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TESTING OF THE INTERACTION OF COASTAL WINDFARM FOUNDATIONS WITH THE SEABED: SCOUR AND LIQUEFACTION

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Key Words
Seabed scour, Scour mitigation, Waves and currents, Seabed liquefaction, Wind farm, Caisson foundation, Physical modelling, Finite Element soils model, CFD modelling

Abstract
This paper presents the results of a study into the scour and liquefaction response of the seabed around a large diameter (19m) suction caisson foundation being considered for use in coastal windfarm developments around the UK. The scour and scour mitigation investigation was carried out in a wave-current basin and the liquefaction investigation in a wave flume. Supplementary Computational Fluid Dynamics modelling was completed to provide an improved understanding of the flow-structure interaction. The liquefaction work was extended through the validation and application of a computational Finite Element soils code. The outcomes of the work are summarised and implications for engineering design are presented.

1 Background

As a direct consequence of the Kyoto Protocol there is a new impetus in developing the capacity of offshore windfarms to provide a significant percentage of the target renewable energy quota. Within the UK a range of seabed locations have been licensed for development. One of the key factors in the siting and construction of offshore windfarms is the influence of the seabed geology and sedimentary environment on the foundation design and stability over the lifetime of the structure.

Recent experience in Europe suggests that costs for offshore foundations take up as much as 25 - 35% of construction budgets and the developers need good information about the ground conditions to understand the potential risks. The risks are likely to be increased where the turbines are sited on potentially mobile sand banks or in areas with shifting subsea morphology. This passive change in bed levels and morphology occurs regardless of the presence of the windfarm.

Developments to date are based on mono-tower piled foundations, but there are uncertainties about driving large diameter piles, and there is a limited capacity world wide for vessels and barges capable of doing the work. Future developments are likely to move into deeper water and will make use of even larger foundations. There is interest in alternative designs including suction buckets (Figure 1), multiple suction or piled anchors, gravity and floating structures.

The foundation solution needs to be optimised for the specific environment and decommissioned at the end of their design life.

HR Wallingford has contributed to environmental and seabed assessments related to proposed windfarm developments around the UK, completed scour modelling for a site in the Republic of Ireland, the Dutch sector of the North Sea (den Boon et al, 2004), and a joint Department of Trade and Industry and industry funded research project on the potential for use of suction caisson foundations (Danson, 2003). It is the latter study which forms the basis for the present paper. A range of foundation options was devised by the project partners (see Acknowledgements) and models of these were tested in the laboratory for seabed scour and liquefaction response (Figure 2).

The structure is a mono-tower and single pile with outside diameter Dm 4.6m, joined to a circular skirted foundation with an outside diameter Dc of 19m. The initial height of the top of the caisson above seabed level is 2m and the skirt tip depth is 9.5m. Penetration of the skirt to this depth around a “plug” of soil and maintenance of the surrounding seabed level is one of the requirements for foundation stability. Three different tower-caisson connection geometries were proposed including a girder top with radial fins, a concrete slab top, and a conical top; a quadruped solution was also developed with Dc 6m and skirt depth 4m. For comparison purposes the situation with a 4.6m pile directly placed in sand bed was also considered in terms of scour response.

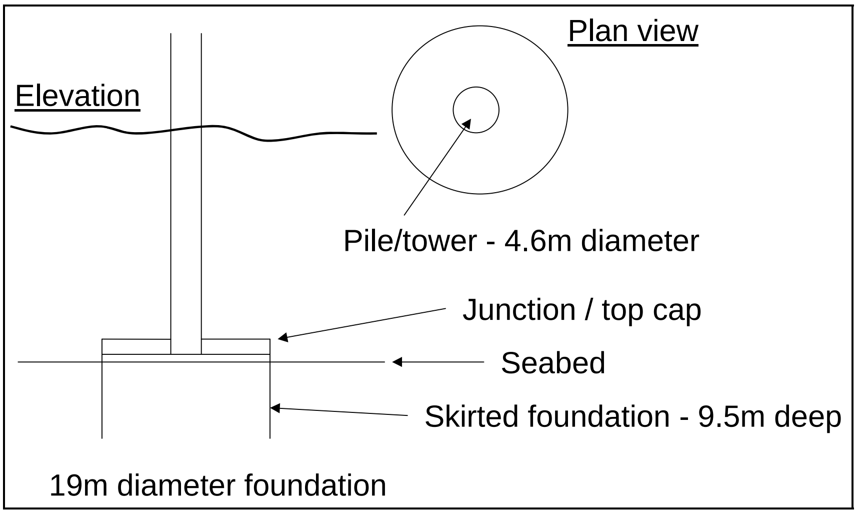

Figure 1 Schematic of suction caisson foundation solution for wind farms

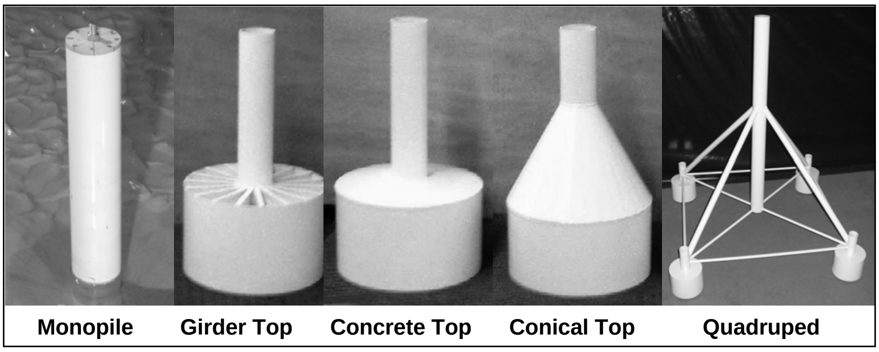

Figure 2 Laboratory scale models of the foundation options tested

2 Methodology

Parallel laboratory scale and computational modelling was completed to provide the necessary understanding of the key aspects of scour development at the different foundations and of the soil response to wave loading.

2.1 Scour

To better understand the way in which the seabed responds to the presence of the caisson foundation in shallow water, with the action of waves and currents, a series of 3D laboratory tests was completed at a geometric scale of 1:40. This scale was selected as appropriate for simulation of bed scour, and the corresponding model velocity and time-scales were 1:6.3. The tests used waves and a current crossing at 90 degrees in a large wave basin at HR Wallingford (Figure 3). The time development of scour depth and the equilibrium scour configuration were determined for different foundations with the conditions tested. The results of the scour testing have been presented by Whitehouse (2004). To provide an improved understanding of the way in which the flow interacts locally with the structure, Computational Fluid Dynamics modelling was undertaken using the CFX5.5 code supplied by AEA Technology. This is a fully implicit and coupled finite element solver based on an unstructured grid.

Tests were undertaken using conditions approximately representative of 1:1 year return period wave and current forcing to provide comparative data on scour at the different foundation types. The implications of scour taking place around the foundations are a loss of skirt depth. Thus it was important to understand how scour might develop around the foundation and measures for scour mitigation were also tested. The modelling was completed in a large test basin at HR Wallingford with waves generated at 90º to the current (Figure 3). A uni-directional current system was implemented to produce a steady flow field. The wave paddle generated irregular period long-crested waves to the required wave spectrum (JONSWAP) with conditions calibrated from wave-probe measurements.

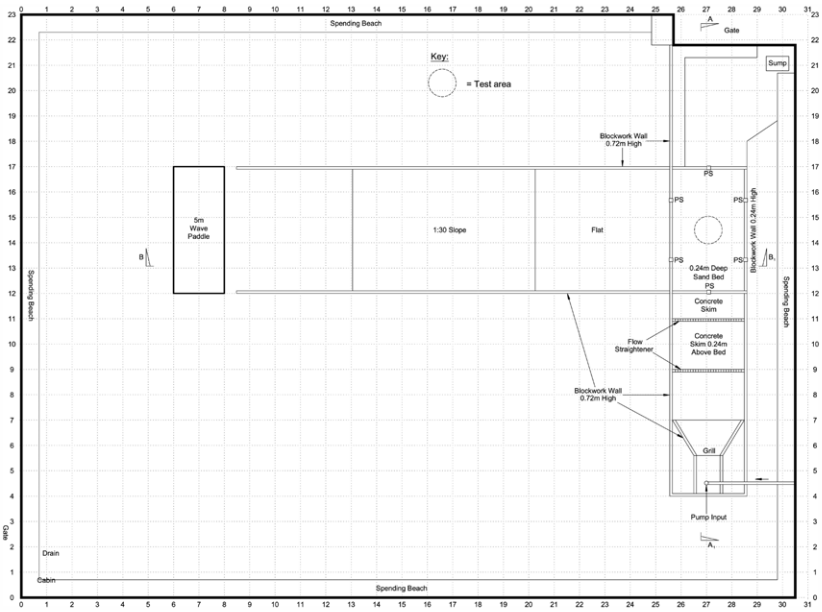

Figure 3 Layout of 31m x 23m wave-current basin for scour testing – the flow flume is on the right of the picture and the wave flume passes from left to right. The test area with caisson (circle) is at the junction of these two flumes

The foundation models were fixed in position to prevent settlement or displacement once scour had occurred. The sediment used in the model was fine quartz density sand of a median diameter of d_{50} = 0.111mm and the model sand bed was 0.24m deep, 5.0m long and 3.0m wide. The wave-alone and current-alone conditions were capable of mobilising the bed sediment, i.e. "live bed" conditions, which was confirmed by observations of the bed during testing, and the sediment was also transported in suspension in the water column. These two factors were important to reproduce similarity of the sediment transport processes.

2.2 Wave-soil interaction

Pore pressures can affect the foundation in a number of ways: 1. generation of net uplift pressures on the foundation, 2. changes to the skin friction on the foundation wall, and 3. potential for seabed liquefaction. The pore pressures have been investigated using a two-fold approach: with wave-flume modelling and advanced computational Finite Element soils modeling.

The role of wave induced pore pressures in the seabed was assessed using conditions representative of extreme wave loading; 1:100 year return period. The physical modelling was carried out in a wave flume at HR Wallingford at a scale of 1:63. Rectangular (2D) and circular (3D) foundation models were tested. The primary purpose of these tests was to provide validation data for the computational model and to determine the magnitude of the three-dimensional effects that are not reproduced in the computational model used. The model, DIANA-SWANDYNE II, which is the acronym of **D**ynamic **I**nteraction **A**nd **N**onlinear **A**nalysis – **SWAN**sea **DYN**amic version II (Zienkiewicz et al., 1999) is an improved version of the DIANA-SWANDYNE I (Chan, 1988) developed originally for earthquake engineering. It is intended for static, consolidation and dynamic analysis for problems in geomechanics.

The computational domain was set up to represent the caisson and the soil. First, the computational model was calibrated against the laboratory experiments and then used to model the full-scale conditions. The results were not only used to determine the likelihood of liquefaction but also the variation in the skin friction and the net pressure forces acting on the bucket due to the oscillating wave loads on the foundations.

For the purposes of this study there was no need to model the gradual build-up of pore pressure due to cyclic loading as the site investigation data had suggested that the sand is in a medium dense to dense state. The oscillatory response of the pore pressure, both inside and outside the caisson was adequately modelled using a simple elastic model with a Mohr-Coulomb type envelope applied to the effective stress state. In this model the Bulk and/or Shear Modulus can vary with mean confining effective stress.

The bed material used in the experiments was a non-cohesive limestone silt with d_{50} = 0.033mm and was the same as that used by Teh et al (2003). The structural material properties and full geotechnical properties of the silt were determined for implementation in the computational model. Representative values were used for the full scale modelling.

3 Results

The results from the study are presented in two sections dealing with the scour and liquefaction aspects of the work.

3.1 Scour and scour mitigation

A conceptual model for flow interaction with the caisson was presented by Whitehouse (2004) and the results from the CFX investigation show clearly how the flow impinges on the caisson structure (e.g. Figure 4). Other quantities such as the local flow and bed shear stress fields, and turbulence levels, were analysed. The scour test results showed that the scour depth due to wave current action could develop to the full depth of the skirt and hence it was necessary to investigate scour mitigation measures. The next part of the paper presents the results from the scour mitigation investigations in waves plus current where a 5m wide ring of armourstone was placed on the initially flat seabed around the foundation.

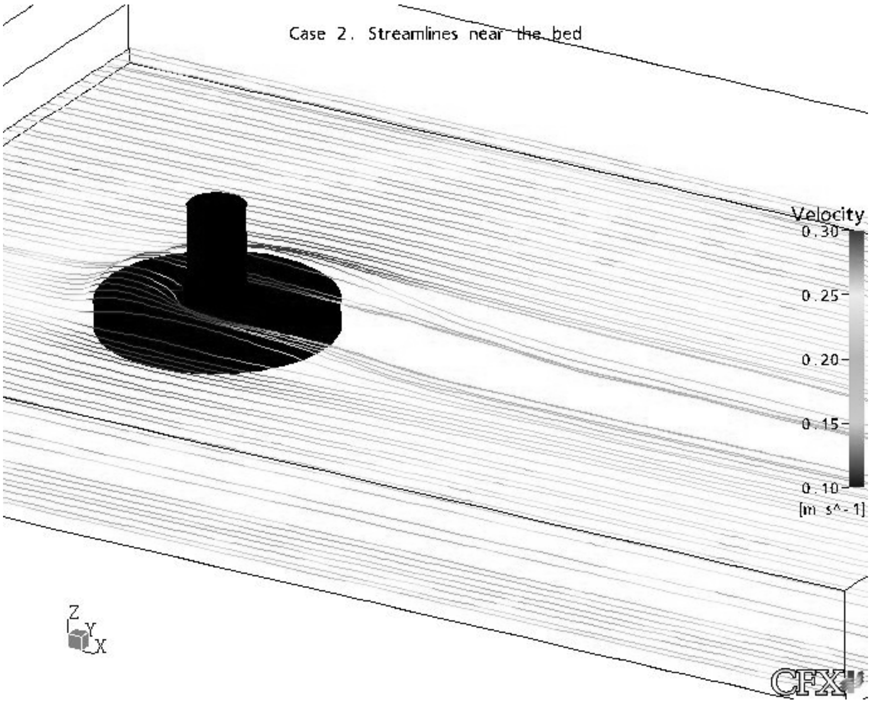

Figure 4 Example plot from CFX model showing streamlines near the bed being deflected over the top lid of the caisson; flow from left to right

Concrete Top, waves and currents, scour protection installed

Within the first few minutes of the testing, suspended sand was seen to settle and fill between the scour protection at points D – F on the up-current side of the foundation. A number of scour protection rocks were seen to move or rotate as any unbalanced armourstones found a more settled position within the armour matrix. From observations it was estimated that this type of movement occurred with less than 2% of the total armour. The scour protection acted effectively to prevent scouring around the skirt (Figures 6, 7).

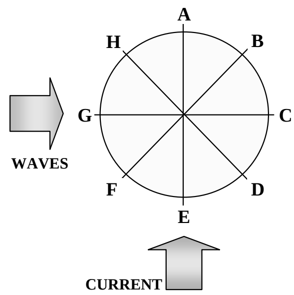

Figure 5 Measurement locations

At the end of testing some of the scour protection material adjacent to H – B on the down-current side of the structure were seen to fall into a scour hole which formed at this location. The outer edge of the horizontal portion of the armour (the crest) retreated from 5m to 3m as a result of this scouring. Rocks moved down the slope of the scour hole and settled along the gradient to form an apron approximately one rock layer thick on top of the sand bed.

Figure 6 With scour protection

Figure 7 Without scour protection

A tracer dye was injected upstream in order to observe the local flow field around the foundation. Contraction of the oncoming current flow around the base of the monopod was observed to produce a tight band of faster flowing fluid around the edge of the skirt, travelling over the top of the scour protection until it reached points B and H. At this point the current flow separated and formed eddies which were advected downstream by the current. At some points the flow overspilled the top of the foundation and passed downstream with the ambient current (similar to the modelled pattern in Figure 4).

The initial bed levels and scour protection rock level were checked with bed profiles taken using a calibrated automatic bed profiler. Pre-and post-test profiles were used to identify any settlement in the scour protection rock and scour in the surrounding seabed.

The results showed that the scour protection rock maintained the initial level but that there was scouring of the bed around the protection by up to 1 m locally. This led to the creation of a falling apron around the foundation with a reduction in the width of the initial thickness layer and spreading out of a single layer of stone on the bed. Typically the edge of the rock protection in the model attained a slope of 1V: 4H.

Concrete Top, waves and currents simulating reversed tidal flow, scour protection installed

Figure 8 Manual excavation, Pre test

Figure 9 Post test configuration

To investigate the role of tidal flow reversal a test was conceived where a mirror image of the scour hole and falling apron that had formed on the downstream part of the model during the first test was created at the upstream side of the model. Therefore the flow would pass over the simulated scour hole and rock apron before reaching the structure (Figure 8).

The results showed that the local reduction in bed level continued during this test, up to a total of 2m drop locally around the rock protection was recorded. Some further thinning of the protection was observed whilst still maintaining a functioning scour protection layer around the foundation (Figure 9). Manual measurements of the scour protection crest position and toe of the protection showed nowhere less than 3m width of protection adjacent to the caisson, with some rock slopes as steep as 1V: 1H locally. Upstream of the structure the pre-excavated scour pit was measured as partially infilling and growing broader in the across flow direction (Figure 9).

Quadruped, waves and currents, scour protection installed

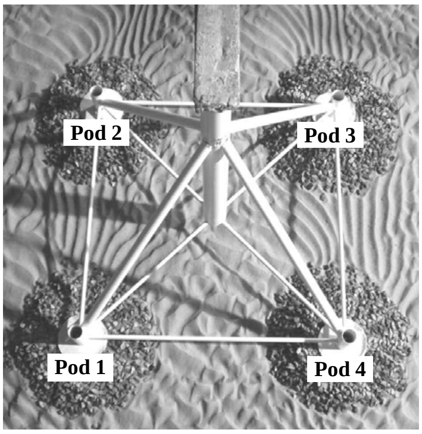

Figure 10 With scour protection

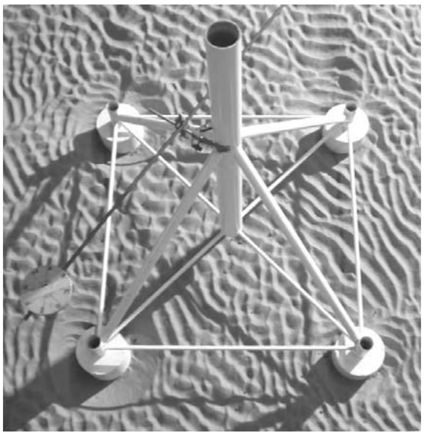

Figure 11 Without scour protection

The aim of this test was to examine the performance of the same scour protection material and extent that had been used in the previous tests on preventing scour at the quadruped foundation. A 5m ring of armour rock in two layers was placed around each of the four foundations.

The scour protection material was seen to fill with sand at points D - F (see Figure 5) during the early part of the test as scour holes began to form around the toe of the scour protection on each footing. Figure 10 shows the final result which can be compared with Figure 11 for the unprotected bed. In these Figures the current was flowing from bottom to top and the waves from left to right, and hence pod 1 was the most exposed footing. The scour in the surrounding sand bed was enough to cause formation of a falling apron in the scour protection.

Distinct wave ripples were seen to form between the footings as the scour holes began to form, this indicated that the area of the bed immediately down current of the rock armour was dominated by wave action rather than current turbulence. Dye injection tests confirmed this was the case.

The results showed a reduction in bed level with this test, up to 1m drop locally around the rock protection between and adjacent to each foundation. There was some thinning and spreading of the protection with some settlement but the rock still maintained a functioning scour protection layer around each foundation.

3.2 Wave-soil interaction

A valve was fitted to the top of both the rectangular and circular caissons; the quantity D was used to characterise the width or diameter of the caissons. Two different scenarios were investigated: one where the lid of the caisson was completely impermeable, and another where it was effectively free draining (in comparison to the drainage characteristics of the silt).

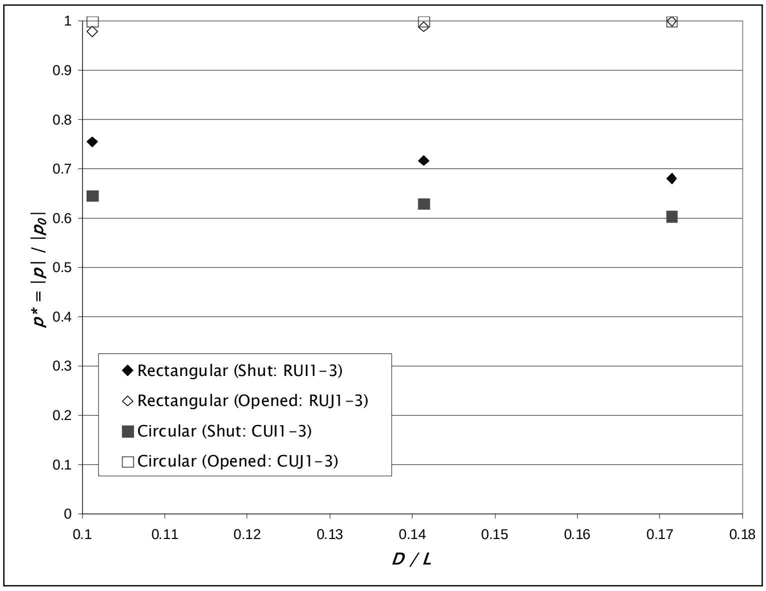

Figure 12 Results from the experimental study of soil pore pressures showing how the dimensionless pore pressure p* under the top plate varies with relative wavelength D/L: p* is the ratio of the amplitude of the pore pressure at the measurement location to the amplitude of the pore pressure at the seabed surface. Shut and opened refer to the valve in the top plate

Figure 12 shows comparisons between the 'shut' and 'open' valve runs. The results show that, as expected, the pore pressure tends to be higher when the valve is open, particularly underneath the lid. The reason for this is that the pressure inside the bucket is able to equalise with the external pressure through the open valve. The data also shows some differences between the rectangular (2D) and circular (3D) cases. The results were reasonably similar for the rectangular and circular caissons. This suggests that it may be possible to use a 2D model to predict the general behaviour of the pore pressure in the 3D case.

Figure 12 also shows how the pore pressure underneath the top plate varies with the parameter *D/L*, where *D* is the diameter/width of the caisson and *L* is the wavelength. In the 'shut' valve cases, as the wavelength increases (i.e. decreasing value of *D/L*), the pressure tends to increase. This is what would be expected as an infinitely long wavelength would give a pressure ratio p* = $|p|/|p_0|$ = 1, everywhere in the bed. On the other hand, if the wave length is decreased the pore pressure will also decrease and p* will tend towards zero.

The computational model was used to predict the variation in p* with depth in the bed in the far field, just outside the skirt and inside the caisson. Figure 13 shows how the pressure amplitude varies with depth at four different locations: (i) centre of the caisson, (ii) inside the skirt, (iii) outside the skirt and (iv) in the 'far field'. The liquefaction potential will be greatest in the regions where the pressure gradient is largest (i.e. outside the skirt). Over the distance 0 z/h < -0.1 there is a sharp gradient in p* which indicates the zone of the seabed likely to experience liquefaction. This corresponds to the top 1 to 2m of the seabed.

The skin friction on the outside wall of the skirt was predicted to vary by about 20% in the wave cycle. Perhaps the single most important result from the experimental data and modelling is the pressure difference across the lid of the closed caisson. For both the circular and rectangular caissons the pressure difference across the top lid of the caisson varies between 20 and 35% of the oscillatory pressure acting on the seabed surface, $|p_0|$. The net force with a closed caisson in the conditions studied is of order 10kN and may be significant for the overall structure stability and therefore should be included in the design study.

4 Discussion

The scour tests for the suction caissons showed the potential need for provision of scour protection measures as scour in the sandy seabed was likely to occur under relatively common conditions of tides and waves (1:1 year return period); as it would for a monopile. A rock dump solution was implemented and tested. The rock dump used 200kg mass rock placed in a concentric ring of 5m diameter and nominally 1m (2 rock layers) thick directly onto the sandbed. This was tested with both the concrete top 19m diameter caisson and the Quadruped foundation under wave and current action.

The scour protection was tested for a similar length of time as the scour development tests on the unprotected bed. The rock tested was apparently stable under the conditions tested, including a simulated reversal of the tidal flow direction, and appeared to provide adequate protection against scour development around the caisson skirt. The erosion of the seabed around the outer edge of the protection led to a local drop in bed level which reinforced the tendency for the edge of the rock dump to spread to form a ‘falling apron’. This produced a thinning of the rock dump and settlement of individual rocks into the bed. In principal the concept works but the following factors need to be borne in mind:

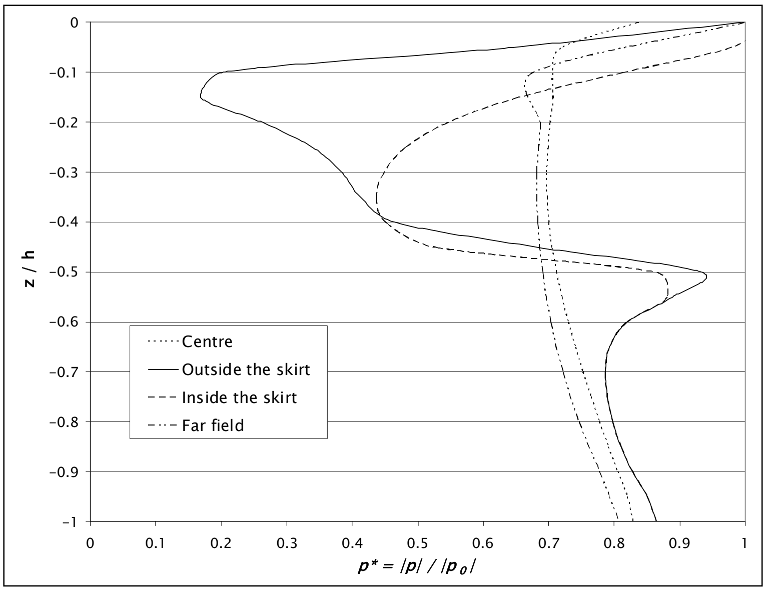

Figure 13 Results from the computational model run at full scale showing how p* varies with depth in the bed z/h. The caisson skirt tip was at z/h = –0.5

- The intact rock and apron are susceptible to disturbance by more extreme conditions or due to the influence of regional lowering of bed level;
- The stability of the rock in higher return period events needs to be tested;

- The influence of bed lowering on the falling apron needs to be taken into account – this influences the overall width of protection to be placed;
- The need for a filter layer between the rock protection and the seabed should be considered;
- An alternative approach is to dredge a shallow hole in the seabed in which to place the foundation, but this may not remove the need to scour protection material.

From the experimental results on soil pore pressures it was concluded that liquefaction can take place under extreme wave conditions. The influence on the rock protection and the benefits of a filter layer in preventing sand winnowing and settlement of the rock protection during storm events needs to be considered.

5 Conclusions

This paper has presented the results from an effective hybrid experimental-computational study of scour and liquefaction response on the seabed around a skirted foundation in a cohesionless seabed soil. Scour can occur under frequently occurring current and wave action that mobilises the bed sediments, and the scour depth can develop to the tip of the foundation skirt. Therefore mitigation measures are required to prevent loss of soil from around the foundation. A rock dump protection was tested and found to be effective in preventing scour at the foundation. Seabed liquefaction under extreme storm wave conditions in sandy soil may occur locally to the foundation. Finally, the influence of wave loading in producing net uplift forces on the foundation should be taken into account.

Acknowledgments

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