Numerical analyses of a skirted foundation under tensile load

Susan Gourvenec
Centre for Offshore Foundations Systems, University of Western Australia, Perth, Western Australia

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ABSTRACT

Skirted foundations are becoming an increasingly popular foundation solution offshore as a practical and economical alternative to piles for jackets and as moorings for floating facilities. In this paper, results from finite element analyses investigating the response of a skirted foundation under transient and sustained tensile loads are presented. A hypothetical case study is considered of a circular skirted foundation with a skirt depth to foundation diameter aspect ratio, d/D, of 0.5. The investigation was carried out in axisymmetric conditions and compared with plane strain conditions to highlight some of the differences incurred with geometry idealisation.

1 BACKGROUND

A skirted foundation is a plate with a thin peripheral skirt that penetrates into the seabed confining a soil plug (Figure 1). Skirted foundations have a variety of applications offshore (Figure 2) and are becoming increasingly attractive due to their economic advantage over conventional pile foundations. One of the key benefits of a skirted foundation over a conventionally embedded shallow foundation lies in its enhanced uplift resistance. This is particularly beneficial offshore where harsh environmental conditions impose significant overturning actions to fixed-bottom structures, and floating structures are subject to increased tension loads in extreme weather conditions.

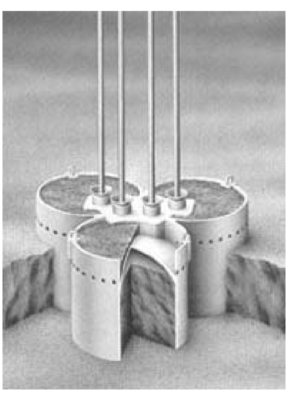


Figure 1: Skirted foundation

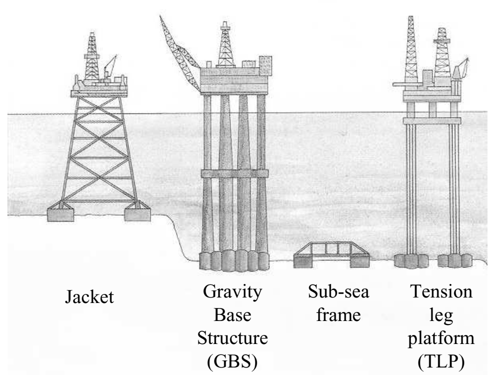


Figure 2: Applications for skirted foundations

During moment or tensile loading, suctions are developed within the soil plug confined by the skirts providing temporary uplift resistance. Under sustained tensile load suctions will gradually dissipate as the hydraulic gradient set up by the uplift load causes water to flow into the soil plug. The dissipation of the passive suction leads to a gradual loss of foundation capacity with duration of the sustained load. While passive suction can be relied on a general shear failure (reverse end bearing capacity failure) can be mobilised. When passive suction cannot be relied on the ultimate limit state of the foundation will be defined by sliding pullout resistance; given by the lesser of the sum of external and internal shaft friction or external shaft friction and the weight of the soil plug. Over the duration of a sustained load a transition from a general shear failure to a sliding pullout failure will occur. Sliding pullout resistance can be several fold less than reverse end bearing capacity and therefore it is important to ascertain the duration over which passive suctions can be relied on.

Current design guidelines for offshore shallow foundation systems (DNV 1992, API 2000, ISO 2002) acknowledge that increased capacity can be mobilised under transient loading due to passive suctions developed within the soil plug of skirted foundations, although to date no formal guidelines exist concerning the duration over which enhanced uplift capacity can be relied on.

2 THIS STUDY

A programme of transient and sustained uplift analyses was carried out to assess the duration over which passive suctions could be relied on to enhance the uplift capacity of a skirted foundation. A hypothetical case study is presented of a circular skirted foundation with a skirt depth to foundation diameter aspect ratio, d/D = 0.5. All analyses were carried out with the software ABAQUS (HKS 2006).

2.1 Finite element mesh

The model represents a foundation of diameter D = 10 m, skirt depth d = 5 m as a discrete rigid body within a mesh of deformable fully-coupled continuum elements representing the soil mass. The foundation is fully bonded along the interface with the adjacent soil elements with slip and separation prevented. The external boundaries of the mesh are located a distance of 5D to either side of and beneath the foundation, sufficiently remote that the response of the foundation is unaffected. Zero-displacement boundary conditions prevent out-of-plane displacements of the vertical boundaries, and the base of the mesh is fixed in all three coordinate directions. Axisymmetry was used to represent a circular foundation. A similar plane strain mesh was constructed to compare the effect of geometry idealisations on the predicted foundation response. Second order reduced integration elements were used in both the axisymmetric and plane strain analyses. In the consolidation analyses fully coupled Biot-type stress-pore fluid elements were used.

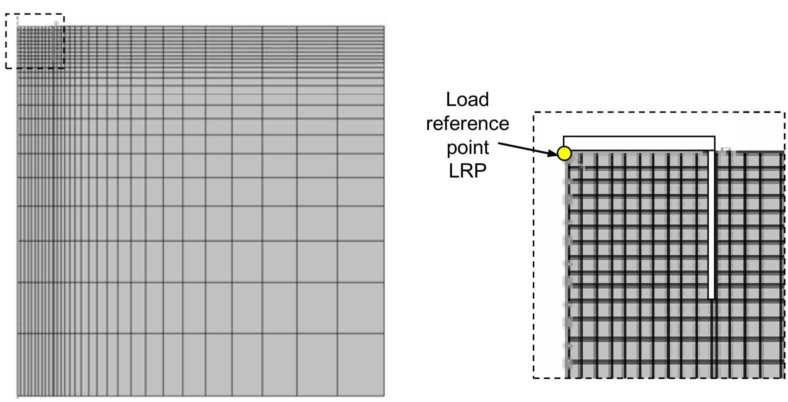


Figure 3: Finite element mesh

2.2 Material properties

A linear elastic Tresca plastic constitutive model was adopted with a uniform shear strength s_u and constant stiffness index E_u/s_u = 1000. Poisson's ratio ν_u = 0.49 or ν' = 0.3 was prescribed in the undrained and consolidation analyses respectively. A coefficient of permeability k = 10E-5 m/day and initial void ratio e_o = 1.5 defined the consolidation response. The unit weight of the soil was taken as γ = 20 kN/m^3.

2.3 Scope and loading method

Undrained bearing capacity, V_u, was identified through displacement controlled tests to enable the post-failure response to be observed. Sustained loading tests were load-controlled, applying a constant load defined in terms of a proportion of the undrained uplift capacity. Each load was applied over 0.1 day and maintained until negative excess pore water pressures had dissipated and displacement ceased. Loads and displacements were applied to the foundation load reference point (LRP) located on the underside of the foundation base plate along the axis of symmetry.

3 RESULTS

3.1 Validation of FE model

3.1.1 Elastic settlement

The vertical surface displacement of an infinite elastic half space beneath a uniform circular pattern of loading is given by Equation 1(Boussinesq 1885):

$$w^{e}_{z=0} = \frac{D(1-\nu^2)}{E} q \tag{1}$$

where ν and E are the elastic parameters, D is the diameter of the loaded area and q the magnitude of the applied load. A surface load applied to an axisymmetric mesh of same discretisation, and using the same element type and elastic material properties as adopted for the skirted foundation model resulted in surface displacement in good agreement with Equation 1.

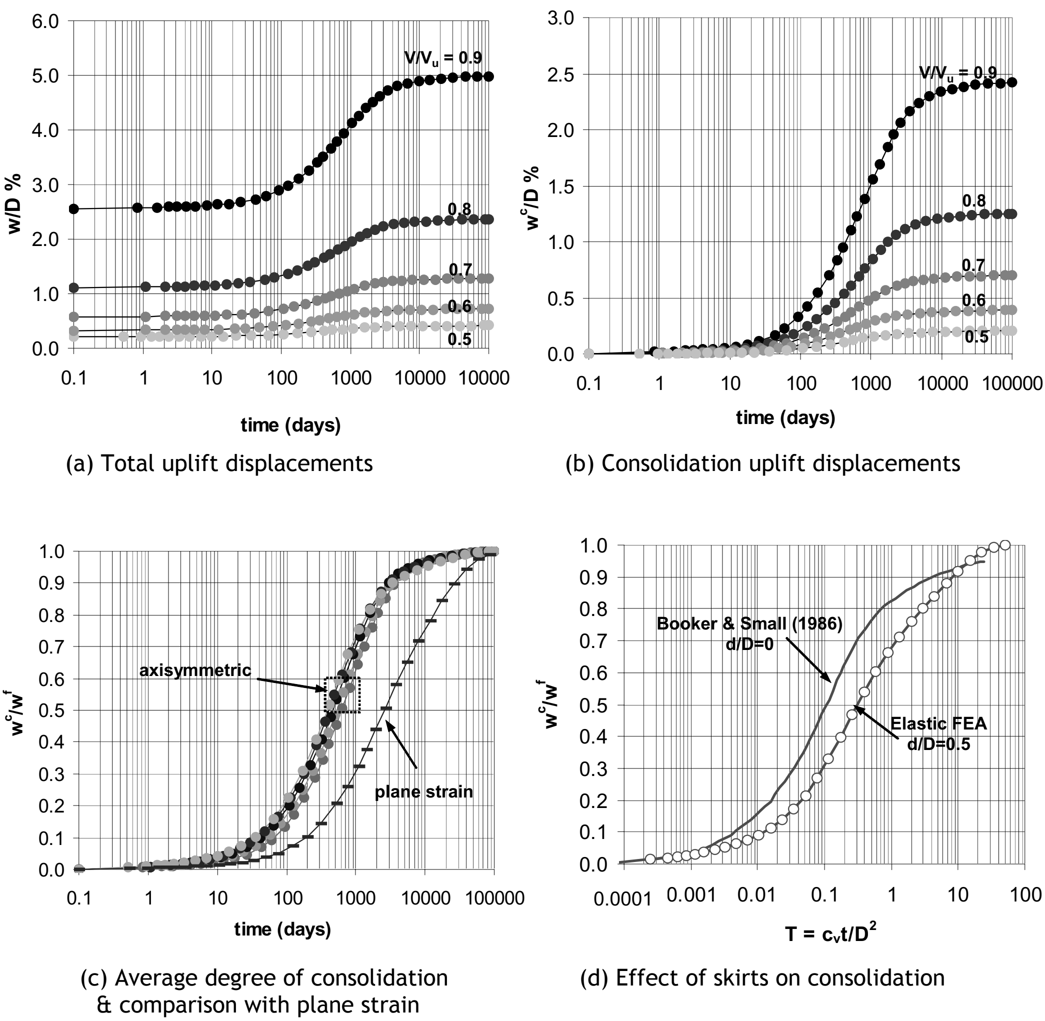
3.1.2 Undrained bearing capacity

For the skirted foundation models described in Section 2 an undrained bearing capacity factor $N_c = V_u/As_u$ of 10.50 was predicted with the axisymmetric model and 7.26 with the plane strain model. The load-displacement response showed a plastic plateau was established by a normalised displacement w/D ~ 7% for the circular foundation and 4% for the strip foundation. As a result of the fully bonded foundation/soil interface and the Tresca constitutive model adopted in these analyses the undrained bearing capacity is the same in tension and compression, enabling the undrained uplift capacity to be compared with theoretical solutions derived for bearing capacity in vertical compression.

Martin and Randolph (2001) present lower and upper bound solutions for vertical bearing capacity of circular skirted foundations with a rough base and rough skirts (analogous to the bonded interface conditions adopted in these analyses) which indicate the bearing capacity factor for d/D = 0.5 lies between $9.07 < N_c < 11.38$, bounding the result predicted by the axisymmetric finite element analysis carried out for this study. There are no published analytical solutions for rough skirted foundations in plane strain conditions. Bransby and Randolph (1999) present an upper bound solution for rough sided conventionally embedded shallow foundations (i.e. a solid structural plug) suggesting $N_c = 7.7$ and Gourvenec (2007) present finite element results for the same boundary conditions which suggest $N_c = 7.3$, in good agreement with the result predicted by the plane strain finite element analysis carried out for this study.

3.2 Time histories

Figure 4a shows time histories of uplift displacement under sustained loads where the applied load V, as a proportion of the undrained capacity V_u, varied between $0.5 < V/V_u < 0.9$. Load cases for $V < 0.5V_u$ were not investigated as displacements were considered small, w/D < 0.5%. Reported displacements refer to those at the load reference point, located on the underside of the foundation base plate along the axis of symmetry (Section 2.3). An immediate uplift displacement is observed on application of the load, equal to approximately half the final displacement irrespective of the magnitude of applied load. The relative magnitude of immediate to total final displacement depends on the value of Poisson's ratio ν'. Davis and Poulos (1968) note the immediate settlement contributes a much higher proportion to the total final settlement when ν' is high. Figure 4b shows the time histories of the consolidation displacements w^c given by deducting the immediate displacements (w^i) from the final displacements (w^f).



(a) Total uplift displacements

(b) Consolidation uplift displacements

(c) Average degree of consolidation & comparison with plane strain

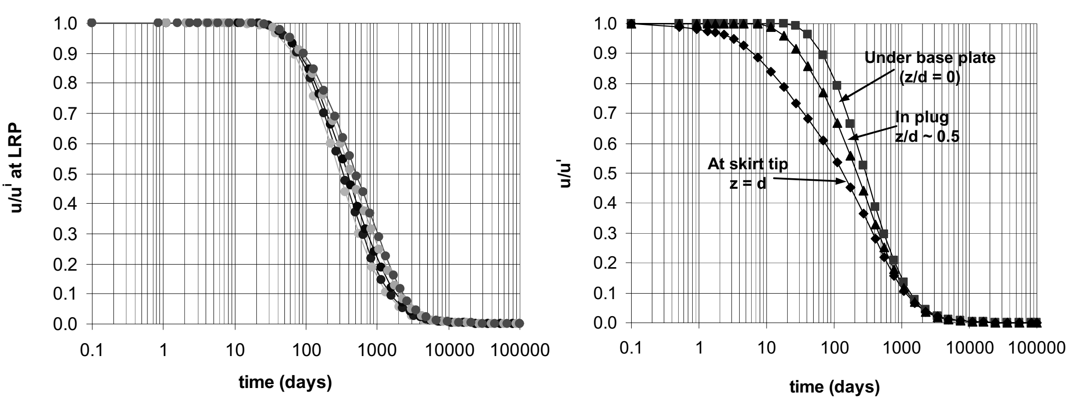
(d) Effect of skirts on consolidation

Figure 4: Time histories of uplift displacement under sustained loading

Figure 4c expresses the time histories in terms of the average degree of consolidation w^c/w^f. Since displacements occur as a result of dissipation of the excess pore water pressures, which is a function of soil permeability and drainage path length and independent of the magnitude of applied load, the normalised time histories for each loading case are closely banded. The rate of dissipation of excess pore water pressure is independent of the direction of the loading, i.e. the relationships shown in Figure 4 are the same in tension and compression (as a result of the fully bonded foundation/soil interface and the Tresca constitutive model adopted in these analyses). The significance of dissipation of negative excess pore water pressures in response to an uplift load is that long term conditions are critical, in contrast to dissipation of positive excess pore water pressures following application of a compressive load that leads to increased bearing capacity. Figure 4c also shows the time history from a plane strain analysis. It is evident that axisymmetry allows faster consolidation than plane strain conditions, as would be expected, since load transfer into the soil and drainage is restricted to occur in-plane in the latter case. The zone of influence of the applied load extending deeper and wider in the plane strain case compared to under axisymmetric conditions. The time for complete consolidation in plane strain conditions is approximately three times that in axisymmetry and during consolidation the difference can be more marked.

Figure 4d compares the normalised time history from an elastic axisymmetric finite element analysis (with the same geometry and boundary conditions as the analyses with plasticity, $d/D = 0.5$) with the theoretical solution for the average degree of consolidation beneath a rigid, smooth, circular surface foundation on an infinite elastic half space (Booker & Small 1986). For comparison with the theoretical solution, the finite element results are expressed in terms of a dimensionless time factor $T = c_v t/D^2$, where c_v is the coefficient of consolidation of the soil ($= kE/\gamma_w$) and D the foundation diameter. The theoretical curve relates to a Poisson's ratio $\nu' = 0.3$ in line with the finite element analyses. Comparing the normalised time histories for the surface and the skirted cases illustrates the beneficial effects of foundation skirts in increasing the length of the drainage path; consolidation times for the skirted foundation are in the region of three times those for the surface foundation throughout consolidation. The duration for complete consolidation is similar for the surface and skirted foundations as the latter stages of consolidation are governed by the far field. It is worth noting that if the skirt-soil interface is rough (as represented in the finite element analyses) a proportion of the applied load will be carried as friction along the skirts such that the magnitude of displacements would decrease with increased embedment ratio.

Figure 5a shows the time histories of the dissipation of the negative excess pore pressure in the soil directly beneath the foundation load reference point for a range of uplift loads between $0.5 < V/V_u < 0.9$. Pore water pressures are expressed as a dimensionless quantity normalised by the initial negative excess pore water pressure following application of the uplift load. As with the time history of normalised uplift displacement shown in Figure 4c the normalised time histories of excess pore water pressure are closely banded. Figure 5a shows that negligible passive suction is lost within the first ten days following loading, and 90% of the passive suction has dissipated within three years. Figure 5b shows the dissipation of normalised excess pore pressures at various locations in the soil mass around the foundation. It is evident that the skirts restrict flow; excess pore water pressures at the skirt tip dissipating more rapidly than within the plug.



(a) Dissipation of suction under base plate $0.5 < V/V_u < 0.9$

(b) Dissipation of suction around foundation $V/V_u = 0.5$

Figure 5: Time histories of suction dissipation

4 CONCLUDING REMARKS

The purpose of the study presented in this paper was to investigate the behaviour of skirted foundations under sustained tensile load. Long term conditions are critical for uplift problems and the duration over which passive suction can be maintained is of engineering importance. The study reported here was intended as a preliminary investigation into the uplift resistance of skirted foundations to identify general trends of behaviour. Further analyses are required to consider, for example, the effect of skirt length and roughness on the rate of pore water pressure dissipation, the effect of swelling and the increase in void ratio on the magnitude of permeability and hence the rate of dissipation of suctions, or the effect of a non-uniform distribution of suction beneath the foundation due to non-vertical loading. When considering uplift problems in low permeability material, the seal between the external skirt walls and the soil is critical. Should a crack form along one side of the skirt, from the seabed down to the skirt tip, communication of water through the crack would significantly influence the time scale over which suctions could be maintained. It is therefore necessary to model an interface between the foundation and the soil able to allow for separation. Representation of cracking requires a full three-dimensional model as a crack is unlikely to form around the entire perimeter of the foundation. Further numerical studies addressing some of the issues outlined above, in conjunction with a programme of centrifuge model testing investigating the uplift resistance of skirted foundations, is underway at the Centre for Offshore Foundation Systems.

5 ACKNOWLEDGEMENT

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