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THE ROYAL SOCIETY

Foundations for offshore wind turbines

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An important engineering challenge of today, and a vital one for the future, is to develop and harvest alternative sources of energy. This is a firm priority in the UK, with the government setting a target of 10% of electricity from renewable sources by 2010. A component central to this commitment will be to harvest electrical power from the vast energy reserves offshore, through wind turbines or current or wave power generators. The most mature of these technologies is that of wind, as much technology transfer can be gained from onshore experience. Onshore wind farms, although supplying 'green energy', tend to provoke some objections on aesthetic grounds. These objections can be countered by locating the turbines offshore, where it will also be possible to install larger capacity turbines, thus maximizing the potential of each wind farm location.

This paper explores some civil-engineering problems encountered for offshore wind turbines. A critical component is the connection of the structure to the ground, and in particular how the load applied to the structure is transferred safely to the surrounding soil. We review previous work on the design of offshore foundations, and then present some simple design calculations for sizing foundations and structures appropriate to the wind-turbine problem. We examine the deficiencies in the current design approaches, and the research currently under way to overcome these deficiencies. Designs must be improved so that these alternative energy sources can compete economically with traditional energy suppliers.

Keywords: foundations; offshore wind turbines; renewable energy

1. Introduction

One of the most promising renewable energy sources is wind energy: electricity realized through the use of large wind turbines. Many countries use wind turbines onshore, but in the UK, where unspoilt countryside is in short supply, this option causes significant controversy. Reasons for opposing such developments are usually on aesthetic grounds, although there is a wide spectrum of opinion about the impact of wind turbines on the landscape. There is significant pressure, however, to put wind turbines offshore (like those shown in figure 1), where they will be out of sight—and perhaps out of mind? Advantageously, by moving offshore, larger structures can be developed which allow a much greater power output. It should be noted that offshore winds are not necessarily stronger, but are usually more consistent. There are also

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Figure 1. Wind turbines at Blyth, Northumberland, UK. (Image courtesy of AMEC.)

large tracts of seabed that are not used for other purposes and may be suitable for hosting wind farms.

The disadvantage is that the environmental (wind and wave) loadings on the larger structures lead to greater forces in the structure than those that would occur onshore, exacerbating the civil-engineering problem. It is necessary to ensure that a sufficient connection with the ground is provided, otherwise the structure will move irreversibly. The foundation of the structure transfers the forces from the structure to the surrounding soil. This is a critical part of the design of a wind-turbine structure. A clear understanding of the load-transfer mechanisms, from foundation to the soil, leads to increased confidence in the overall design. It is critical that the foundation can sustain all loads that may be applied, particularly during extreme environmental conditions—principally because financial loss will result if the structure fails (unlike other offshore applications where loss of life is the primary concern).

The UK Government, in the Renewables Obligation (UK Government 2002), set a firm target of 10% of electricity generated from renewable sources by 2010. This has recently been extended (UK Government 2003) to include an aspiration of 20% by 2020, and a much longer recommendation of a 60% reduction in CO_2 emissions by 2050. To illustrate the effectiveness of harnessing wind energy, Greenpeace sponsored a report by Border Wind Limited (1998), which estimated that, to produce the required 10%, 12 000 MW of wind-turbine capacity would be required.

At the time of writing (July 2003), the British Wind Energy Association (BWEA) quote data indicating that the wind-power capability installed onshore in the UK is *ca.* 580 MW in about 80 wind farms. Typical recent installations employ turbines of *ca.* 0.66 MW capacity. The turbines employed offshore are larger, with current projects mainly using 2 MW turbines. The largest turbines currently being designed are 3.5 MW, and already there is talk of 5 MW installations. Using 3.5 MW as typical of offshore installations in the next few years, the 12 000 MW target would require

Figure 2. Proposed UK offshore wind-farm sites (BWEA 2003). (Open circles, 30 turbines (1 developer); grey circles, 60 turbines (2 developers); black circles, 90 turbines (3 developers).) (Image courtesy of Crown Estates.)

about 3500 turbines. These would supply electricity equivalent to the needs of about seven million homes. Currently this need would be provided by say 12 coal- or gas-fired power stations, each supplying 1 GW, but also producing some six million tonnes of CO_2 per annum from each power station (Energy Networks Association 2003).

The Border Wind report suggests that the total seabed required to reach the 10% target would have an area of *ca.* 1200 km^2 based on the use of the 2 MW wind turbines (this area would reduce with the introduction of the larger turbines). The total area of seabed around the coast under UK control amounts to *ca.* 800 000 km^2, so the 1200 km^2 is therefore a tiny percentage of the overall area (*ca.* 0.15%). The introduction of wind turbines does not necessarily render these areas sterile, as it will still be possible for fishing and other activities to occur there. Anecdotal evidence suggests that offshore structures, likened to artificial reefs, can in fact lead to an increase in the amount of marine life in an area.

Recently, Crown Estates started the development of offshore wind energy when they released 13 sites to accommodate 540 turbines around the UK, as shown in figure 2. Most of these sites will be developed over the next few years and it is anticipated that further sites will be released by Crown Estates for development in the future (UK Crown Estates 2003). Already one experimental wind farm has been installed off the UK (AMEC 2002; Grainger & den Rooijen 2000). This was in the latter part of 2000, near Blyth. The facility consists of two 2 MW wind turbines (those shown in figure 1). The Horns Rev development of 80 turbines has recently been completed offshore from Denmark.

There are many facets to the engineering of wind turbines, including civil, electrical, mechanical and control engineering. This paper concentrates solely on the civil-engineering aspects of the designs for wind turbines. The civil engineering of the Blyth structures was reasonably simple, as the water was very shallow and the shallow geology of the area involved sound rock at seabed level. The structures were concreted into holes drilled into the bedrock.

It is clear that the seabed around the UK consists of very diverse materials, such as loose, mobile sand banks, glacial till and soft clay, so the type of foundation used at Blyth would not be suitable at many other sites. A review of the different locations released for the first phase of wind-farm development around the UK has revealed that the soils at shallow depths are mainly sands. At some locations the sand overlies clay, but this is located at some depth below the surface. This paper will therefore focus on design approaches for sandy soils, as these are likely to dominate the designs at a majority of the sites. Each site, however, may require different engineering strategies, depending on the soil conditions, such as sand density and depth to the clay stratum, as well as the strength of the underlying clay. For significant progress to be made on the UK energy targets, and in the longer term to allow the wind resource to compete on an economic basis with more traditional power sources, the development of these different engineering strategies is a priority. The following sections will discuss some novel structural and foundation systems that might be employed for offshore wind turbines. They may not be suitable for every site, but the development and testing of competing strategies will help to drive down the cost of more traditional and established engineering approaches.

2. The foundation design problem

Moving offshore will allow the use of very large wind turbines capable of supplying typically 3.5 MW (although this will probably increase with time), installed in farms of 50 or more turbines. In contrast to typical oil and gas structures used offshore, for a wind turbine the foundation may account for up to 35% of the installed cost. Currently the cost for each such turbine is estimated at £ 1.2 million per megawatt, which compares with onshore turbines at £ 0.65 million per megawatt (Musgrove 2002). The weight of each structure is relatively low, so the applied vertical load on the foundation will be small compared with the overturning load from the wind and waves. Further, it will be necessary to have a single design that can be mass-produced for use over a whole wind farm site, rather than have each structure/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.

These structures will be large; the turbine hub for a proposed 3.5 MW machine is expected to be some 90 m above the sea floor, with the rotor diameter likely to be of the order of 100 m. Initially the structures will be installed in relatively shallow water (5–20 m in depth). While installing structures offshore is hardly novel, these structures are different from typical offshore structures (usually oil and gas structures) in two respects, both related to the applied loads on the structure and hence on the foundations. The loads will clearly vary with the size of the installation, the detailed design, and the local environmental conditions, but the following figures give estimates of the values for an anticipated 3.5 MW design offshore the UK.

(i) The maximum vertical load, V, applied to the foundation would be relatively small, of the order of *ca.* 6 MN. The maximum horizontal load and applied overturning moment on the foundation would be substantial compared with the vertical load. For a 3.5 MW turbine, the applied horizontal load, H, might be *ca.* 4 MN and the overturning moment, M, *ca.* 120 MN m (or equivalent to the horizontal load being applied 30 m above the base). The foundation must be designed to resist these loads adequately.

(ii) The loads are comprised of wind and wave loads and are cyclic in nature. The worst load case is usually when the turbine is operating in moderate winds while the sea is in an extreme state. The combination of extreme sea and wind states is generally not critical, as the blades are fluttered during extreme winds to reduce the blade load and therefore the probability of blade damage. Typically, the maximum operational wind load would be *ca.* 1 MN. This would be applied at the hub (say 90 m above the sea floor) and would be relatively constant over a long time period. The current and wave loads might be *ca.* 1 MN ± 2 MN. These are applied at a much lower level, depending on the depth of water (say 10 m) and cycle at periods of *ca.* 10 s, considerably faster than the wind loads. This combination of loads translates to a resultant horizontal load of 2 MN ± 2 MN with a resultant moment of 100 MN m ± 20 MN m. This is an unusual loading case as the ratio of moment to horizontal load is fluctuating rapidly with time, rather than remaining constant as would be more typical in offshore design. Furthermore, the wave direction may not be coincident with the prevailing wind direction. Therefore, the loads (moment and horizontal) acting on the foundation may not be coincident. Note that the wind force contributes *ca.* 25% of the horizontal load but *ca.* 75% of the overturning moment, because it is applied at such a high level.

Figure 3 shows the wind turbine described above compared with a typical large jack-up rig drawn to the same scale to illustrate the differences in the loading.

One solution for the foundation is to use conventional methods such as driven piling (i.e. using large 'nails' driven into the ground with a large hammer). This is shown in figure 4*a*. However, at some sites it may prove more economical to use foundations that bear only on the surface sediments, and, in particular, foundations with perimeter 'skirts' embedded into the sea floor so that the effect of scour is mitigated, as shown in figure 4*b*, *c*. Two structural configurations are shown: option B shows a typical 'jacket' structure where there are three or four individual foundations, while option C shows a monopod structure with only one foundation. In figure 4*b* the overturning loads applied by the wind and waves are resisted predominantly by a 'push–pull' action, involving equal and opposite vertical loads at foundation level. In this design, the foundations are likely to be embedded in sand, and it will

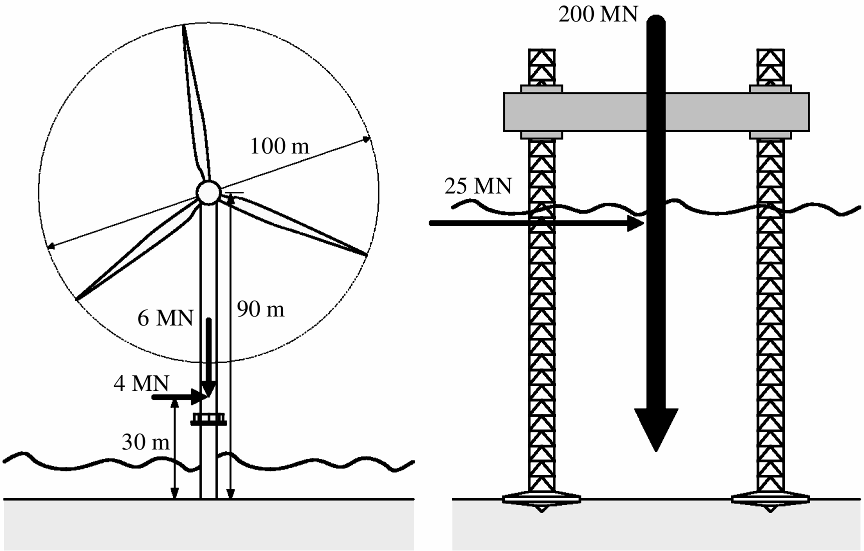

Figure 3. An offshore wind turbine and a jack-up rig drawn to the same scale showing typical loads.

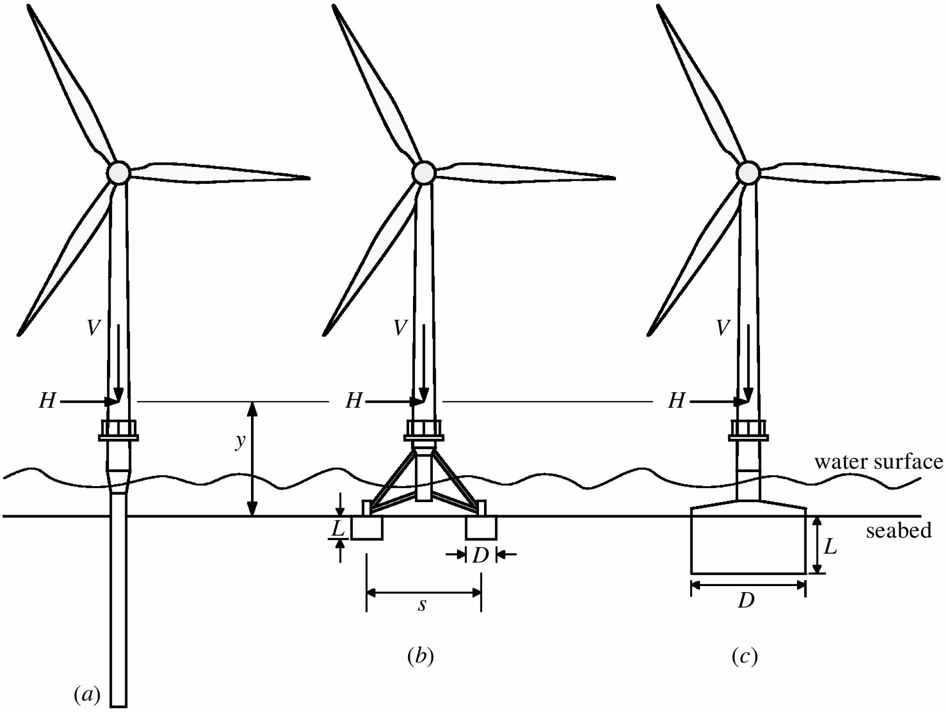

Figure 4. Proposed structures for offshore wind-turbine applications: (*a*) piled foundations (option A); (*b*) suction caisson multi-foundation structure (option B); and (*c*) suction caisson monopod (option C).

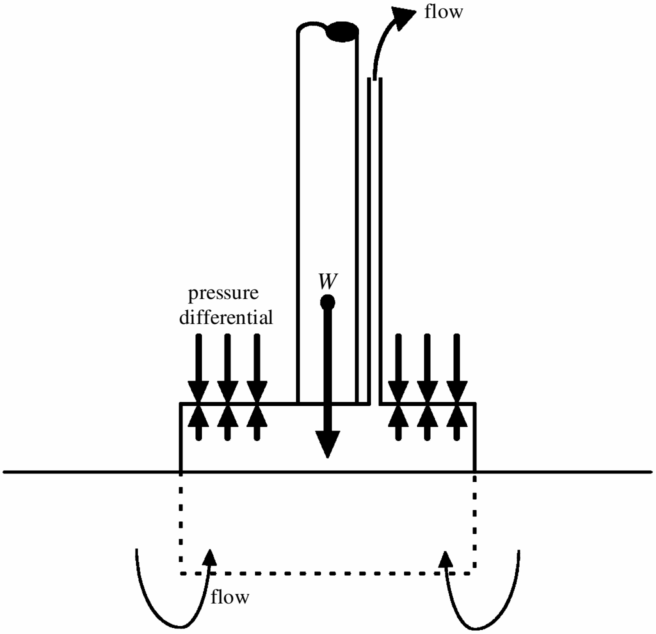

Figure 5. Suction installation mechanism.

be the response of the foundation to vertical loads that is critical. In figure 4*c*, the overturning load is applied directly to the single large foundation. In this case the caisson may be embedded solely in the sand, or may extend into clay, which is present at some of the sites, and the foundation response to an overturning moment will be critical.

These skirted surface foundations, usually called ‘suction caissons’, are a novel design. They can be installed very quickly with the aid of suction, as shown in figure 5. This ‘upturned bucket’ concept was originally developed by the oil and gas industry for securing the anchor chains of floating offshore platforms, but it is equally suitable for use with fixed structures. Once the rim makes a sufficient seal with the seabed, the water in the internal cavity is pumped out. This produces a net downwards pressure, forcing the foundation into the ground. Once the foundation is installed, the pumps are removed and any outlet/inlet valves are sealed. By comparison with traditional foundation systems, such as piles or massive concrete bases, large savings can be made on installation time and materials. The first of these can be very important to the overall budget, as offshore construction equipment is very expensive to hire. Driving piles into the ground requires heavy equipment, and usually takes a considerable time (in the order of days to weeks).

The skirted foundations have the added advantage that they can be removed easily by reattaching the installation pumps and pumping water *into* the cavity, forcing the bucket out of the ground.

3. Early developments in foundation design methods

Initially, researchers were interested in the maximum permissible load that could be applied to the footing and be sustained by the soil; this problem is known as the bearing-capacity problem. Some exact solutions to the bearing-capacity problem, dating from initial work by Prandtl (1921) for a strip footing on a weightless perfectly plastic material, have been found. However, the most important early work was by Terzaghi (1943), who suggested the superposition of the effects of soil self-weight (γ), cohesion (c) and overburden (q) for determining the bearing capacity. He defined the bearing-capacity factors N_γ, N_c, and N_q to represent the effect of the various components,

$$V_{\text{peak}} = (\tfrac{1}{2}B\gamma N_\gamma + cN_c + qN_q)A_{\text{footing}},$$

where V_{peak} is the peak vertical load that can be sustained by the foundation, B is the foundation width and A_{footing} is the plan area of the foundation. The bearing-capacity factors are dependent on the angle of friction of the soil, and various expressions have been proposed for them.

The action of a moment and horizontal load as well as a vertical load significantly complicates the bearing-capacity problem. Early research on the combined loading problem can be traced to Meyerhof (1951, 1953), Hansen (1961, 1970) and Vesic (1975). They suggested expressions, based on Terzaghi's original bearing-capacity equation, to calculate the failure load under combined loading. These procedures, still commonly taught and used today, use ad hoc factors to account for shape, depth and load inclination to adjust the allowable bearing pressure. They have been the principal method of calculating the capacity of foundations under combined loads in the offshore industry.

4. A simplified foundation design study

Prior to exploring the more recent approaches to designing offshore foundations, it is possible to carry out some simplified static design calculations. The aim of the study is to investigate the effects of typical foundation sizes and spacing (in the case of option B), and the effect of critical parameters, such as vertical load, on the capability of the foundation to sustain the horizontal and moment loads. The loads applied to the structure will be as given above. Typical factors of safety and soil parameters are adopted. We will not consider option A, as that involves a routine engineering approach, but will concentrate on options B and C, as they involve novel solutions.

Option B consists of a multi-footing design with suction caisson foundations. There are two facets to the design: the separation of the foundations (defined by a spacing s, see figure 4*b*); the dimensions of the foundations (defined by a diameter D and skirt length L). The critical calculation for establishing the *separation* of the footings relates to the case where the structure rotates about two downwind foundations. In this case the overturning moment (M) will consist of the net horizontal load (H) acting at a height y above the foundations (as shown in figure 4). The net restoring moment will consist of the vertical load (V) acting through the centre of gravity of the structure. In the case of the quadruped structure the centre of gravity will be a distance of $\frac{1}{2}s$ from the rotation point if the spacing of the foundations is s. For the tripod structure this distance will be $s/(2\sqrt{3})$. To simplify the calculation it is assumed that the upwind foundations cannot provide any tensile resistance. This is a conservative assumption, as the foundations may be able to sustain tension through friction along the skirts. The minimum required spacing between foundations can be

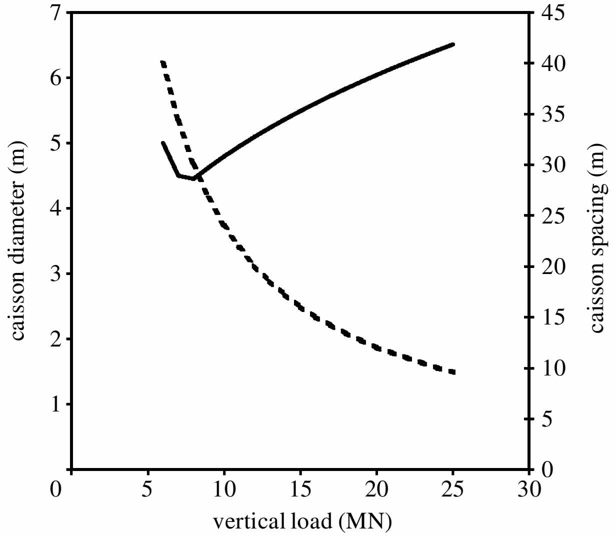

Figure 6. Design of a quadruped structure (——, diameter; – – –, spacing).

found as:

$$s = \begin{cases} 2\dfrac{Hy}{V} = 2\dfrac{M}{V} & \text{for a quadruped structure,} \\ 2\sqrt{3}\dfrac{Hy}{V} = 2\sqrt{3}\dfrac{M}{V} & \text{for a tripod structure.} \end{cases}$$

The critical calculation for the *capacity* relates to the case where the wind direction is such that only one foundation is downwind. The capacity of the foundation can be found using conventional bearing-capacity theory:

$$V = \tfrac{1}{4}\pi D^2(\gamma' L N_q + \gamma' \tfrac{1}{2} D N_\gamma).$$

Note that there will be no difference in the size of the caisson foundations for tripod and quadruped structures as the vertical load is the same in both cases. A secondary calculation for this design case is used to check that the horizontal capacity is sufficient. Figure 6 shows results for a quadruped structure for a variety of vertical loads starting from $V = 6$ MN. The aspect ratio (length divided by diameter) of the caisson is kept constant at 0.5. Initially, the caissons are 5.0 m in diameter and 2.5 m in length and are positioned at the corners of a 40 m × 40 m square. As the vertical load increases, which could be the result of adding ballast, the size of the structure could be reduced. Interestingly, the size of the foundation initially reduces as the horizontal loading dominates, but then increases as the vertical load becomes critical. Adding ballast is clearly favourable up to about $V = 15$ MN, after which there are diminishing returns.

Option C is difficult to evaluate, as there are currently no standard design calculations that can be adopted. On the evidence of a very small number of model tests undertaken by Byrne (2000) and Byrne *et al.* (2003), we postulate that there is an approximately linear relationship between M and V at low vertical loads such that

$$\frac{M}{D} = \left(f_1 + \frac{f_2}{k}\right)^{-1}(V + f_3 W), \tag{4.1}$$

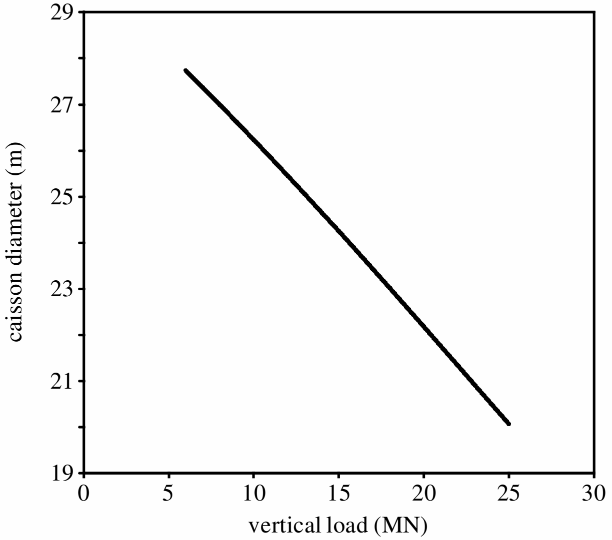

Figure 7. Design of a monopod foundation.

where $k = M/(DH)$ is the ratio of moment to horizontal load and $W = \frac{1}{4}\pi D^2 L\gamma'$ is the weight of the plug of soil within the caisson. Based on the model tests, the following factors have been determined: $f_1 = 3.26$, $f_2 = 1.073$ and $f_3 = 0.71$. Further model tests are currently under way to evaluate these factors. The first term in the small brackets on the right-hand side of (4.1) refers to the vertical load applied to the foundation, while the second term includes a proportion of the weight of the soil plug within the caisson. Using this simple approach, assuming an aspect ratio (L/D) of 0.2 and a vertical load applied to the foundation of $V = 6$ MN leads to a diameter of 27.7 m and a skirt length of 5.54 m. Figure 7 shows how the foundation diameter varies as further vertical load is added. It is clear that the increase in vertical load, say through ballast, is very beneficial for the design. A controlling factor on the aspect ratio for this type of caisson will be whether enough suction can be generated for the caisson to be installed.

Although in principle, as shown from the calculations above, this new type of foundation could be used, engineers are generally conservative in their approaches until a new technology is proven to be acceptable. This requires that a robust and conservative design framework, validated by tests, be developed. The calculations shown above give preliminary estimates of the sizes of the caissons required for different structural configurations. These are the types of calculations that might be carried out during the conceptual design phase. To progress the design further it is necessary to carry out advanced computational analyses.

5. The computational model

The optimal structural configuration will only be achieved by accounting for the complex interaction of the structure with the wind, wave and soil. This can be achieved by using numerical-analysis techniques. Fatigue of structural components and of the foundation, as well as the ultimate capacities, are just some of the issues to be addressed. The dynamic excitation forces will consist of the waves on the structure, the wind on the turbine blades and the interaction between the blades and the structure as the blades rotate. These will all affect the loading of the foundation on the soil. Obviously, the structural configuration will be important, as it will be important to design the structure so that its natural frequencies are such that resonance with the frequencies of the excitations can be avoided.

Computational analyses of offshore structures requires: the modelling of the structural response, including dynamics, of the structure; the evaluation and modelling of the wind, wave and current environment; and the accurate modelling of the interaction of the structure with the ground through the foundation. These components are described briefly.

(i) Structure: the dynamics of structures lends itself well to computer simulation, and is usually achieved using techniques such as the finite-element method. If the structure remains elastic, then it is efficient to carry out computations in the frequency domain using modal analysis techniques, but if there are non-linearities present (as there will be when the foundation is properly modelled) it is necessary to carry out a full time-domain analysis, which is more demanding on resources.

(ii) Environmental loading: to obtain the water velocities with depth it is necessary to understand the fluid mechanics, and in particular theories of large ocean waves. There has been much recent research in this area (Tromans *et al.* 1991; Taylor *et al.* 1995), particularly as more data from instrumented offshore platforms become available. The outcome is that the extreme waves can now be simulated efficiently using deterministic methods. Each extreme wave can be embedded into a randomly generated sea state, to enable the response of dynamically sensitive structures to be studied. These approaches will need to be modified for shallow water, which will be the case for the majority of the wind farm sites.

(iii) Foundations: the traditional soil mechanics theories of bearing capacity are not suited for implementation into numerical-analysis programs. Early structural analyses have therefore adopted unrealistic assumptions about the foundation, assuming for instance that they are pinned (no rigidity), fixed (infinite rigidity) or can be treated as linear springs. There is a pressing need to develop models that replicate the foundation behaviour more accurately.

In the case of (i) and (ii) there has been substantial progress in improving the theoretical understanding, and therefore the accuracy of numerical predictions of the loading and response, so we concentrate here on developments in modelling of the foundation (iii).

6. Recent developments in foundation-modelling methods

A more recent approach to understanding the bearing capacity of foundations under combined loading has been through concepts of strain-hardening-plasticity theory. These theories reproduce the results of experiments well, and in particular provide information about displacements as well as loads. The response of the foundation

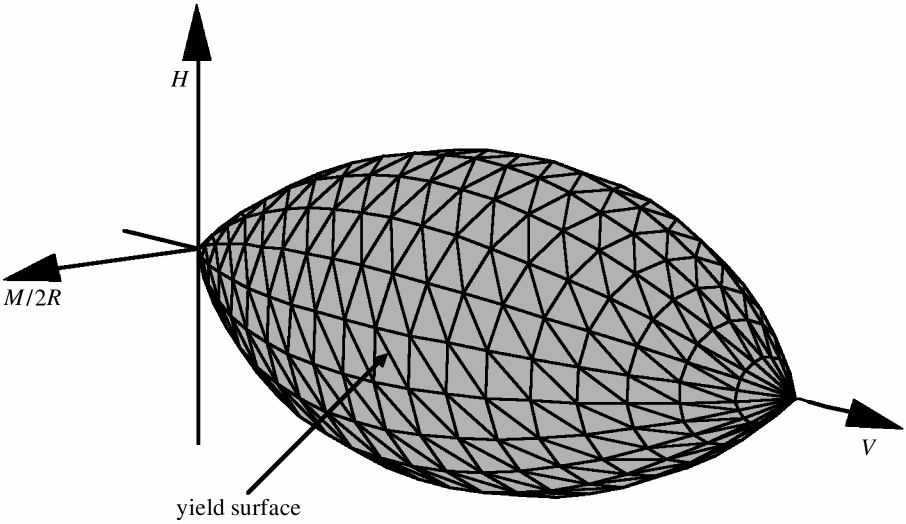

Figure 8. A yield surface for flat footings on sand.

is expressed in terms of force resultants on the footing and the corresponding displacements. The particular advantage of this approach is that the strain-hardening-plasticity framework is consistent with the time-domain analyses used for the structures, so it is a vehicle for incorporating the behaviour of the soil directly into the structural analysis.

This type of approach to the problem has its roots in the work of Roscoe & Schofield (1957), but has been developed much more recently (see, for example, Tan 1990; Gottardi *et al.* 1999; Martin & Houlsby 2000). A large body of experimental research has been carried out to develop these models. The state-of-the-art theoretical models at present are those based on this experimental research, including model B for clay (Martin 1994; Martin & Houlsby 2000, 2001) and model C for sand (Gottardi *et al.* 1999; Houlsby & Cassidy 2002; Cassidy *et al.* 2002). The components of the plasticity models include:

(i) a yield surface, such as that shown in figure 8, which defines the allowable combinations of load;

(ii) the strain-hardening expression, which defines how the yield surface expands or contracts;

(iii) a flow rule which defines the plastic displacements at yield;

(iv) a model for the elastic response within the yield surface.

The theoretical model links displacements to the loads applied to the footing, but properly accounts for the nonlinearities observed in experimental tests. If the combination of load is such that the load point is within the yield surface, then just elastic displacements occur. If the load is such that the load point reaches the yield surface, then 'plastic' behaviour occurs with the displacement increments defined by the flow rule. Various experimental techniques have been developed to determine the components of the theoretical model to be accurately calibrated against data.

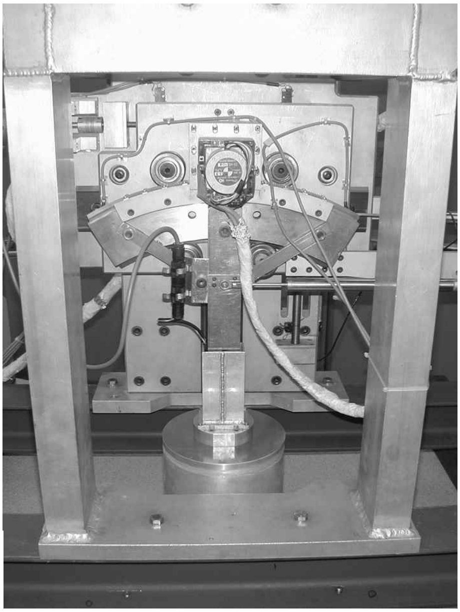

Figure 9. Three-degrees-of-freedom loading at the University of Oxford.

This approach was first used to derive models for the spudcan footings on mobile drilling units (see, for example, Schotman 1989; Martin & Houlsby 1999; Cassidy *et al*. 2001). The models are used in the numerical assessment of the rigs, where the connection with the ground can significantly affect the dynamics of the structure and therefore has an impact on the assessment of the structure's suitability at a given site. Wind-turbine structures, like jack-ups, are dynamically sensitive structures, and modelling their foundations can be achieved using similar approaches.

7. Shortcomings in foundation modelling

The theoretical models outlined above, based on plasticity theory, represent the current state of the art. They are a significant improvement on the traditional soil-mechanics approach to the foundation and are able to be implemented into commercial numerical codes. They are, however, still restrictive in two important ways.

(*a*) *Planar loading*

The models described above (models B and C) are based on experiments carried out using the loading device shown in figure 9. This assumes that the loads acting on the foundation are in one plane only, and therefore represent a three-degrees-of-freedom system (Martin & Houlsby 2000; Gottardi *et al*. 1999). The first author (with funding from The Royal Society) is extending this understanding to out-of-plane loads, so that the foundation response is described for the full six-degree-of-freedom loading (as defined in figure 10). A computer-controlled loading rig is currently being commissioned. This will be capable of applying the complex six-degree-of-freedom loading sequences. An accurate understanding of the response of the foundation to the loading regimes appropriate to the wind turbine, where the horizontal load and the moment may not be coincident, will be critical.

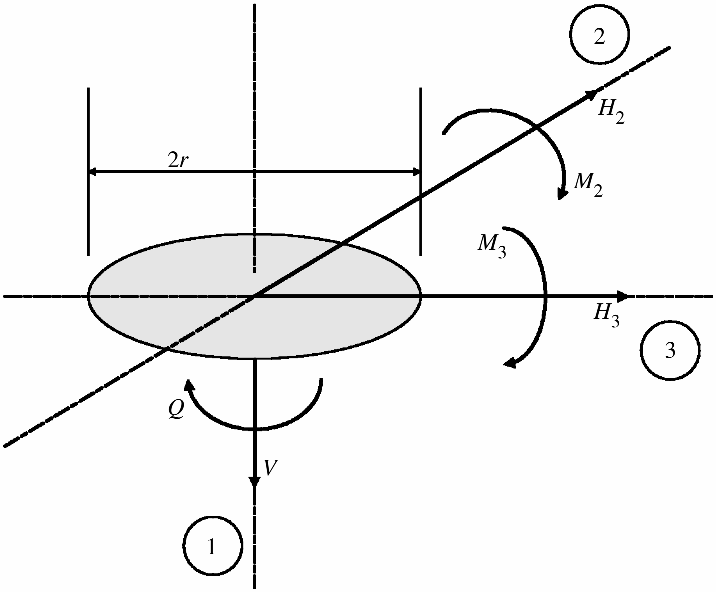

Figure 10. Six-degrees-of-freedom loading on a disc of diameter D.

(*b*) *Monotonic loading*

The theoretical models are not able to replicate accurately the behaviour of the foundation when subjected to repetitive loading, such as that shown in figure 11*a*. They were developed from the results of monotonic-loading tests, and do not incorporate effects of rate or load reversal. For example, figure 11*b* shows how a plasticity model would model a test where increasing cycles of stress are applied to the footing. Initially the footing load state is located at the apex of the yield surface. When a moment is applied to the footing, the yield surface expands as plasticity occurs. On unloading, a much stiffer response occurs, as the footing load state is now entirely within the yield surface. On reloading, plasticity only occurs once the footing load state again reaches the yield surface. This clearly does not model well the behaviour shown in figure 11*a*.

8. Future developments for foundation modelling

To improve the foundation modelling to a level comparable with the modelling of structural components and environmental conditions, it will be necessary to address the shortcomings mentioned above. The extension of the models to six degrees of freedom is already in progress and is conceptually straightforward. The more difficult task of attempting to model the cyclic loading is described in the following.

Cyclic-loading data, from experimental investigations, for foundations on sand are very sparse. Typical vertical cyclic-load time-history and load-displacement results are shown in figure 12 (Byrne & Houlsby 2002). The stiffness of response changes as tension is applied to the foundation. Byrne & Houlsby (2002) observed that the monotonic tests passed through the extreme points of the cyclic-loading tests. This was further confirmed by a series of experiments conducted by Johnson (1999), where the mean loads applied to the foundation were 50 kN and the foundation diameter

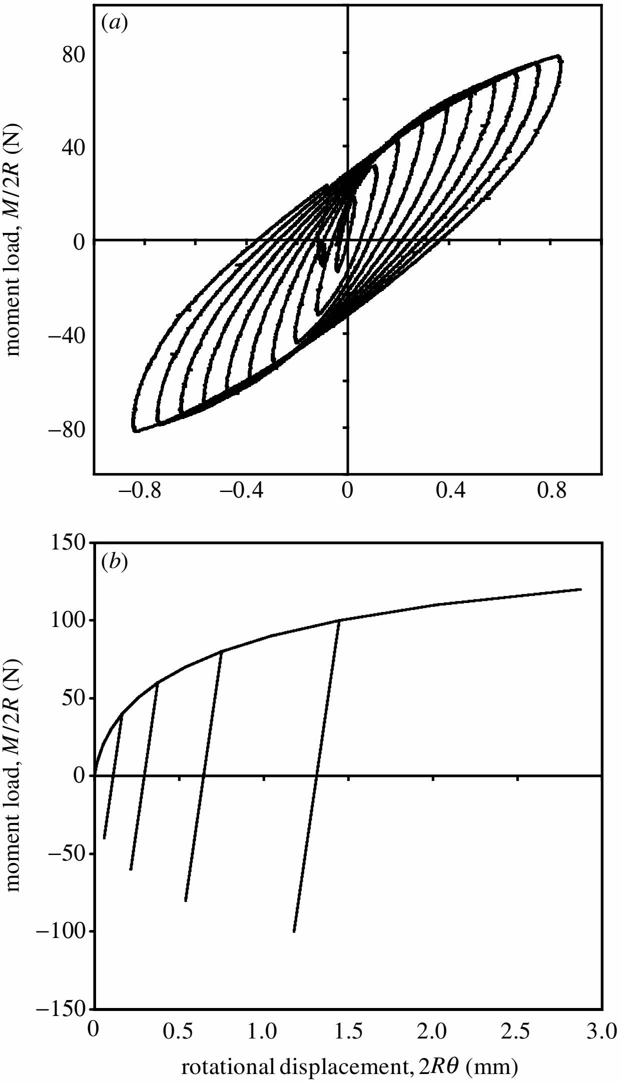

Figure 11. Constant-vertical-load cyclic-moment tests: (*a*) laboratory result and (*b*) theoretical result using a single-surface-plasticity model (after Byrne *et al.* 2002*a*).

was 300 mm (i.e. substantially larger than the loads applied for the example in figure 12). Similar observations were made for combined cyclic loading. Figure 11*a* shows a moment cyclic-loading test with the vertical load kept constant at 200 N. The response is hysteretic, and the stiffness reduces as the strain level increases. A monotonic-loading test carried out under the same conditions was observed to pass through the extreme points of the cyclic-loading test (shown in figure 13). This is an example of a frequently observed material response known as Masing behaviour (Masing 1926), which is of significant consequence in developing theoretical models for the response.

An extension of the single-surface plasticity theories, described above, is to include multiple yield surfaces so that a gradual transition of stiffness can be approximated.

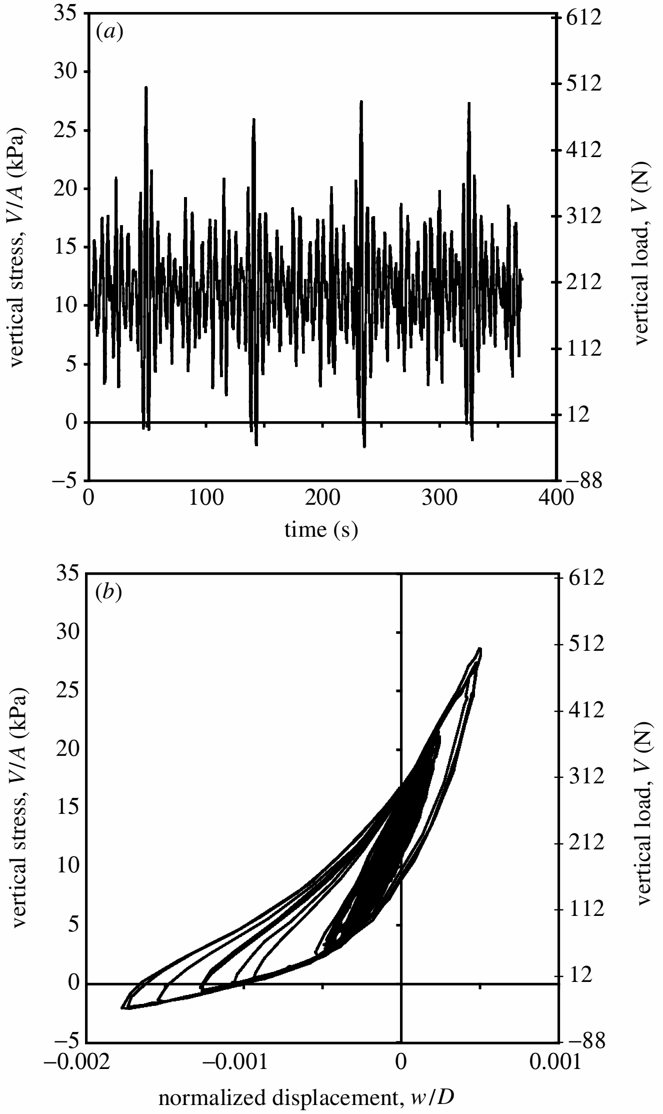

Figure 12. Pseudo-random vertical cyclic-load tests: (*a*) time history; (*b*) load-displacement response (after Byrne & Houlsby 2002).

Each surface must also be accompanied by a plastic potential to describe the direction of plastic flow. This type of theory has been developed for constitutive modelling of soils (see, for example, Houlsby 1999). The main drawback is that a number of parameters must be specified for each yield surface. For accurate modelling this leads to a rather unwieldy theory. More recently, advances have been made in an approach termed 'continuous hyperplasticity' (Houlsby & Puzrin 2000; Puzrin & Houlsby 2001*a*, *b*). This approach to plasticity theory has been formulated in such a way that any theory developed within it is guaranteed to obey thermodynamic principles.

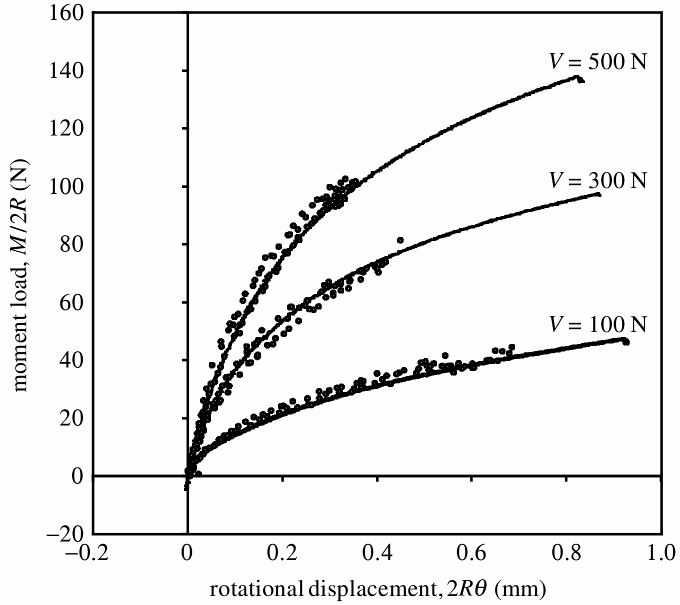

Figure 13. Monotonic tests passing through the extreme points of cyclic tests.

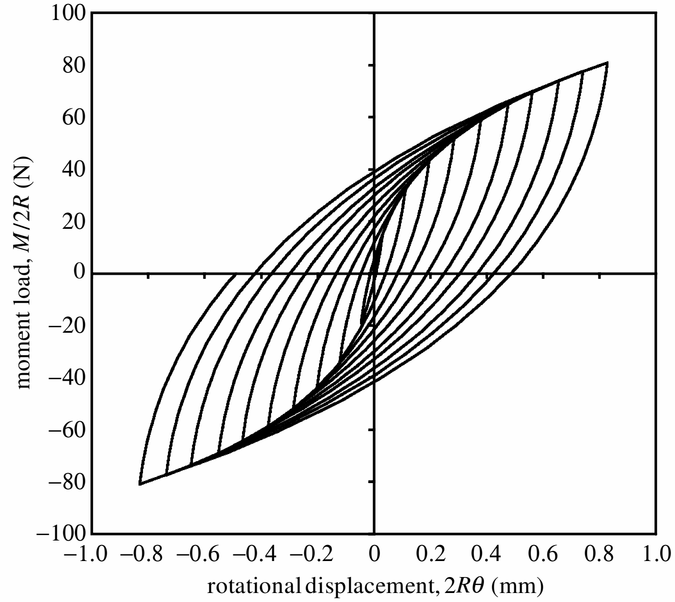

Figure 14. 'Continuous hyperplasticity' simulation of the result shown in figure 11.

In essence the theory replaces the 'plastic strain' in conventional plasticity theory with a continuous field of an infinite number of plastic strain components, each associated with a separate yield surface. This is achieved within a manageable mathematical framework by deriving the plasticity theory for a dissipative material entirely from two potentials. The first is the Gibbs free energy or the Helmholtz free energy.

The second potential is the dissipation function. For the case of the infinite field of plastic strains, these potentials are functionals (‘functions of functions’) of the plastic strain and its rate. Conventional plasticity theory is a special case of the new approach. Both Puzrin & Houlsby (2001*b*) and Byrne *et al.* (2002*a*) have shown that ‘continuous hyperplasticity’ can model the Masing behaviour described above accurately as shown in figure 14. The result is therefore that theories can be constructed in which responses of the character shown in figure 11*a* can be modelled accurately and with computational efficiency.

To facilitate the development of a cyclic-loading model, aimed directly at the suction caissons for wind turbines, a £ 1.5 million research project is being funded by the Department of Trade and Industry (DTI), Engineering and Physical Sciences Research Council (EPSRC) and a consortium of industrial partners (as described by Byrne *et al.* (2002)). This project began in August 2002 and will last until mid 2005. The main aim is to produce cyclic-loading models, based on theoretical and experimental research, that can be implemented into commercial finite element packages. Part of the project will involve using the models to investigate and compare the various designs, such as those given above, for offshore wind turbines. A key component of the research is to gain an understanding of how the experimental results at laboratory scale translate to prototype scale. To this end, a number of field trials, on caissons of diameters up to 3 m, will be carried out starting in December 2003. The programme also involves extensive laboratory testing. The results of this research will be fed into the design process for offshore wind turbines as they become available.

It is anticipated that some wind turbines using novel foundation systems may be developed for installation within the next three to five years. In any case, it is highly probable in the very near future that the offshore wind resource will be harvested by large numbers of offshore wind turbines.

9. Conclusions

We have described a topical civil-engineering problem that impacts significantly on society—that of erecting large wind turbines offshore to reap the ‘green energy’ from wind. A critical component of the design will be the foundation. Current offshore-engineering strategies for the foundations, such as piles, may not be suitable for these structures to be cost effective at locations around the UK. New techniques of design and analysis must be developed, such as for suction-installed skirted foundations, which may be more cost effective at some locations. This will include the development of better theoretical models for including foundation response within structural analyses. The most promising of these models are based on strain-hardening plasticity, but are currently capable of modelling only monotonic loading. A future challenge is to extend these models to cyclic and transient loading so that they can be used within time-domain structural analyses. A research programme focused on solving this problem for wind-turbine foundations is currently under way. It is hoped that some of the developments outlined here will impact on practice in the near future.

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AUTHOR PROFILES

Byron W. Byrne

Although he was born in Sydney, New South Wales, Byron Byrne (right) grew up in Esperance, a remote coastal town 720 km southeast of Perth, Western Australia. He moved to Perth in 1990 to study at the University of Western Australia, graduating in 1995 with first class honours in Civil Engineering and a Bachelors degree in Commerce. He then worked as a research assistant at both the University of Western Australia and the University of Oxford, as well as for a small Perth-based engineering consultancy, Advanced Geomechanics. In October 1996 he moved to Balliol College, Oxford, as a Rhodes Scholar, to study for a doctorate in Engineering. Upon completing his DPhil in 2000 he took up an 1851 Research Fellowship awarded by the Royal Commission for the Exhibition of 1851 and was independently elected to a Fellowship by Examination at Magdalen College. In 2001 he was appointed to a Departmental Lecturership and, in 2003, to an Official Fellowship at Magdalen College. Currently, at the age of 31, his main research interests relate to problems involving soil mechanics and offshore engineering. As well as his research and teaching, he represents Magdalen College on the Joint Venture Executive that manages the Oxford Science Park (http://www.oxfordsp.com), a joint-venture property-development project between Magdalen College and Prudential Assurance Ltd. His recreations are mainly sporting, and include cricket, at which he won four University Blues while a graduate student at Oxford.

Guy T. Houlsby

Guy Houlsby (left) studied at the University of Cambridge, where he graduated with first class honours with distinction in Engineering in 1975. After two years in civil-engineering consultancy he returned to Cambridge, obtaining his PhD in 1981. In 1980 he moved to the University of Oxford, first as a Junior Research Fellow at Balliol College, and then as a University Lecturer and Tutorial Fellow at Keble. In 1991 he became Professor of Civil Engineering at Oxford, and a Fellow of Brasenose College. He was elected an FREng in 1999. His main research interests are in geotechnical engineering, related to problems of offshore construction, and on the understanding of the fundamental mechanics of soils. He is honorary Editor of the scientific journal *Géotechnique*. His recreations include ornithology, woodwork and rowing.