International Journal of Environmental Studies, 2013
http://dx.doi.org/10.1080/00207233.2013.798496

Routledge
Taylor & Francis Group

An investigation into the use of push-in pile foundations by the offshore wind sector

DAVID IGOE†, KENNETH GAVIN†* AND BRENDAN O'KELLY‡

†School of Civil, Structural and Environmental Engineering, University College Dublin, Newstead Building, Belfield, Dublin 4, Dublin, Ireland; ‡Department of Civil, Structural and Environmental Engineering, Trinity College Dublin, Dublin 2, Dublin, Ireland

This paper presents the results of a field test performed to study the effects of installation method on the load–displacement response of piles used to support the offshore wind turbines. An instrumented open-ended model pile was installed by jacking in a deposit of medium-dense sand. Pile jacking has environmental benefits over the traditional method of pile driving which can cause noise and vibration damage to the marine mammals. Pile installation by jacking was shown to enhance the pile-soil stiffness response during compression loading. Residual stresses, generated during the installation process, caused the pile to exhibit a relatively soft stiffness response during tension loading. Environmental loading caused by wind and waves which causes piles that support jacket structure to experience tension loading and the serviceability limit state of the foundation to these loads governs the design.

Keywords: Offshore; Piles; Sand; Foundation; Wind; Loading; Design

Introduction

Dynamic pile installation, otherwise known as pile driving, employs large drop hammers that can cause significant noise and vibration over the period of installation. For offshore wind turbines, foundation costs represent 25–35% of the total capital expenditure costs, with installation times for the three or four piles required to support offshore jacket structures typically taking between two to four days. Therefore, the construction of foundations for modern wind farms, comprising of hundreds of turbine locations, could result in significant environmental effects over a prolonged period [1].

De Jong and Ainslie [2] and Snyder and Kaiser [3] have found that driving large diameter offshore piles can cause severe effects to sea mammals including hearing damage or loss, and displacement from breeding grounds over distances of tens of kilometres from the location of the pile installation. Bailey et al. [4] reports the measurements of noise levels during the installation of 1.8 m diameter piles, which were driven 44 m into the sea bed to support a 5 MW wind turbine founded on a jacket structure installed in the Firth of Moray in north-east Scotland. Whilst the noise levels recorded suggested that no form of hearing impairment would occur for the majority of indigenous mammals who were at least 100 m from the piling hammer, harbour porpoises which are highly sensitive to noise experienced

*Corresponding author: Email: kenneth.gavin@ucd.ie

disturbance at distances of up to 70 km from the piling operations. In recognition of these environmental concerns, Governments and consenting authorities are introducing strict guidelines on piling noise which are a significant challenge for an industry in which pile sizes and, therefore, piling hammers are becoming increasingly larger.

Alternative ways of installing piles, including methods which minimise soil displacement such as rotary, bored and augured piles are increasingly being considered by offshore developers. In recent years, jacking or push-in piling has been growing in popularity in onshore market as it minimises noise and vibration which are both critical in an urban environment. Because of the prevalence of large jack-up vessels in the offshore sector, there it is potential for using jacked or pushed-in piles to support offshore wind turbines. In this paper, the technical aspects of said potential are investigated using an instrumented open-ended model pile pushed into a medium-dense sand deposit. The development of the pile load capacity and residual loads during installation were carefully monitored. After installation, the pile was subjected to a series of load tests to compare its static resistance and stiffness response to both compression and tension loading, respectively. In addition, a sequence of cyclic loading was applied to replicate the environmental loading which is experienced by the offshore piles.

Background

The choice of foundation system used to support offshore wind energy turbines mainly depends on water depth, local ground conditions, availability of materials and installation vessels and ultimately cost. The majority of the support systems commonly used to support turbines (figure 1) were developed for the offshore oil and gas sectors, where loading conditions are invariably significantly different from those encountered by wind turbines [5,6]. Many of the current foundation systems rely on the use of open-ended steel tubes (piles) that are driven into the seabed in order to provide resistance to structural and environmental applied loads. This includes monopiles (see figure 1): single piles having diameters in the range of 5–8 m that are driven usually between 25 and 50 m into the seabed. The vertical self-weight of offshore wind turbines is usually only a small component of the overall loads; horizontal and moment loading produced by wind and wave loading are

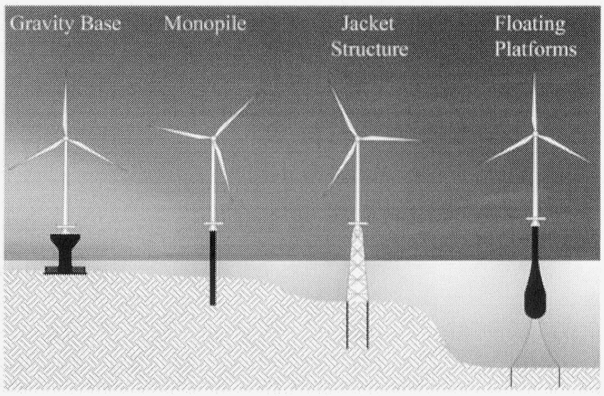

Figure 1. Foundation concepts for offshore wind turbines (after Doherty and Gavin [7]).

much more significant. Monopiles resist the applied loads by bending or tilting, thereby mobilising large horizontal earth pressures in competent near-surface soils. Monopiles have been used successfully to support the wind turbines in water depths of up to 30 m and support, approximately, 75% of wind turbine structures installed to date [6,7].

The choice of foundation system used to support offshore wind energy turbines depends on water depth, local ground conditions, availability of materials and installation vessels and ultimately cost. The majority of the support systems commonly used to support turbines (see figure 1) were developed for the offshore oil and gas sectors, where the loading conditions are significantly different to those encountered by wind turbines [5]. Many of the current foundation systems rely on the use of open-ended steel tubes (piles) driven into the seabed to provide resistance to the structural and environmental applied loads. This includes monopiles (see figure 1), which are single piles with diameters in the range of 5–8 m and which are usually driven between 25 and 50 m into the seabed. The vertical self-weight of offshore wind turbines is usually a small component of the overall loads encountered and horizontal and moment loading as a result of wind and wave loads (environmental loading) are much more significant. Monopiles resist applied loads by bending or rotating, thus mobilising large horizontal pressures in the near surface soils. Monopiles have been used successfully to support wind turbines in water depths of up to 30 m and support, approximately, 75% of the wind turbines installed to date [7].

In deeper water, jacket structures, although considerably more expensive than monopiles, provide a proven solution in the offshore environment. The structural frame of the jacket transfers the vertical and horizontal loads and moments on the superstructure into axial loads on the piles installed in the sea bed. Therefore, these piles tend to be longer, yet they have smaller diameters, usually in the range 1.5–3 m, as they are relying on vertical shear stresses developed at the pile-soil interface and end-bearing stresses at the pile tip to develop axial vertical resistance rather than relying on the bending or rotational capacity of the pile. Due to the dominant effect of wave and wind loading on these relatively light and flexible structures, the axial uplift or tension capacity of the piles used to support the jacket structures normally governs their design [8].

Push-in or jacked piles have been proven to provide an environmentally friendly alternative to driven piles when used in urban settings, where noise and vibration generated by pile driving creates discomfort for humans and can result in structural damage to surrounding buildings and infrastructure. White et al. [9] compared the noise levels generated during typical onshore piling operations with the operation of the Giken push-in 'Silent Piler' which is capable of installing a pile with a jacking force of 4 MN. The noise level measured at a distance of 1 m from the the generator used to power the push-in pile rig was 75 dB. This was compared very favourably to the measured noise levels in the range 98–135 dB for piling hammers, with the lower value being measured for an enclosed drop hammer and the upper value corresponding to standard double acting air and diesel hammers. As a point of reference, the peak noise level recorded by Bailey et al. [4] at a distance of 100 m from the offshore pile during hammer used in the Firth of Moray was 205 dB.

In addition to environmental benefits offered by a push-in or jacked pile systems, piles installed using minimal load cycles may also develop higher load resistance. Gavin and Lehane [10] presented data from field tests which indicated model open-ended piles installed in dense sand by jacking, developed greatly enhanced stiffness at the pile base when compared with driven piles. Deeks et al. [11] performed static compression load tests on pipe piles which were pushed into loose to medium-dense sand at the Takasu test site, Kochi, Japan. They compared the piles axial stiffness during loading with the stiffness response of driven and bored piles reported in literature. They suggest that because of the beneficial effect of pre-loading of the soil that occurred during installation and the presence of large residual loads, the stiffness of the push-in piles were between 2 and 10 times higher than conventional driven and bored piles.

Jacked piles have been proven to have both technical and environmental benefits over driven piles, as they have comparable ultimate resistance values to driven piles and tend to exhibit a higher stiffness response when loaded. For offshore wind turbines, where serviceability requirements (i.e. minimising displacements) are critical, an enhanced soil-structure system stiffness is a major benefit when considering the response of the structure to environmental loading.

Description of field experiments

Site description

The pile tests were performed in a natural sand deposit in Donabate, North County Dublin, Ireland. The ground conditions at the site and the sand properties have been reported previously by Igoe et al. [12,13]. A shell and auger borehole drilled adjacent to the pile test location revealed that the ground profile at the site (see figure 2) comprises of a layer of sand to 4.2 m below ground level (bgl), overlying a deep layer of stiff clay. The sand layer was divided into an upper medium-dense sand deposit, a lower dense sand and gravel deposit. The upper and lower deposits were separated by a thin (0.2 m) peat layer. The groundwater table was located at the base of the upper sand layer. Three Cone Penetration Tests (CPT's) were conducted in the immediate vicinity of the test pile. The CPT end resistance, q_c values in figure 2 show the upper sand to have q_c values in the range 5–10 MPa, whilst the lower sand had values in excess of 10 MPa. The CPT q_c profile in both the upper and lower sand layers was affected by the presence of the peat layer, with q_c reducing to a minimum value of 1.2 MPa in the peat layer, but rapidly increased to

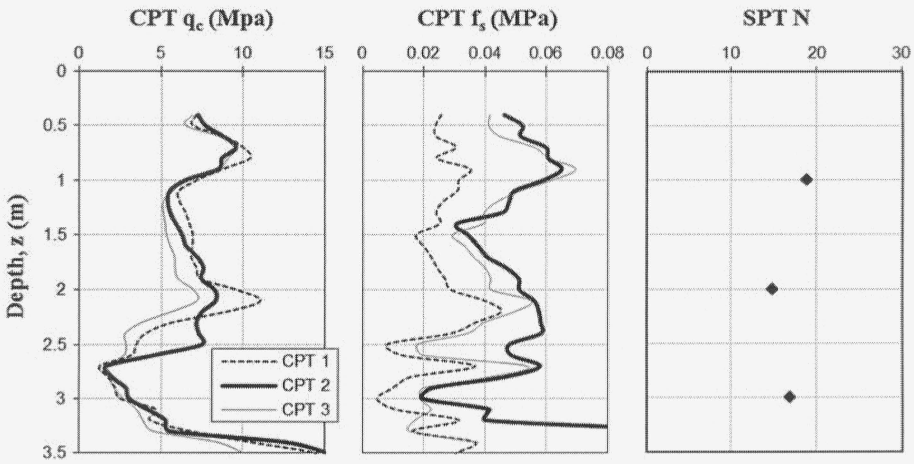

Figure 2. (a) CPT cone resistance q_c, (b) CPT sleeve friction f_s and (c) SPT blow count profiles from Donabate.

between 12 and 15 MPa in the underlying dense sand layer. The cone sleeve friction values in figure 2(b) are more variable than the q_c values, although the profiles at the three CPT test locations were generally consistent. The standard penetration test (SPT) *N* values in the sand ranged from 15 to 20.

UCD instrumented pile

The pile tests were performed in a natural sand deposit in Donabate, North County Dublin, Ireland. The pile housed three levels of sensors, each level measuring total radial stress and pore pressure (see figure 3). The sensors used were Kyowa PS-5KA miniature pressure transducers, 6 mm in diameter with a maximum capacity of 500 kPa. The total stress was measured directly using a sensor mounted flush with the pile surface. The pore pressure units involved mounting a porous ceramic disc flush with the pile surface in front of the pressure transducer, and saturating the gap in between with a viscous fluid so that only fluid pressure acted on the sensing face. The total and pore pressure sensors were mounted diametrically opposite to each other at $h/D = 1.5$, 5.5 and 10.5 from the pile base where h/D is the distance from the pile base normalised by the external diameter. A depth gauge allowed for the monitoring of the plug length during installation and load testing. Electrical resistance strain gauges were glued to the walls of the inner and outer tubes at multiple levels along the pile, with each level housing four strain gauges at 90° offsets. These allowed the distribution of load acting along the inner and outer tubes to be determined. The base resistance, which is a combination of the soil plug and annular resistances, was measured from the gauges on the inner tube. The annulus and plug resistances were estimated using the strain gauge extrapolation technique described in Paik and Salgado [14]. Full details on the design of the pile can be found in Igoe et al. [15].

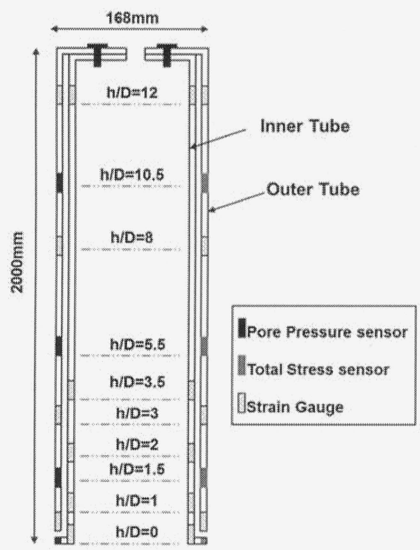

Figure 3. The UCD open-ended model pile instrumentation layout.

Testing program

The test pile was pushed from an initial starting hole with depth of 0.9 m to a final depth of 2.8 m bgl using a 20 t capacity CPT truck (figure 4). The pile was jacked at a rate of 20 mm/s and installation was paused after each 100 mm jacking stroke in order to monitor the pile plug development. The pile was installed in 19 jacking strokes. During the first six jacking strokes, a small load was maintained on the pile head during pauses between the jacking strokes. This ensured that the pile remained vertical during the early stages of installation. For subsequent jacking strokes, the pile head load was completely removed between each stroke in order to allow for the measurement of the development of residual loads. After pile installation was complete, two displacement transducers were attached to an independent reference beam to allow the vertical displacement of the pile head to be monitored during load testing. A description of the static and cyclic load testing procedure followed is summarised in table 1. A static compression maintained load test was first performed by applying the load in incremental steps with each step increase ≈15% of the final installation load. The load was applied manually using the CPT jack and the resulting pile head load was determined by observing the load cell output in realtime. This static load test was followed by two cyclic compression load tests, in which 50 cycles of loading were applied at load levels corresponding to 33 and 66% of the piles compressive load capacity. A static tension test was then performed by attaching a specially fabricated loading hanger to the pile head and adopting the same procedure as that utilised in the static compression test (see figure 5). This test was followed by two cyclic tension tests, with each including 50 load cycles representing 40 and 80% of the pile static tension capacity, respectively.

Experimental results

Pile installation stage

The degree of soil plugging occurring during installation of an open-ended pile has an important influence on the axial pile capacity [14,16]. The development of the soil plug is conventionally described using the Incremental Filling Ratio (IFR, defined as the ratio of

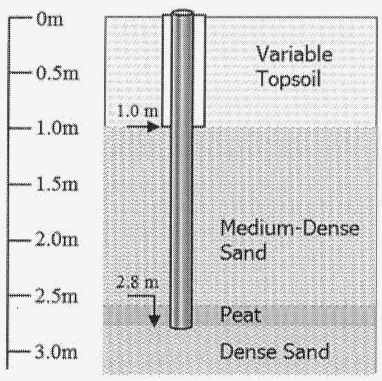

Figure 4. Section including installed pile.

Table 1. Summary of installation of load testing of the pile.

Test type	Details	Comments
Installation	Jacked in 100 mm strokes	Starting depth = 0.9 m, final depth = 2.8 m
Static compression test	Maintained load test	Total displacement = 30 mm
Cyclic compression test	50 cycles from 0 to 33%	0.1 mm permanent displacement
Cyclic compression test	50 cycles from 0 to 66%	36 mm permanent displacement
Static tension test	Maintained load test	Total displacement = 34 mm
Cyclic tension test	50 cycles from 0 to 40%	0.04 mm permanent displacement
Cyclic tension test	50 cycles from 0 to 80%	55 mm permanent displacement

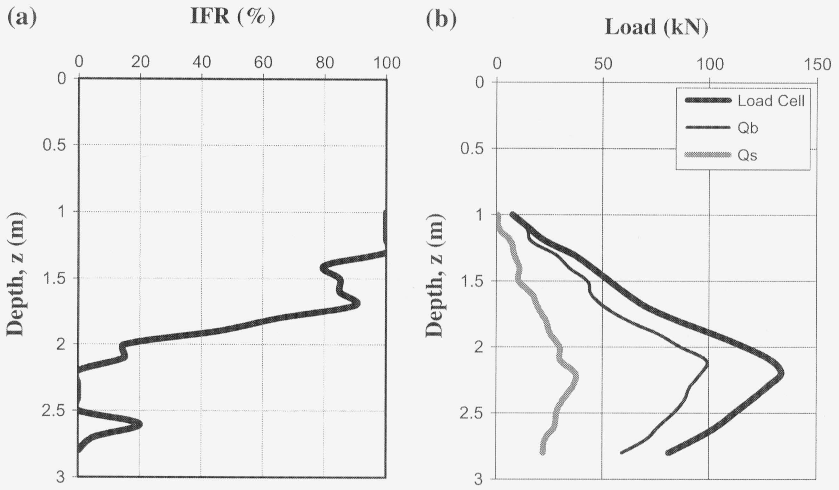

Figure 5. (a) IFR profile and (b) installation resistance during installation (after Igoe et al. [13]).

change in plug length to change in pile penetration during a driving or jacking stroke). The IFR profile measured during pile installation is shown in figure 5(a). The pile was approximately fully coring (i.e. IFR $\approx$100%, meaning the soil level within the inner pile tube was coincident with the ground surface level) to a depth of 1.7 m bgl; partly coring (i.e. 0% < IFR < 100%) to a depth of 2.2 m bgl after which it became fully plugged (IFR = 0%), and approximately remained in this condition to the final installation depth of 2.8 m bgl.

Figure 5(b) shows that the total pile resistance and its shaft and base resistance components (Q_s and Q_b, respectively) all increased in magnitude until the pile tip had reached 2.2 m bgl. The rate of increase in the total pile resistance increased slightly as the plug began to form (with noticeable increases occurring for depths of 1.2 and 1.7 m bgl). For greater depths, the pile resistance decreased, mirroring the drop that occurred in CPT q_c resistance (figure 2(a)). The base resistance provides the majority (60–80%) of the total pile resistance. The shaft resistance peaked at 2.2 m bgl, after which it decreased slowly, despite increasing the shaft length for further penetration. This reduction in shaft resistance below 2.2 m bgl was most likely caused by a combination of effects including the lower pile shaft entering the weaker peat layer and the upper section of the shaft experiencing an increased number of installation load cycles.

Residual loads

During installation of the pile, the residual load developed between jacking strokes was measured. This was possible as the strain gauges did not experience drift from their 'no-load' position, and the inferred residual load was completely removed after a large displacement tension test was performed. The residual base load, Q_{rb}, developed during installation is shown in figure 6(a), and it is noted that residual loads were not measured during the first six jacking strokes as the pile head load was not removed for operational reasons. During the early stages of the pile installation, residual load initially developed at the pile annulus, Q_{rann}, and only started to develop in the pile plug, Q_{rplug}, after the pile had become fully plugged (IFR = 0) at 2.2 m bgl. At the end of installation, a residual load of 12 kN remained with 3.5 kN contributed from the annulus and 8.5 kN from the pile plug. A profile of the residual loads remaining on the inner and outer tube at the end of installation is shown in figure 6(b). It is evident that the entire residual plug load (measured from the strain gauges on the inner tube) was mobilized within the first three diameters of the pile tip.

Static compression load test

The load–displacement response of the pile measured during the static load test is set out in figure 7. The pile developed a total load resistance of 65 kN after a pile head displacement of 8 mm (approximately 5% of the pile diameter, *D*). At the start of the load test, the pile loads were in equilibrium (under zero external applied load) and the residual compressive base load of ≈12 kN was resisted by tension shaft resistance (of −12 kN) mobilised along the external pile shaft. The piles' compressive shaft resistance developed rapidly during the load test, becoming fully mobilised at a pile head displacement of 1.5 mm (≈% of *D*). The base load continued to increase until the pile resistance was fully mobilised. The importance of accounting for the residual load present in the pile is clearly evident from the measured data in figure 8. If the strain gauges were zeroed prior to the load tests, as it

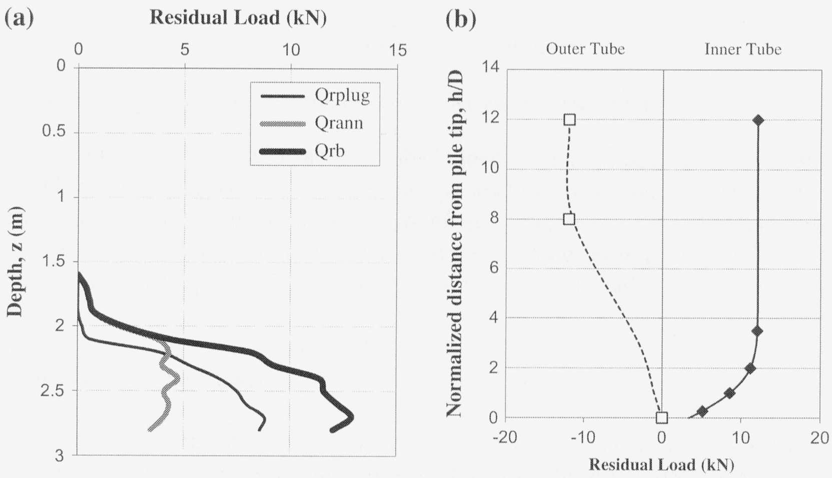

Figure 6. (a) Residual base load during installation and (b) residual load profile at the end of installation.

(2) During each load cycle, the tensile (negative) residual shaft friction was fully

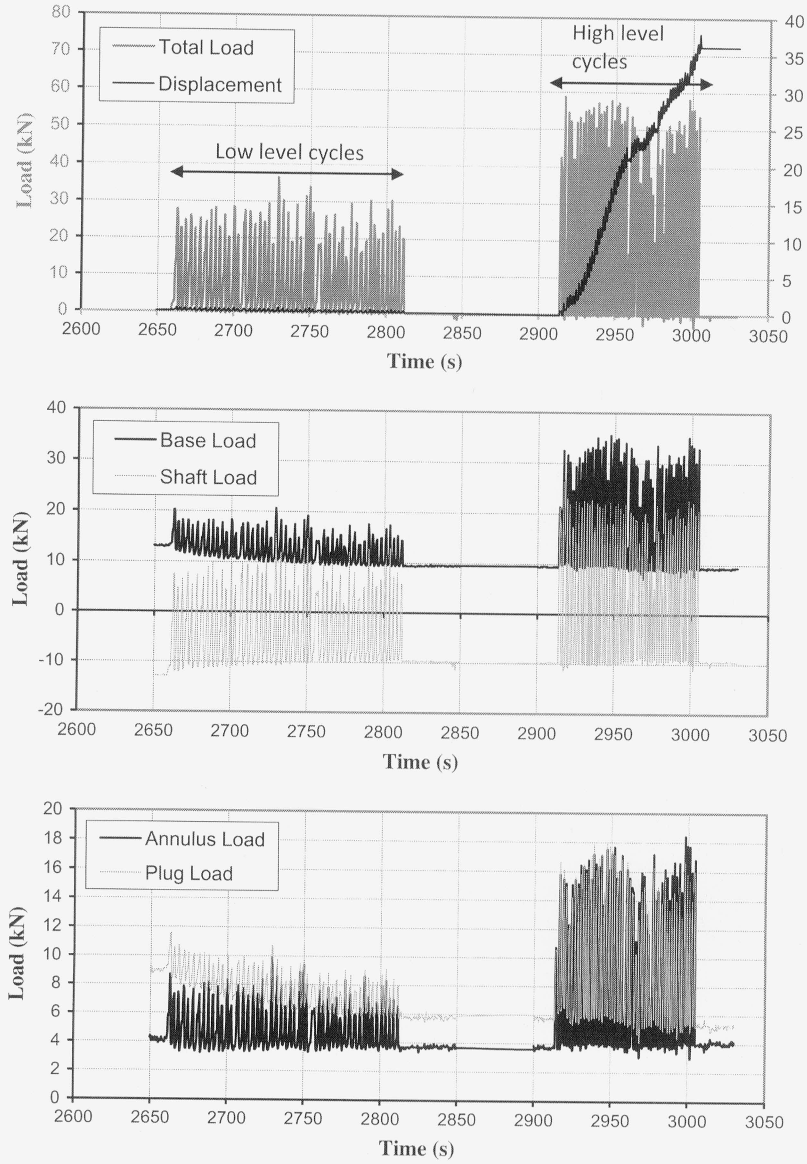

Figure 8. Time profile of the mobilized loads during cyclic compression tests.

(3) The residual base and shaft loads reduced from their initial (post installation) values of 12 to ≈9 kN during low level cycling. Virtually all of this reduction was seen to occur due to a reduction in the residual plug load over the first 30 cycles (see figure 9(c)).

During high level compression cyclic load test, see figure 8, the following behaviour was noted:

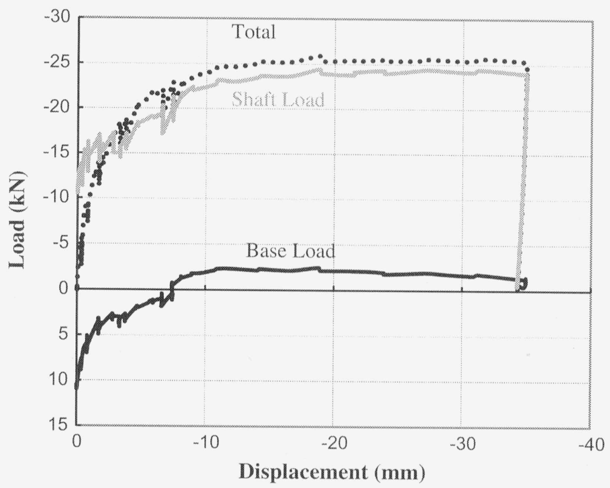

Figure 9. Load displacement response during static tension load test.

(1) The higher level cycles resulted in significant displacement accumulation and unstable behaviour. The pile experienced 36 mm of non-recoverable displacement over the 50 high level cycles (equating to approximately 0.7 mm displacement per cycle).
(2) The high level load cycles mobilised shaft resistance equal to peak value measured in the compression load test (20 kN), while only mobilizing 70% of the compression base resistance from the static test (approximately 30 kN).
(3) The residual base and shaft loads did not change during the high level cycling, with a posttest value of 9 kN remaining on the pile.

Tension static load test

After the completion of the compression cyclic load tests, a static tension load test was carried out. The load displacement curve from the static tension test is shown in figure 9. The following observations were made:

(1) The pile developed a total pull-out capacity of 25 kN, which was mobilised after a pile head displacement of 15 mm (9% of *D*). The total shaft load measured was 24 kN with a 1 kN contribution from the pile and soil plug self-weight.
(2) The mobilised tension shaft resistance (24 kN) was higher than the compression shaft resistance (20 kN) which is in contrary to the findings of many researchers (Jardine et al. [19], Lehane et al. [20] amongst others) who suggest that the tension shaft resistance should be lower. This behaviour may be explained by the fact that the lower pile shaft (where the majority of shaft friction is usually developed) was in a weak soil layer (contributing only a small amount to the capacity). The cyclic load test conducted between the two static load tests may also have affected the pile response.

(3) The pile exhibited a relatively soft stiffness response during tension loading when compared to the response to static loading in compression. This is evident in figure 9(a) which plots the pile capacity mobilised during the static load tests as a ratio of the ultimate pile capacity against normalised pile head displacement. Because of the contribution of end bearing resistance; when the pile was loaded in compression it developed a much higher resistance (65 kN) than it did during tension loading (25 kN). Despite this, the pile response was stiffer during compression loading. The reason for this is the effect of residual stresses/loads in the pile developed during installation. At the start of the compression load test, the pile base was pre-loaded by 12 kN. The shaft and base resistance had been fully mobilised during the last installation jacking stroke and the pile was essentially re-loaded during the static load test. By contrast, at the start of the tension load test (where the pile resistance depends almost exclusively on the external tension shaft resistance) the pile base was pre-loaded by 9 kN, see figure 9(b), at the start of the test. As the external load was applied to the pile head, the large base load was gradually removed. Such load reversals usually result in a reduced stiffness response as the stress-path is reversed and the confining stress at the pile base is reduced. This leads to the overall soft load-displacement response of the pile during tension loading.

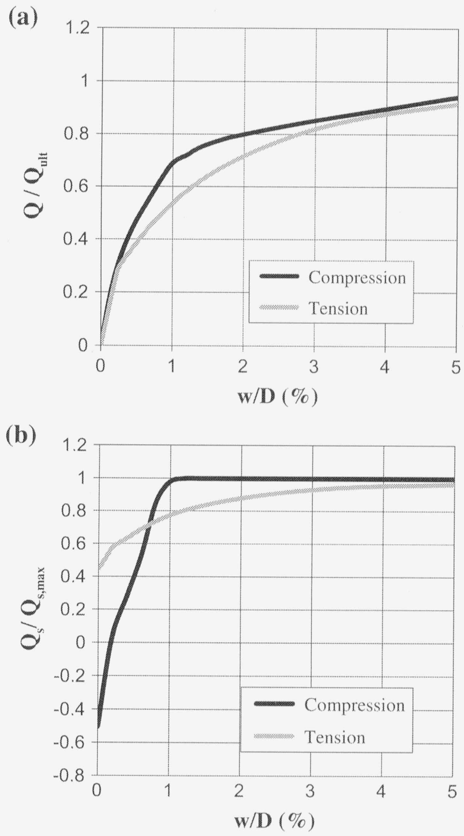

Figure 10. Comparison of pile stiffness during compression and tension loading.

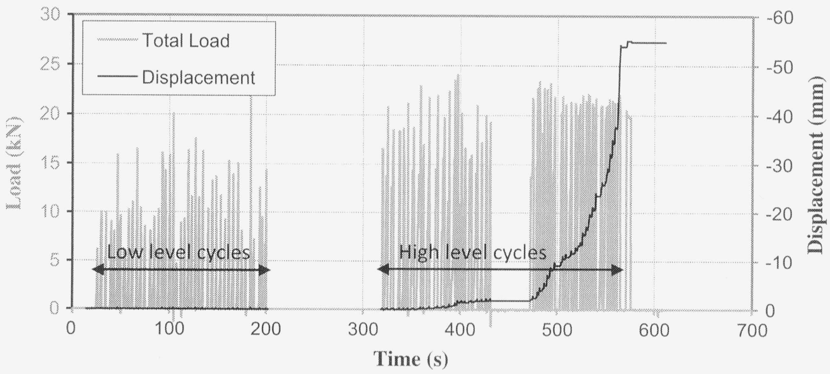

Figure 11. Time profile of the mobilized load and displacement during cyclic tension tests.

Tension cyclic load test

Following the tension static load test, two tension cyclic load tests were performed with the load varying from 0 to 10 kN and 0 to 20 kN (representing 40 and 80%, respectively, of the tension capacity) (figure 10). The results of the cyclic tension tests are shown in figure 11. The following trends are noteworthy:

(1) For low level load cycles (from 0 to 10 kN) each cycle was virtually fully elastic, with a permanent non-recoverable displacement of 0.04 mm accumulated over the 50 cycles. This level of cycling can be regarded as stable, requiring many hundreds or thousands of cycles before failure would occur.
(2) The higher level cycles resulted in unstable behaviour with a displacement of 55 mm accumulating over the 50 load cycles. The first 25 cycles of 0–20 kN only resulted in a 2 mm displacement, with the further cycles accelerating the rate of displacement causing fully unstable behaviour.
(3) Some degradation of the shaft capacity occurred during the high level cycles. The penultimate cycle from 0 to 21.5 kN results in a displacement of ≈10 mm suggesting a 15% reduction in tension capacity from its pre-test value.

Summary and conclusions

With the trend of locating wind turbines at greater distances offshore, jacket structures are being increasingly used by the offshore wind sector, with structural and environmental loading on the wind turbine and lattice structure transferred into axial tension/compression loading that is resisted by piles installed in the seabed. The findings of the instrumented model pile tests performed to assess whether the tubular piles pushed or jacked into sand deposits provide a suitable foundation system from a geotechnical standpoint are summarised as follows:

(1) Large residual loads developed on the pile once it became fully plugged. At the end of installation, the residual base load (and the opposing residual shaft shear stresses) were equivalent to ≈50% of the piles tension shaft capacity.

(2) During low level cyclic compression testing (between 0 and 33% of the piles static capacity), the residual load was seen to reduce by, approximately, 25% with virtually all of the reduction occurring in the pile plug.
(3) The pile had a larger tension shaft resistance than compression resistance. This is attributed to the weak soil adjacent the lower pile shaft and large tensile residual shear stresses present on the upper portion of the shaft.
(4) The piles' stiffness was much lower in tension loading than it was in compression loading.
(5) The piles responded well to low cyclic loading levels of between 33 and 40% of the piles ultimate resistance, corresponding to typical factors of safety adopted in traditional design codes (factor of safety (FOS) in the range 2–3). At these load levels, pile displacements were low and largely elastic and recoverable.
(6) At high cyclic load levels the piles were unstable, with large accumulations of plastic displacement being observed during both compression and tension load cycling, and reduced pile capacity occurring during cyclic loading in tension.

An important finding of the study was that the development of residual loads during pile installation was strongly linked to the degree of pile plugging experienced during driving. The residual stresses mobilised had the effect of causing the pile to develop a very stiff response to compression loading. However, the stiffness response during tension loading was much lower in comparison. Given the importance of tension loading stiffness to the performance of tubular piles supporting offshore jacket structures, this finding has serious implications for the use of such piles in industry. If they are to be used to support offshore structures, it is important that a means to minimise the plugging of piles during installation maybe adopted; for instance, by using larger diameter piles, internal plug restrictors, vibrators or other methods. This would have the added advantage of reducing the pile base resistance during installation, thereby also reducing the jacking force necessary to obtain the required pile penetration into the seabed in order to resist uplift loading in-service with an adequate FOS.

Acknowledgements

The authors thank Frank Dillon, Dr Paul Doherty and Dr David Gallagher of University College Dublin for constructing the UCD test pile, Dave McAuley and Martin Carney for performing the CPT tests and in assisting with the pile installation and load tests at the Donabate site. The Department of Civil, Structural and Environmental Engineering, Trinity College Dublin, are acknowledged for the use of the TCD CPT truck.

References

[1] Betke, K., Glahn, M. and Matuschek, R., 2004, Underwater noise emission from offshore wind turbines. Proceedings of the Joint French–German Meeting, Congrès Français d'Acoustique and Deutsche Jahrestagung für Akustik, Strasbourg, pp. 591–592.
[2] De Jong, C.A.F. and Ainslie, M.A., 2008, Underwater radiated noise due to the piling for the Q7 offshore wind park. Proceedings of Acoustics 08, Paris, pp. 117–122.
[3] Snyder, B. and Kaiser, M.J., 2009, Ecological and economic cost-benefit analysis of offshore wind energy. *Renewable Energy,* **34**, 1567–1578.
[4] Bailey, H., Senior, B., Simmons, D., Rusin, J., Picken, G. and Thompson, P.M., 2010, Assessing underwater noise levels during pile-driving at an offshore wind farm and its potential effect on marine mammals. *Marine Pollution Bulletin,* **60**, 888–897.

[5] Doherty, P., Casey, B. and Gavin, K.G., 2010 July, Laying the foundations for sustainable energy: the geotechnical challenges facing the offshore wind sector. *The Engineers Journal,* **64**(6), 262–266.
[6] Arshad, M. and O'Kelly, B.C., 2013, Offshore wind-turbine structures – a review. Proceedings of ICE Energy, Vol. 166. Manuscript ID. ENERGY D-12-00019, Thomas Telford, London.
[7] Doherty, P., Gavin, K.G. and Gavin, K.G., 2012, Laterally loaded monopile design for offshore wind farms. *Proceedings of ICE, Energy,* **165**(EN1), 7–17.
[8] Gavin, K.G., Igoe, D.I. and Doherty, P., 2011, Piles for offshore wind turbines: a state of the art review. *Proceedings of ICE Journal, Geotechnical Engineering,* **164**(4), 245–256.
[9] White, D., Finlay, T., Bolton, M. and Bearss, G. (Eds.), 2002, *Press-in Piling: Ground Vibration and Noise During Pile Installation*, (Reston, VA: American Society of Civil Engineers, ASCE Geotechnical Special Publication). Vol. 1, pp. 363–371.
[10] Gavin, K.G. and Lehane, B.M., 2003, End bearing resistance of small diameter pipe piles in dense sand. Proceedings of the International Conference on Foundation Behaviour, Dundee, September, pp. 321–330.
[11] Deeks, A.D., White, D.J. and Bolton, M.D., 2005, A comparison of jacked, driven and bored piles in sand. Proceedings of the 16th International Conference on Soil Mechanics and Geotechnical Engineering, Osaka, pp. 2103–2106.
[12] Igoe, D., Gavin, K., O'Kelly, B. and Gallagher, D., 2006, Base resistance of an open-ended pile installed in medium dense sand. Third National Symposium on Bridge and Infrastructure Research in Ireland, Dublin, 12–13 October, Vol. 1, pp. 243–250.
[13] Igoe D.J.P., Gavin K.G. and O'Kelly B.C., 2008, Field measurements of the base resistance of pipe piles in medium dense sand. Proceedings of the Second BGA International Conference on Offshore Foundations, ICOF 2008, Dundee.
[14] Paik, K.H. and Salgado, R., 2003, Determination of bearing capacity of open-ended piles in sand. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE,* **129**(1), 46–57.
[15] Igoe, D., Doherty, P. and Gavin, K.G., 2010, The development and testing of an instrumented open-ended model pile. *ASTM Geotechnical Testing Journal,* **33**(1), 1–11.
[16] Igoe, D.J.P., Gavin, K.G. and O'Kelly, B.C., 2011, Shaft capacity of open-ended piles in sand. *ASCE Journal of Geotechnical and Geoenvironmental Engineering,* **137**(10), 903–913.
[17] Fellenius, B.H., 2002, Determining the resistance distribution in piles. *Geotechnical News Magazine,* **20**(2), 35–38.
[18] Jardine, R.J. and Standing, J.R., 2000, Pile load testing performed for HSE cyclic loading study at Dunkirk, France, *Report OTO 2000 007* (London: Health and Safety Executive).
[19] Jardine, R., Chow, F., Overy, R. and Standing, J. (Eds.), 2005, *ICP Design Methods for Driven Piles in Sands and Clays* (London: Thomas Telford).
[20] Lehane, B.M., Scheider, J.A. and Xu, X., 2005, CPT based design of driven piles in sand for offshore structures, *Internal report No. GEO:05345* (Perth: University of Western Australia).