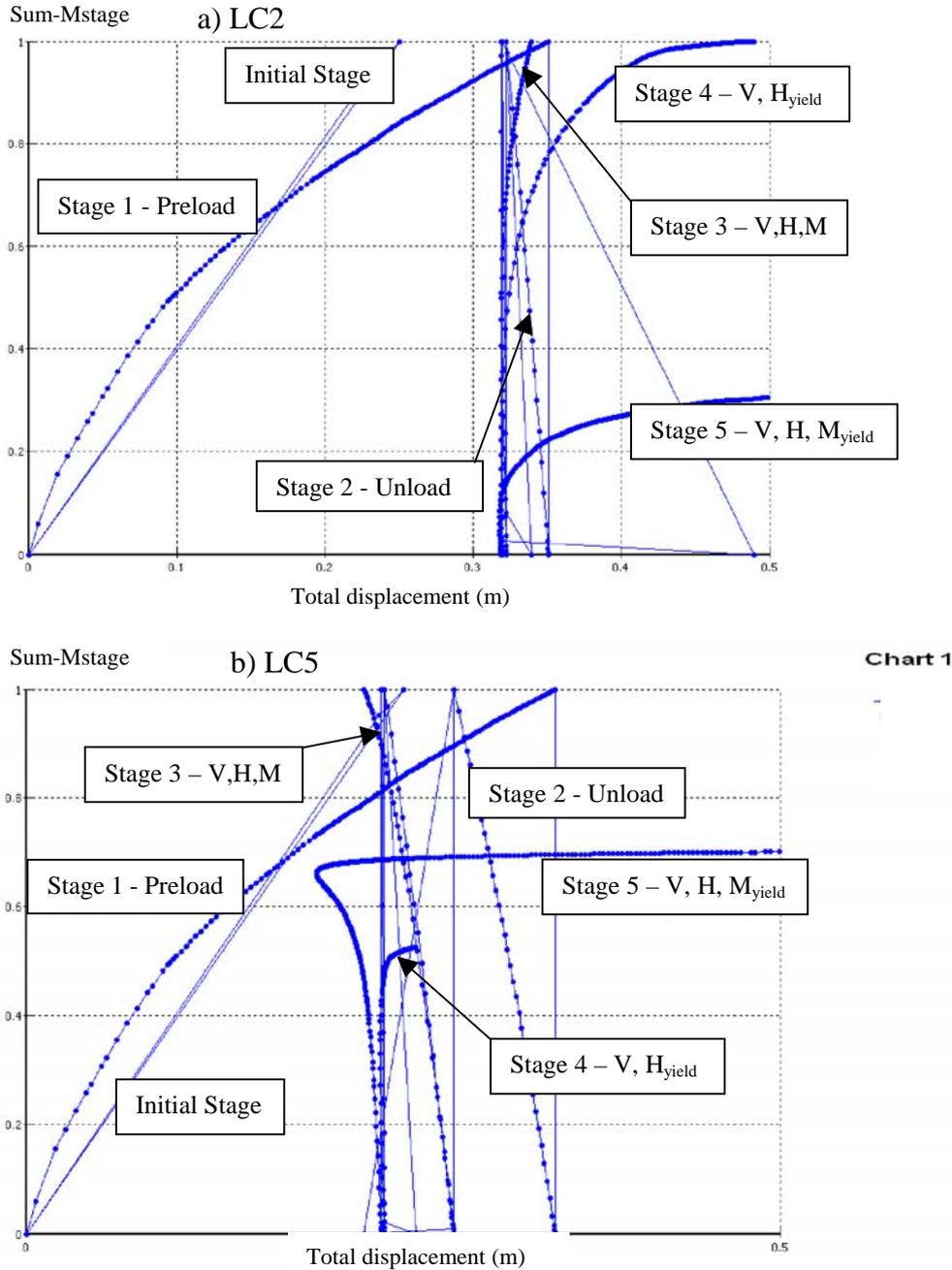

Fig. 7. *Stiffness curves, central point, total displacement versus load for a) LC2 b) LC5*

Elastic-plastic behaviour is observed in the stiffness curves for LC2 and LC3 and elastic-perfectly plastic behaviour is observed in the stiffness curves for LC4 and LC5 respectively. Hence for small or large deformations the yield capacity does not change much and the deformation criterion is not very important.

COMPARISON OF CONVENTIONAL AND FE ANALYSES

In general the conventional, 2D FE and the 3D FE yield capacities differ from each other depending on the LCs. The conventional analyses are however, carried out for circular footing. Therefore the results from these analyses should be compared with the 3D ones.

Ḟhe conventional and 3D FE sliding and moment yield capacities are plotted in Fig. 8a and b respectively. The FE values correspond to $\alpha=1$ [1], which means that those are the maximum loads footings can resist disregarding any deformation criterion. Such a criterion is not relevant for the conventional analyses representing the ultimate footing capacity.

Fig. 8 represents part of (V-H) and (V-M) for H constant design yield envelopes, respectively.
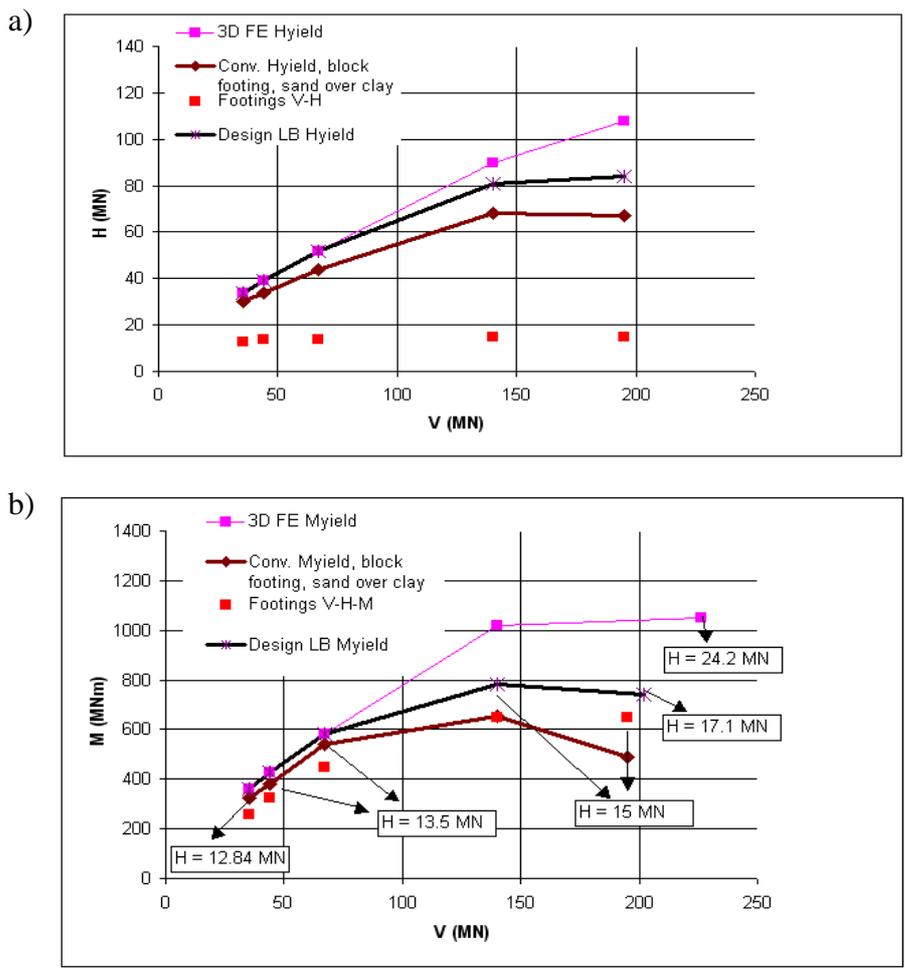
a)

b)

Fig. 8. *(a) 3D ultimate sliding capacity (b) 3D ultimate moment capacity*

The footings combined loads fall inside the 3D FE yield envelope but partly outside

the conventional (V-M) yield envelope.

The differences in the 3D FE and 3D conventional yield capacities, the latter in the current case being always smaller than the 3D FE, is explained with the predefined failure mechanism in the conventional method. This mechanism is found to be nearly similar for (LC3 – LC5) and different for (LC1 - LC2), depending on the V for sliding capacity and (V – H) for M capacity.

Based on the 3D FE analyses applying deformation criterion, and conventional analyses considering the restrictions regarding the failure mechanisms, 'Design LB H_{yield}' and 'Design LB M_{yield}' are proposed for sliding and moment loadings where LB stands for lower bound. The capacities are listed in Table 5 for each design combined loads scenario, respectively. H_{yield} correspond to constant V and zero eccentricity and M_{yield} are derived for constant V and H as shown in Fig. 8.

Table 5. Design yield capacities

Design load combinations (LCs)		1	2	3	4	5
Design Yield	H_{yield} (MN)	84	81	52	39	34
	M_{yield} (MNm)	740	780	583	427	362

Footing-soil nonlinear stiffness

The horizontal, vertical and the rotational foundation stiffness, applied in the structural analyses, are derived direct from the 3D FE soil-footing modelling.

The rotation stiffness, which together with M comprise footing fixity, is derived from the vertical stiffness for the left and the right points in the footing circumference. The 3D stiffness differs from the 2D one and both are mesh and deformation parameter dependent.

FE Investigated Failure Modes

Various failure modes are investigated for different LCs from the 2D and 3D FE analyses. Critical failure figures depend on the load components and the soil conditions. These failure modes can be identified in Fig. 5 and 6.

Typical failure modes encountered are: sliding along the soft clay layer below skirt tip; sliding along base of skirt tip; sliding at base with local failure around skirt tips; conventional deep-seated bearing failure;

CONCLUSIONS

Conventional, 2D and 3D FE soil-skirted spudcan interaction modelling under five design general load combinations is carried out for the world's largest jack-up rig installed in the North Sea. The soil conditions consist of very dense sand overlying firm / stiff clay with varying strength.

Conventional analyses further developed for block / embedded footings and a soil profile consisting of the sand overlying clay with constant c_u give yield capacities smaller than 3D FE analyses depending on the (V-H) and (V-M) variation and the applied clay strength.

As expected, 2D FE analyses give conservative capacities compared to the 3D ones. However, the differences in the capacities are marginal when the 2D failure mechanism is

located at the footing area.

These results are valid for the current analyses and cannot be generalized. The yield capacities are not strongly dependent of the mesh refinement. The stiffnesses though are meshing dependent and should be evaluated by an experienced engineer.

2D FE analyses are recommended for initial design of jack-up skirted footings resting on layered soil profile. Depending on the developed failure mechanism (shallow / deep) application of the 3D FE modelling is recommended for the final design.

Based on conventional and 3D FE analyses, 'Design LB H_{yield}' and ' Design LB M_{yield}' envelopes are proposed. Deformation criterion is applied to the 3D FE results and the limitations of the conventional analyses with regard to averaging the strength of the clay layers and assumption on the failure figure are taken into account.

As all 5 considered LCs fall within the design LB yield capacities and footings deformations are acceptable, skirted spudcans bearing capacities / integrities are considered satisfactory. The rig is currently operating in the North Sea.

ACKNOWLEDGEMENT

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