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INVESTIGATING NOVEL FOUNDATIONS FOR OFFSHORE WINDPOWER GENERATION

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ABSTRACT

In recent years there has been a worldwide increase in the pressure to develop sources of renewable energy. The UK government is committed to ensuring that ten percent of UK energy consumption will be supplied by renewables by the year 2010. Central to this commitment is the need to develop wind farms particularly in the offshore environment. Moving offshore will allow very large wind turbines capable of supplying 2 MW (first generation) to 5 MW (second generation) of power to be installed in large farms consisting of up to fifty or more turbines. In contrast to typical oil and gas structures the foundation may account for up to forty percent of the projected installed cost. The weight of each structure is very low, so the applied vertical load on the foundation will be small compared to the moment load derived from the wind and waves. Further, it will be necessary to have a single design that can be mass-produced over each site rather than have each foundation individually engineered. In combination these points lead to a very interesting engineering problem where the design of the foundation becomes crucial to the economics of the project.

One solution is to use conventional piling. However, at some sites it may prove more economical to use shallow foundations, and, in particular suction installed skirted foundations [1]. It will be necessary to develop an adequate design framework for these novel foundations under the relevant combinations of load so that the optimum structural configuration can be achieved. At Oxford University a program of research on skirted foundations has been underway for the last five years, and much progress has been made on the understanding of this type of foundation under combined

loading. This progress has been in both experimental and theoretical areas. This paper explores various structural options that might be used for the wind turbine application. These different options lead to different loading conditions on the foundations. Experiments investigating these different loading conditions are explored. A theoretical approach that describes the experimental results in a way that can be implemented in typical structural analyses programs is outlined. Finally details of a major research program into developing the necessary design guidelines for foundations for offshore wind turbines is described.

Keywords: Foundations, Suction Caissons, Renewable Energy, Wind Turbines, Offshore Structures.

INTRODUCTION

One of the more mature renewable energy concepts is that of wind energy, reaped through the use of large wind turbines. Many countries use wind turbines onshore, but in the UK where green land is sparse this option causes significant controversy. Reasons cited for opposing such developments usually relate to the destruction of the aesthetic beauty of the landscape, with resultant reduction in property values. Consequently there is pressure to put wind turbines offshore where they will be out of sight (and perhaps out of mind). Advantageously, by moving offshore, larger structures can be developed which thus lead to a much greater power output. It should be noted that offshore winds are not necessarily stronger, but are usually more consistent. There are large tracts of seabed that are not used for other purposes and may be suitable as wind farms.

For example, in a Greenpeace sponsored report produced by Border Wind [2], it is estimated that to produce 10% of the UK energy by 2010, 12 000 MW of wind turbines will be required. Based on a rated turbine size of 2MW this translates to 6000 turbines. Of course over time the size of the turbines will increase, with suggestions that 5MW wind turbines might be produced. It will therefore be likely that somewhere in the order of 4000 turbines could produce the required 10% of UK electricity. The report suggests that the total seabed required for the 10% target would be approximately 1200 km² based on the use of the 2MW wind turbines (this would reduce for larger turbines). The total amount of seabed around the coast under UK control amounts to approximately 800 000 km². The 1200 km² is a tiny percentage of this (~0.15%) [2]. The introduction of wind turbines does not necessarily render areas sterile, as it will still be possible for fishing and other activities to occur there. Anecdotal evidence in fact suggests that offshore structures, likened to artificial reefs, lead to an increase in the amount of marine life in an area.

Recently the Crown Estates opened the way for the development of offshore wind energy when they released 13 sites for development of 540 turbines around the UK, as shown in Figure 1. Towards the latter part of 2000 the first UK offshore wind farm, consisting of two 2 MW wind turbines as shown in Figure 2, was commissioned in Blyth [3]. The foundation was relatively simple as the supporting pile could be socketted into the very shallow bedrock. However, the coastline around the UK consists of very diverse seabed materials, such as loose mobile sand banks, glacial till and soft clay. Each site, with vastly different soil types, will require different engineering strategies to be developed and followed. For significant progress to be made on the UK energy targets the development of these engineering strategies is a priority. This may allow the wind resource to compete on an economic basis with the more traditional power sources. This paper will discuss the various structural and foundation systems that may be employed. They may not be suitable for every design site, but the establishment of competing strategies will help to drive down the cost of more traditional and established approaches.

NOMENCLATURE

| | |
|------------|--|
| D, R | Diameter (L), Radius (L) |
| H | Horizontal Load (F) |
| h | Installed length of caisson skirt (L) |
| K | Earth pressure coefficient |
| L | Length of caisson skirt (L) |
| M | Moment (FL) |
| m_o | Moment dimension of yield surface |
| p_a | Atmospheric pressure (F/L^2) |
| s_u | Undrained shear strength (F/L^2) |
| u | Footing horizontal displacement (L) |
| V, V' | Vertical load, Effective vertical load (F) |
| V_{mean} | Mean vertical load (F) |

| | |
|------|---|
| w | Footing vertical displacement (L) |
| a | Adhesion factor |
| d | Interface friction angle (degrees) |
| g' | Effective unit weight of the soil (F/L^3) |
| q | Footing rotation (degrees) |

STRUCTURAL DESIGN STRATEGY – MULTI-LEG VERSUS MONOPOD

Some structural configurations available for an offshore wind turbine are shown in Figure 3. The foundation might consist of a driven pipe pile. This technology is well established, so this paper will not deal with it. Suffice it to mention that it is the main option competing with the systems that are described here. Instead this paper will concentrate on shallow foundation options that might be used in the various structural layouts. In this paper it is necessary to adopt some approximate values for loads, so that reasonable comparisons can be drawn between the various options. These are not definitive values, and depend very much on the nature of the structure and the environmental conditions at the site. However, for the purpose of this paper, the extreme loads will consist of (a) a vertical load (V) of 8MN, (b) a horizontal load (H) of 4MN acting at a height of 20m, and consequently (c) an overturning moment (M) of 80MNm, all acting in plane. Of course it may be possible that the wind, wave and current directions are not coincident, which might result in the horizontal and moment loads being non-coincident. The extreme loads are usually associated with cyclic loading and dynamic phenomena. Further the critical conditions may not be the extreme loading events, but accumulation of displacements under lower levels of loading (fatigue). These cases will not be considered here, as they are the subject of current research.

The simplest foundation that might be employed would be a monotower on a gravity base foundation. The combination of loading is unusual in offshore design as the moment loading is very high compared to the vertical loads, so that the eccentricity of the loading would, in this example, be $e = M/V = 80\text{MNm} / 8\text{MN} = 10\text{m}$. Assuming an elastic distribution of stresses under the footing, a conservative assumption, this implies that the foundation would have to be approximately 60m wide so that no tension exists anywhere under the foundation – not really an acceptable design. This diameter could be reduced if it can be assumed that a tension could be sustained, as a suction under the foundation for a short period of time, while the extreme conditions apply to the structure. However as the drainage paths are very short at the edges of the foundation, it is likely that a suction could not be sustained for prolonged periods of time. Alternatively it might be better to increase the effective weight of the structure by ballasting using concrete or iron ore. Figure 4 shows how the diameter of the base (foundation) would decrease depending on the height of a concrete layer added to the structure and assuming

the no-tension requirement. Using this method it might be possible to increase the effective vertical weight by a factor of two to three. Increasing the height of the foundation above the seabed, however, has the undesirable effect of attracting more wave and current load, so there will be diminishing returns beyond a certain height.

Another alternative would be to extend the foundation into the ground using skirts so that the load is carried by more competent soils. The foundation can be installed into the ground using suction as shown in Figure 5. By evacuating the water from the internal cavity a net downward pressure is applied to the foundation forcing it into the seabed. Adding skirts has the added advantage that suctions might be allowed to develop, providing that the permeability of the soil is reasonably low. In this case the drainage paths are dependent on the depth of the skirts, and the drainage time is inversely proportional to the permeability of the soil. A check has to be made on the magnitude of the hydraulic gradients. At present there are no design guidelines for a skirted foundation subjected to large moments but small vertical loads. This means that the foundation is designed on a site specific basis, rather than by applying a generic design strategy. A combination of gravity base and skirted foundation will offer many advantages. These types of structures have been used to great effect in the Norwegian sector of the North Sea during the last thirty years. By adding the skirt it may be possible that the soil trapped within the skirts acts as a rigid block. If this can be assumed then the bearing capacity problem is transferred to skirt tip level and so the trapped soil might be counted as part of the effective weight of the structure. In both cases a check on the horizontal capacity needs to be carried out.

The alternative design would be a tripod or four-legged structure supported on caisson foundations at each corner. We will assume that the limiting design criteria will be the tension loading of the upwind leg/legs with the structure being rotated about the downwind legs, as this is the worst loading case. So that it is possible to carry out some simple calculations we will ignore moment and horizontal loads on the footings. In any case it would be difficult to estimate these without performing a full analysis of the structure and the foundation-soil interaction. In this preliminary design we shall ignore any interaction between footings, although in a final design this possibility would need to be considered. The effect of this interaction is the subject of current research. Figure 6 illustrates the problem for the three legged and the four legged cases. For this design it is possible to perform some simple calculations to form a basis for comparison between the two designs. We will consider both the undrained and drained tensile failure and take the lower as the governing criterion. For both cases we will assume that the vertical tensile capacity

of the caisson will be limited by the friction carried on the skirt only. The undrained capacity can be estimated by:

$$-V' = (\mathbf{a}_o D_o + \mathbf{a}_i D_i) p h s_{u1} \quad 1$$

where \mathbf{a} is an adhesion factor (typically taken as 0.5), D is a diameter, h is the current installed depth of skirt, s_{u1} is the shear strength averaged over the depth of the skirt, and, the subscripts o and i refer to the outer and inner surfaces of the caisson. The inner frictional capacity will be limited by the weight of the soil plug within the caisson. We could also add the reverse bearing capacity failure around the annulus of the skirt but this might be a non-conservative addition. The drained calculation involves a similar calculation that is simply expressed as:

$$-V' = ((K \tan \mathbf{d})_o D_o + (K \tan \mathbf{d})_i D_i) p \frac{\mathbf{g}' h^2}{2} \quad 2$$

where $K \tan \mathbf{d}$ is the factor between the vertical stress and the shear stress on the skirt (typically 0.5 for most cases), \mathbf{g}' is the effective unit weight of the soil (typically about 8kN/m³), and other definitions are as above. Again the internal friction is limited to the weight of the soil plug.

The distances between the caissons will be taken as 17.5m for the four caisson structure and 20.2m for the tripod – this gives the same lever arm for the upwind caisson/s, and a similar plan area. Scour is likely to be a problem so a scour pit of 0.5m will be assumed around the outside of each caisson. We will assume that for the undrained case the shear strength of the soil will be 35kPa. The wall thickness of each caisson will be 20mm and the thickness of the caisson cap will be 40mm. Figure 7 shows the relationship between skirt depth and diameter for caissons that are adequate foundations for the different structural configurations. Obviously the four-legged structure will be the most stable against any loading direction as the worst loading case involves two foundations in tension. For the tripod structure the worst loading case involves one caisson in tension. For the four-caisson structure the self-weight load also acts at a greater distance from the downwind caisson than for the tripod case. Thus for the tripod the required tensile load on the upwind caisson is much greater than for the equivalent four caisson structure.

There are likely to be restrictions on the length (L) to diameter (D) ratios that are permissible in the shallow water depths where wind farms will be located. In sand the maximum L/D ratio is typically 1 before piping failure occurs. In stiff clays the maximum L/D ratio will be 2.5, increasing to 5 for soft clays with increasing strength with depth. In this case the limiting ratio is governed by the condition when the suction pressure within the caisson causes a reverse bearing capacity failure of the soil plug at the skirt tip. However, if the water depths are sufficiently shallow, the limiting factor for the clay case will be the cavitation limit of the water. This will be equal

to the water depth plus atmospheric pressure, so in 5m of water there will be a maximum suction of 150kPa possible. For shallow waters and typical strength profiles it might be that the maximum L/D ratio will be less than 1.5. If we look at our design case assuming that we are interested in L/D ratios of 1 only we can see that for the four caisson case the diameter of each caisson is 2.8m (~3m). The effective weight of each caisson is approximately 51kN. For the tripod case the diameter for each caisson is 5.3m with each caisson having an effective weight of 179kN. The decrease in caisson size and effective weight for the four caisson design is offset by the extra structural steelwork for the bulkier structure. This larger structure may also attract more wave and current loading than the tripod structure but the effect of that has not been included in this analysis.

It is easier to carry out a preliminary geotechnical design for the multi-legged structures, as the moment loads applied to the structure are carried as vertical loading on the foundations. This requires a relatively straightforward calculation to be carried out, assuming that the structure rotates about the downwind caissons with the critical loading condition being the tension of the upwind caisson. From a mass-production perspective the skirted gravity base monopod, however, holds great advantage. This solution is likely to be favoured once a rigorous design framework can be established and verified. It is highly likely that ballast will be required so that the diameter of the foundation is reduced to a suitable size. A possible solution could be a composite structure with a steel skirt and concrete base.

CURRENT AND FUTURE WORK

Work has been progressing at Oxford University over the past few years so that some of the issues discussed above can be addressed. This work has been initially focussed on the oil and gas problem, but now there is more emphasis on the wind engineering problem. A three degree-of-freedom loading rig has been developed at Oxford for studying these foundation problems. Further details may be found in Martin [4] or Byrne [5]. The loading rig has been developed so that independently controlled paths can be applied for vertical, moment or horizontal loading. The corresponding footing displacements are measured. Alternatively displacement paths, or indeed a mixture of both, can be applied to the footing. The results are typically interpreted through theoretical frameworks based on work hardening plasticity. These types of models, valid for monotonic loading only, have been developed for overconsolidated clay [6, 7], dense sand [8], and loose carbonate sand [9]. More recently research has concentrated on developing models for cyclic loading such as the typical loading derived from the waves. Sufficient control of the loading rig has been achieved so that transient loading paths, which might correspond to extreme events, can be applied to the foundation.

Moment Loading

For a monopod structure, the vertical load will remain relatively constant whilst the moment load would vary with the incidence of waves. This loading regime has been replicated in the laboratory and Figure 8 shows a typical experimental result for a footing test on dense saturated sand. The footing was 150 mm in diameter with 50 mm skirts. The mean vertical load during the test was 200N. During the test there is a stiff response at low strain levels followed by greater amounts of plasticity at larger strain levels. Tests were carried out at different rates and at different values of mean vertical load to investigate the effect of these parameters. A non-dimensional relationship was found that related the loads to the displacements, taking into account the different mean loads. This relationship, written in the form;

$$2Rq\sqrt{\frac{p_a}{V_{mean}}} = f\left(\frac{M}{2Rm_o V_{mean}}, \frac{V_{mean}}{g' D^3}\right) \quad 3$$

was found to be suitable provided that the mean load was much less than the peak vertical bearing capacity of the soil. In the equation q is the rotation of the footing, R is the footing radius, V_{mean} is the mean vertical load applied to the foundation, p_a is a reference pressure (atmospheric pressure) and m_o is a parameter defining the capacity of the caisson under pure moment loading. There were no observable effects due to the loading rate in the tests undertaken. However, rate effects need wider investigation. A fuller description of these and related results can be found in Byrne [5] and Byrne and Houlisby [10].

Vertical Loading

For the tripod structure the vertical loading is the most important, particularly when the foundations are loaded into tension. Figure 9 shows some results from a typical vertical cyclic test on saturated dense silica sand. The foundation has a stiff response in compression but this softens substantially on reaching tension. As these experiments were carried out on the laboratory floor, the magnitude of the skirt friction is very small, and it is difficult to determine the exact stage at which the softened tensile response occurs. The postulate is that the skirt friction is mobilised relatively quickly, but larger strains are required to mobilise the reverse bearing capacity response. The ultimate capacity was governed by this reverse bearing capacity and was found to be limited in the laboratory test by the cavitation of the pore fluid. The design of the foundations in tension is likely to be limited by serviceability criteria. In a similar fashion to the moment loading it was possible to find a relationship between loads and displacements that allowed for different mean vertical loads. Such a relationship can be written in the form:

$$w\sqrt{\frac{p_a}{V_{mean}}} = f\left(\frac{V - V_{mean}}{V_{mean}}, \frac{V_{mean}}{g' D^3}\right) \quad 4$$

where w is the vertical displacement, V is the vertical load and all other parameters are as defined previously. More details can be found in Byrne and Houlsby [11]. The change in stiffness of the foundation as it progresses from compression to tension could affect the dynamics of the structure. As yet the effect of this has not been explored.

Theoretical Modelling

The experiments discussed have been designed so that they can be interpreted within appropriate work-hardening plasticity models. The hypothesis is that a yield surface is set-up in vertical:moment:horizontal load space ($V:M/2R:H$). The size of the surface is governed by the magnitude of the vertical plastic displacement. Any footing movements within the surface result in an elastic response whilst an elasto-plastic response occurs when the load state reaches the yield surface. Models such as these have been developed for both clays [6, 7] and sands [8] as discussed above. These models are useful when considering monotonic behaviour but are not appropriate for cyclic loading. Recently a new approach called ‘continuous hyperplasticity’ shows the requisite capabilities for successfully capturing the cyclic loading response [12, 13]. The theory has a basis rooted in thermodynamics, and relies only on the specification of two potential functions, namely an energy function and a dissipation function. Further information is given by Puzrin and Houlsby [13]. A typical result from the application of the theory can be seen in Figure 10. This shows an experiment and the theoretical reproduction of the experiment.

The DTI and EPSRC Project

Research on the application of caissons as foundations for offshore wind turbines is being supported by the Department of Trade and Industry (DTI), the Engineering and Physical Sciences Research Council (EPSRC) and a number of industry partners. The parts of the project being carried out at Oxford are described here briefly. Other parts of the project will be carried out by the industrial partners. This project started in early 2002 and will continue for three years.

The first theme will be laboratory testing. The tests will be similar to those described above, but focussed on loading appropriate to wind turbines. The key design issue, as stated above, for the tripod design is the tension response of the upwind leg, whilst for a monopod it is the overturning moment. In each case the issue is not a matter of ultimate load capacity (which is large) but of accumulated deformations under typical cyclic loading. For the tripod it appears that, as the foundation goes into tension, there is a transition to a much softer response. The data are, however, rather few at present. Various factors will be explored including (a) different sizes of foundation to investigate scaling, (b) different soil types (sand at different densities, clay at different strengths), (c) different

possible loading regimes, and, (d) tests directly shadowing the proposed field tests (see later). A range of tests will be carried out with loads up to 500kN applied to a footing up to 400mm in diameter. Additionally it is proposed to do some tests in a sealed chamber, as the issue of the ambient water pressure could be important.

The second theme of research is field testing. Although the model tests will cover a wide variety of conditions, it will be necessary to establish their relevance to the field through larger scale tests. It is therefore intended to carry out tests at two or three sites (including one sand and one stiff clay), in which intermediate scale caissons (probably 2m diameter) would be installed. It is planned that three foundations would be installed in line, with a cross beam linking them. Loading devices on the cross-beam would allow the central foundation to be subjected to (a) cyclic tensile/compressive loading and (b) cyclic overturning moment. Appropriate instrumentation will be installed, including pore pressure sensors, strain sensors and applied load sensors. The tests will be used to establish appropriate scaling factors from model tests for both displacements and loads. It is envisaged that monitoring will also be performed during caisson installation and removal. The tests will probably be carried out at onshore sites, in a shallow flooded excavation.

The final theme of research is theoretical modelling. There has been extensive development of the “force resultant” models for the behaviour of shallow foundations. “Model B” for clay [4, 6, 7] and “Model C” for sand [8, 14] have been developed and used in finite element analyses of jack-up units under wave loading. In these models the behaviour of the foundation is reproduced in terms of the relationship between the resultant forces (vertical, horizontal, moment) on the foundation and the corresponding displacements. New models will be developed so that caisson foundations can be represented appropriately within structural analyses packages. The work will involve merging “Models B” and “C” and extending them to the 6 degrees of freedom that is required for implementation into a 3 dimensional analysis package. This will also involve developing the models to allow the prediction of displacements under cyclic loading using the ‘continuous hyperplasticity’ approach. This will require detailed modelling of the behaviour at the tension/compression transition, as this appears to be important. It will be possible to validate and calibrate the numerical models against the data from laboratory tests and field trials. The models will be coded in numerical structural analysis programs.

CONCLUDING COMMENTS

This paper has described research into foundations and structures for offshore wind energy structures. It is likely that shallow foundations will be a viable alternative to the more traditional piled foundation at some sites. The simplest

structure would be a monopod on a large spread footing. Current design methodologies are inadequate for a footing subjected to large moments but small vertical loads. It will be necessary to increase the effective vertical load of the structure, so that sufficient moment capacity can be provided for a reasonable foundation diameter. An alternative structure will be a jacket structure with shallow skirted foundations at the corners. In this case the foundations will be subjected to primarily vertical cyclic loading including cycles into tension, as the overall weight of the structure is low. When considering typical loadings it is likely that four legged structures will be preferable to tripod structures. Various experiments were described that can be used to investigate the performance of the foundations under these different loading regimes. The experiments were conducted in a manner so that information could be gathered to develop theoretical approaches. Such an approach, called 'continuous hyperplasticity', was introduced and a typical result illustrated. Finally a large research program into the development of design guidelines for skirted foundations for offshore wind turbines was described. This program will last for three years from February 2002.

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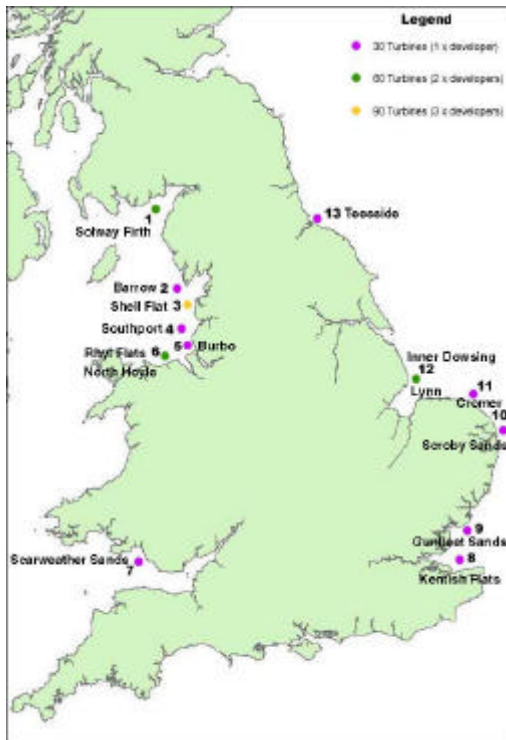


Figure 1 - Sites released around the UK for offshore wind developments (sourced from British Wind Energy Association - BWEA - <http://www.BWEA.com>).



Figure 2 - The offshore wind turbines at Blyth - the UK's first offshore wind farm (sourced from BWEA).

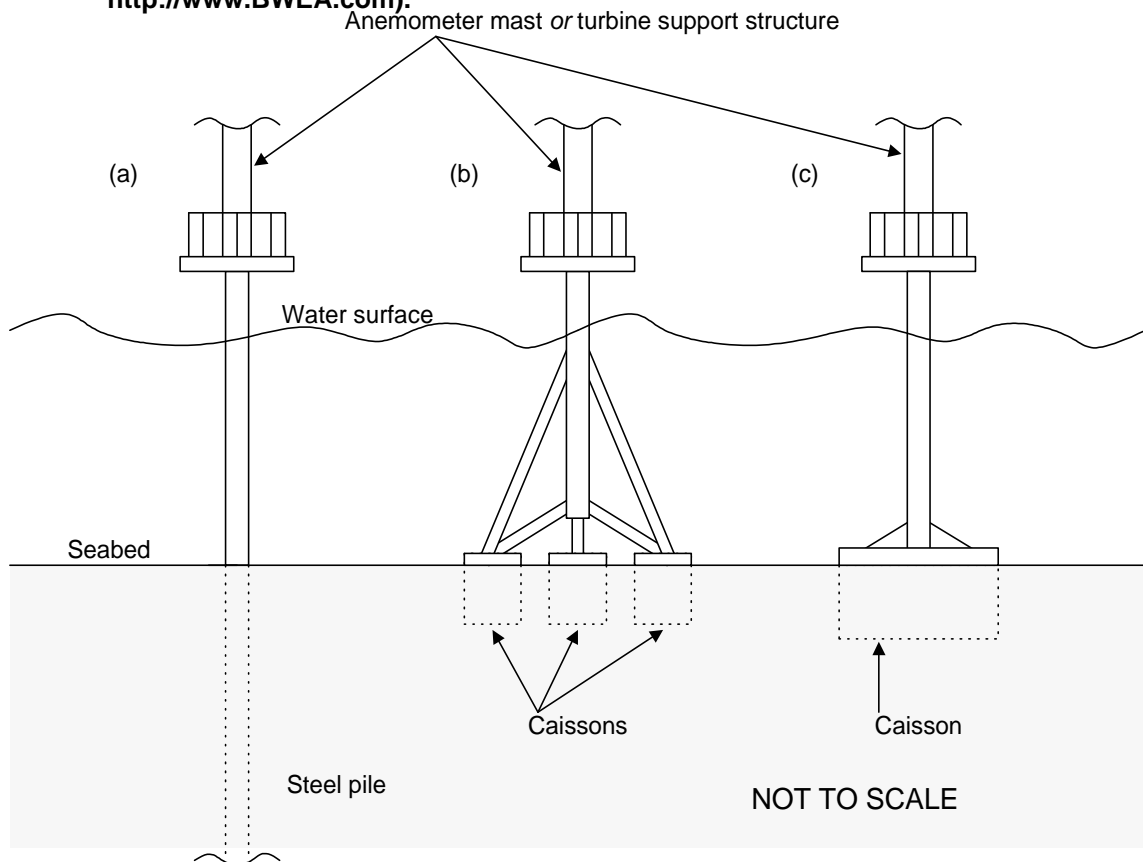


Figure 3 - Some typical options for structure/foundation systems for offshore wind applications (after Housby and Byrne [1]).

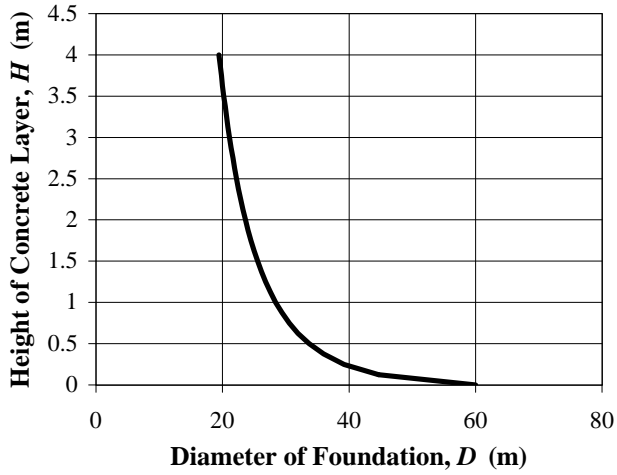


Figure 4 - Using concrete ballast to reduce diameter.

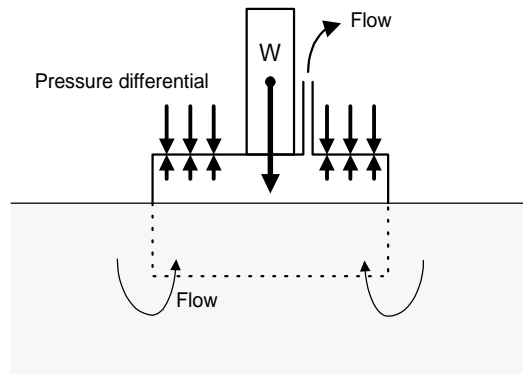


Figure 5 - Suction installation of foundations.

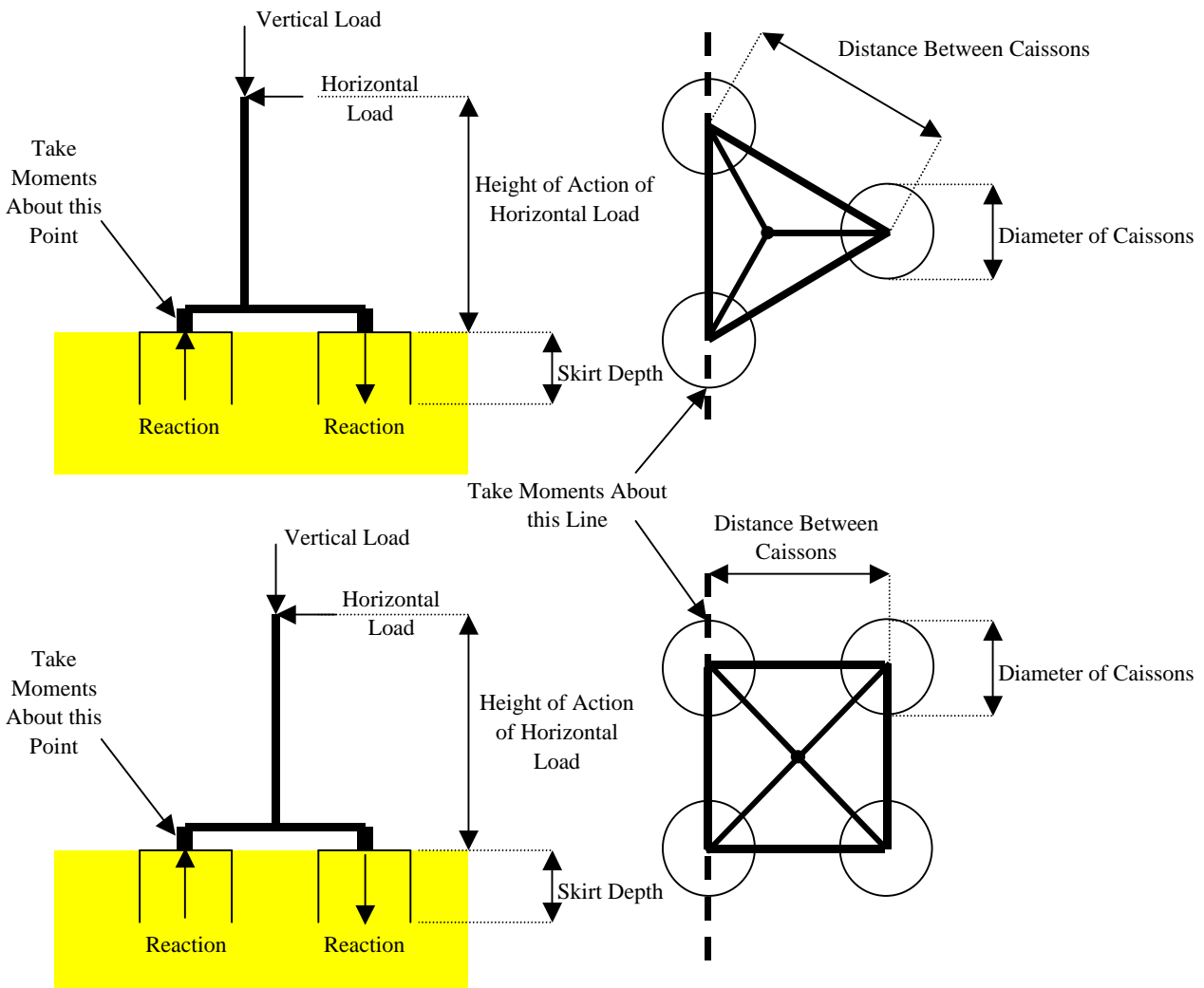


Figure 6 - The layouts for a tripod and a typical jacket type structure for the offshore wind application.

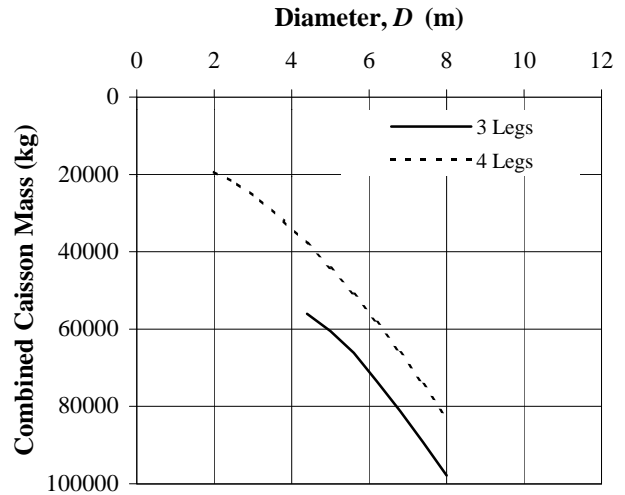
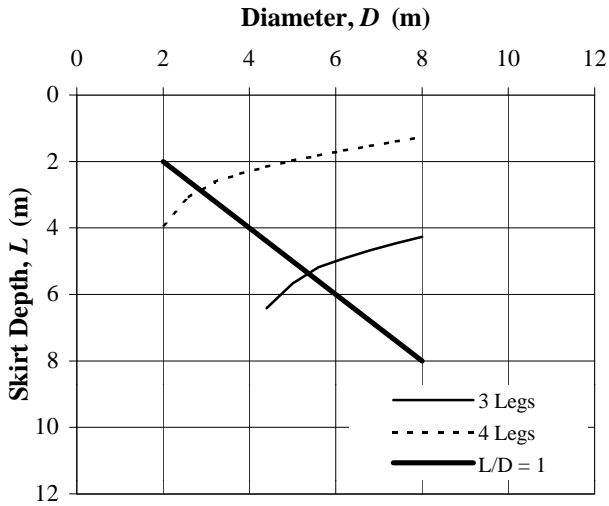


Figure 7 - Variation of caisson sizes and masses depending on structural configuration.

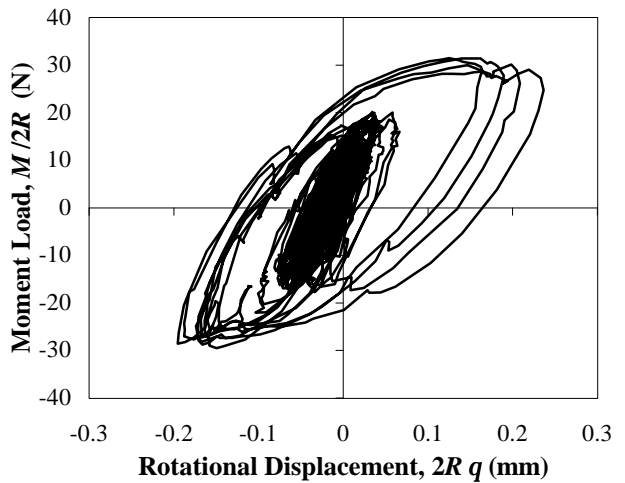
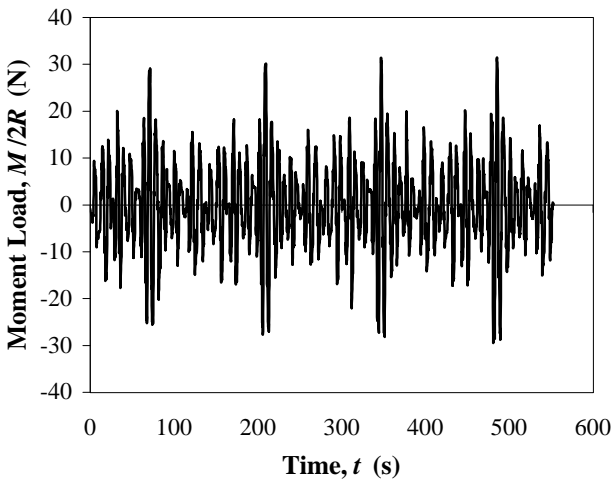


Figure 8 - A typical cyclic moment loading showing time:load and displacement:load responses.

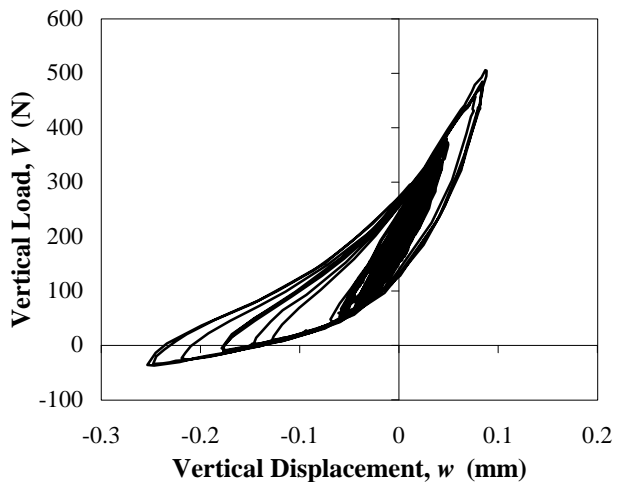
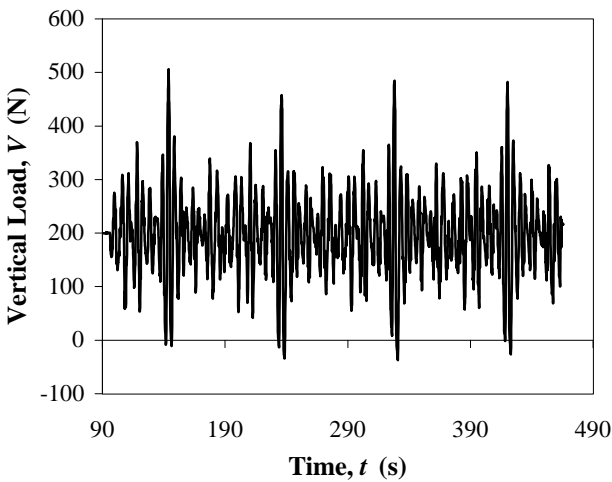


Figure 9 - A typical cyclic vertical loading showing time:load and displacement:load responses.

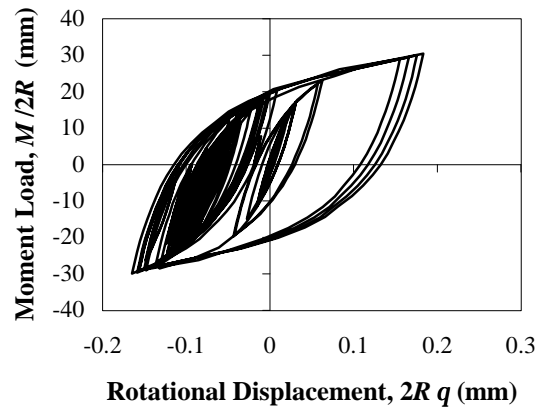
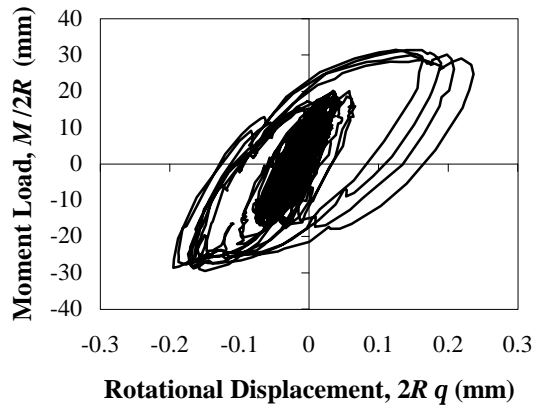


Figure 10 - Experimental load displacement response and theoretical model result.