Field and numerical study of the lateral response of rigid piles in sand

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Abstract

Large diameter, short monopiles are the preferred foundation type for offshore wind turbines. These piles demonstrate a rigid response with significant rotation of the pile base under ultimate lateral load. As traditional empirical p-y curves used in lateral loaded pile analysis have been derived from tests on small diameter onshore piles, there is some doubt about their applicability to rigid monopiles, particularly in view of the significant difference in the response of the pile base compared with a typical onshore pile. To address this issue, this paper reports on a unique series of field tests using instrumented driven pipe piles, complimented by numerical analysis, which was performed to examine explicitly the contribution of the pile base to the response under lateral load. The field tests, which included tests on pipe piles that had the sand plug removed to below pile tip level prior to testing, confirmed that the influence of the base on the lateral response for the tested 273- and 457-mm-diameter piles was negligible. Numerical analyses that were calibrated using the field test data showed that the contribution of the base to the lateral capacity of a monopile with a diameter as large as 10 m is negligible. Results also indicated that p-y curves are not affected by the length to diameter ratio and can be used to predict the response of monopiles in sand.

Keywords Base effect • Field tests • Finite element modelling • Monopiles • p-y curve

1 Introduction

Pile foundations have been widely used for onshore and offshore infrastructure when high horizontal loads are present. Piles with a diameter (D) less than 2 m and length to diameter ratio (L/D) larger than 20 are generally employed in jacket structures used by the oil and gas industry and resist the applied large overturning moments in a push–pull action. The offshore wind industry prefers use of a single or monopile to resist the applied moments and shear through bending when axial loads are generally relatively small. The most efficient design is usually one involving a large diameter pile (typically with $D > 4$ m) with $L/D < 8$. As the construction of wind farms moves into deeper water, monopiles with diameters as large as 10 m to support 8–10 MW turbines are currently under consideration [4, 37].

The most popular method employed for the analysis of laterally loaded piles is the p-y method [5, 12, 30, 34]. In this method, the pile is modelled as a series of beam elements and the soil is represented by non-interacting, non-linear springs distributed along the pile length [30]. While the p-y method has been used successfully for the design of
small diameter piles for decades, the suitability of the approach for large diameter monopiles has been questioned by many workers both numerically [21, 33, 36] and experimentally [1, 9, 15, 40]. Monopiles experience rigid rotation under lateral load [18, 23] with a consequence that large movement is produced at the pile tip, accompanied by a base shear force and base moment [24]. Lam and Martin [17] and Ashour and Helal [3] argue that correct modelling necessitates inclusion of resistance provided by the base shear, base moment and the shaft friction induced distributed moment in addition to the $p$-$y$ curves. Byrne et al. [8] observed a limited contribution of pile base based on numerical simulation on a 10-m-diameter and 20-m-long pile and proposed a new design method for large diameter piles to incorporate all of these components based on a series of field tests on large diameter piles [6, 28, 31, 35]. However, no experiments have been carried out to quantify explicitly the influence of the pile base on the lateral response.

This paper therefore presents results from a unique series of field tests using instrumented driven pipe piles to obtain definitive information regarding the influence of the base on the lateral response. The experimental programme included tests on piles that had their sand plugs removed prior to testing. Numerical analyses are used to generalize the observations made.

2 Test description

2.1 Site conditions

The field tests reported in this study were carried out at the University of Western Australia (UWA) Shenton Park test site. The ground conditions at the test site have been investigated comprehensively using a range of in situ and laboratory tests [20, 32]. Various parts of the site have been used in the past for a variety of foundation and retaining wall experiments, e.g. Schneider [32], Lehane et al. [19], Li and Lehane [23], Lim and Lehane [25] and Anusic et al. [2]. The soil comprises aeolian siliceous sand with minor carbonate content (< 5%) and overlies weakly cemented limestone at depths between 5 and 10 m. The sand is sub-angular to sub-rounded with a mean effective particle size $d_{50}$ of 0.47 mm and a uniformity coefficient of 2.2. The maximum and minimum void ratios are 0.81 and 0.45, respectively. The water table is located within the limestone at depth, and the degree of saturation ($S_w$) of the sand is 15 ± 2%. Sand replacement tests at 2 m depth show that the average bulk density is 1670 ± 25 kg/m$^3$ [2].

The upper 0.5 m of the profile was excavated in the area of lateral load tests to minimize variability in near surface soil conditions controlling the lateral pile response. Two cone penetration tests (CPTs) were conducted within the excavated pit and, as shown in Fig. 1, indicated closely comparable cone tip resistance ($q_c$). After an initial development length of about 0.27 m (7.5 cone diameters), the mean $q_c$ increases from 2.5 to 9 MPa at 3 m depth. The relative density ($D_r$) is estimated using the following equation employed by Xu [39]:

$$D_r = 0.64 \ln \left( \frac{q_c}{\sigma'_v P_a} \right)^{0.5}$$

(1)

where $\sigma'_v$ is the vertical effective stress and $P_a$ is atmospheric pressure (= 100 kPa). Equation 1 is seen to match the $q_c$ profiles well using a uniform $D_r$ value of 64%.

2.2 Pile details and test programme

Six static lateral pile load tests were performed with details of each test pile provided in Table 1. Tests were conducted on 273- and 457-mm-diameter pipe piles with embedded lengths ($L$) of 1.0 and 1.5 m employed for each diameter. The wall thickness ($t$) of all piles was 6.35 mm and the pile

![Fig. 1 Profile of cone tip resistance](image)

Table 1 Summary of the field test programme

| No. | $D$ (m) | $L$ (m) | $L/D$ | $t$ (mm) | $e$ (m) | Soil plug |
|-----|---------|---------|-------|---------|-------|-----------|
| S1* | 0.273   | 1.00    | 3.66  | 6.35    | 0.33  | Removed   |
| S2  | 0.273   | 1.00    | 3.66  | 6.35    | 0.34  | Removed   |
| S3* | 0.273   | 1.50    | 5.49  | 6.35    | 0.35  |           |
| L1* | 0.457   | 1.00    | 2.19  | 6.35    | 0.34  | Removed   |
| L2  | 0.457   | 1.00    | 2.19  | 6.35    | 0.35  |           |
| L3* | 0.457   | 1.50    | 3.28  | 6.35    | 0.34  |           |

*Piles instrumented with strain gauges
length to diameter ratios ranged from 2.2 to 5.5. Ten pairs of electric resistance strain gauges arranged in half bridge configurations were located on the outside wall of four of the test piles (see Table 1) to measure bending strains. The gauges were protected from damage during installation using an epoxy resin.

The piles were driven to their target depths using an air hammer. This driving method induced a fully coring mode installation with the sand plug extending to ground level (as is typical of full-scale piles). This plug was removed carefully in the two un-instrumented test piles using a hand auger to a distance of at least 100 mm below their tips (see Table 1). This operation ensured that the corresponding base shear and moment during the subsequent lateral load tests on these piles was zero.

2.3 Test setup

A typical setup employed for the lateral load tests is shown in Fig. 2, which involved pushing pairs of piles apart using a hydraulic jack. Wooden blocks carved to the same curvature of the piles were used as loading platens. A load cell was placed in series with the jack and a steel tube, all of which were supported on a wooden frame to facilitate accurate alignment before testing. The lateral load was applied at an elevation \( (e) \) of 0.34 ± 0.01 m above ground level. Although this loading eccentricity is much smaller than that of a typical offshore wind turbine \((e/L\) in the range of 1–2), numerical and experimental studies in Wang et al. [38] and Klinkvort and Hededal [16] indicate little influence of the loading eccentricity on rotation centre position (typically at a depth of 0.75\( L \)) and the pile-soil interaction. Linear variable differential transformers (LVDTs) and string potentiometer displacement sensors, supported on a reference frame, were used to measure the lateral pile deflections at two different heights above ground level.

3 Interpretation of field tests

3.1 Load deflection response at ground surface

The measured variations of applied lateral load with the interpreted deflection at ground level observed in the six test piles are presented in Fig. 3. All piles are seen to respond in a typical ductile manner with higher stiffness and capacity for longer and larger diameter piles. Displacements to generate ultimate capacities for the 1-m-long piles are between 10 and 15\% of the pile diameter, while the ultimate capacity for 1.5-m-long piles required larger displacements and was not fully developed in the tests (due to a limited stroke range of the hydraulic jacking system and insufficient available reaction from the adjacent pile).
The strain gauges on all piles remained within the elastic limit, consistent with a ‘rigid’ or ‘short pile’ response. Gapping was observed on the active side of all piles after large lateral displacements. Such gapping, which has also observed in field tests reported by McAdam et al. [28] and Li et al. [24], arose at this site due to very light suctions in the partially saturated sand ($S_r = 15\%$). Lehane et al. [20] show that the in situ suction has a negligible effect on the sand’s strength and stiffness at $S_r [10\%$.

The load–deflection curves for piles with and without the sand plug removed are included in Fig. 3. It is evident that the respective responses are virtually identical. A slightly larger initial stiffness for the 273 mm pile and slightly softer initial stiffness for the 457 mm pile are observed when the plugs are absent. These minor differences can be attributed to natural variability in the sand at the site and to standard experimental errors, and it can be concluded that the contribution of base shear and moment to the responses of these piles is negligible.

3.2 Pile and soil response at depth

The bending moment ($M$) profiles measured after application of a selection of lateral load increments are presented in Fig. 4 for the four test piles with their plugs intact. It is evident that, as the load level increases, there is a progressive increase in the maximum moment and a very slight increase in the depth at which this maximum moment occurs. The maximum applied moment for the 1.0-m- and 1.5-m-diameter piles is recorded at a depth of 40% of the pile length below the ground surface for the 273-mm-diameter pile and at a depth of 30% of the pile length for the 457-mm-diameter piles.

As indicated in Fig. 4c and d, four of the gauge levels on the 457-mm-diameter piles (two on each pile) did not provide reliable strain outputs and therefore, $p$-$y$ load transfer curves were only calculated using the strain gauge data on the 273-mm-diameter piles. These curves were derived by first obtaining a best-fit to the moment profiles at given load increments using a six-order polynomial function, i.e.

$$M = a_1 z^6 + a_2 z^5 + a_3 z^4 + a_4 z^3 + a_5 z^2 + a_6 z + a_7$$

(2)

The seven coefficients ($a_1$–$a_7$) in the polynomial are determined by fitting the measured bending moment with a least-squares curve fitting technique and imposing constraints comprising (i) A known bending moment and shear force at the ground surface ($z = 0$) and (ii) Zero soil resistance at the ground surface. Strain gauges were located at approximately 100 and 250 mm above the pile tips and, as seen in Fig. 4, these show extrapolated moments at the tip tending to be zero. This constraint together with an assumption of zero shear load at the base (in line with the observations made in Fig. 3 showing no influence of the plug) was also used in the curve fitting exercise.

Lateral net soil reactions per unit length ($p$) are then obtained by double differentiation of Eq. (2), while the deflection ($y$) can be derived by double integration of this equation and dividing by the pile’s flexural rigidity, EI. The integration process involved two additional constraints to determine the two integration constants namely (i) The deflection measured using the string pot transducers located about 500 mm above the ground surface and (ii) Zero displacement at the location of zero net resistance (i.e. at point of pile rotation [14]). LVDTs located about 100 mm above ground level had been put in place to measure the pile head rotation. However, these indicated an unrealistic depth of pile rotation due to flexure of the timber support down-stand on which the LVDTs were mounted (see
Fig. 4 Measured bending moment profile of pile: a $D = 0.273\ m$, $L = 1.0\ m$; b $D = 0.273\ m$, $L = 1.5\ m$; c $D = 0.457\ m$, $L = 1.0\ m$; d $D = 0.457\ m$, $L = 1.5\ m$
Displacements at ground level were consequently determined from the string pots located 500 mm above ground level assuming rigid pile rotation about the location of zero net pressure (following the approach of Haiderali and Madabhushi [14] and Li et al. [22, 24]).

The measured and fitted bending moment profile, derived shear force, soil reaction and deflection profiles for the 1.5-m-long, 273-mm-diameter pile are provided in Fig. 5. The six-order polynomial function (Eq. 2) is seen to capture the measured moments for all load increments and be consistent with the zero soil resistance condition at the ground surface and zero shear at the pile base. The derived deflection profile demonstrates a stiff pile response with rigid rotation about a point about 1.12 m (0.74L) below the ground surface. This depth to the point of rotation is consistent with observations of Li et al. [24], Klinkvort and Hededal [16] and others.

Load transfer data determined from the 1.0- to 1.5-m-long, 273-mm-diameter piles are presented in normalized form at three different depths for each pile in Fig. 6, where the net lateral pressure ($P = p/D$) is normalized by the vertical effective stress ($\sigma_v$) and the lateral deflection ($y$) is normalized by the diameter ($D$). It is seen that there is a good agreement at the three depths and for the two pile lengths. In addition, the following equation using the $q_c$ as input is also plotted in Fig. 6 and is seen to provide a good representation of the derived $p-y$ response.

$$P/\sigma_v = 2.7q_c \sqrt{\sigma_v p_a}^{0.7} \tanh\left[5(y/D)^{0.55}\right]$$

The first term of this equation corresponds to the normalized ultimate soil pressure and incorporates the same dependence on $q_c$ raised to the power of 0.7 deduced by Suryasentana and Lehane [34]. Both the experimentally derived $p-y$ curves and those calculated by Eq. 3 were used to input into the program LAP [11], which is web-based software employing the traditional 1D beam element to model the pile and nonlinear, non-interacting spring springs to model the soil [11, 13]. The lateral responses of the test piles predicted using LAP are compared with measured responses in Fig. 7. No soil springs were input at the pile base in the LAP analyses because of the identical responses of the piles with and without sand plugs seen in Fig. 3. As shown in Fig. 7a, the measured load–deflection response of 0.273 m piles is very well predicted by the experimentally derived $p-y$ curves and there is no requirement to include additional resistance from pile base shear and moment. Byrne et al. [8] performed numerical simulations for a large diameter short pile ($D = 10$ m, $L/D = 2$) in sand and also found that the contributions of base moment and base shear force were small but recommended inclusion of additional resistance for shaft friction for accurate predictions. It is of important to note, however, that the $p-y$ curves used in general practice are derived by double differentiation of the measured bending moment profile, where the bending moment at a given level is derived from diametrically opposite strain gauges placed on the tension and compression face to measure the pile curvature (Reese et al. [30]; Cox et al. [10]). As the measured strains are the resultant point values of all the forces acting on the pile (Byrne et al. [7]), the shaft friction contributes to the measured moment (i.e. the tension and compression strains) and is therefore incorporated implicitly in the experimentally derived $p-y$ curves.

Given that Eq. (3) was derived from the bending moment profile on the 273-mm-diameter piles, it is not surprising to see that the nonlinear responses of these piles are well predicted. However, it is also seen that the calculated responses of the 457-mm-diameter piles provide a reasonable (but imperfect) match to measured responses (and a tendency to under-predict capacity at large displacement). Despite the difference, which site variability in $q_c$ alone could account for, the results indicate that the $P-y/D$ relationship given in Eq. (3) is not affected significantly by pile diameter and that a $p-y$ type analysis can provide practical approximate predictions for piles with low $L/D$ ratios.

### 4 Finite element analysis

A series of finite element (FE) analyses was performed to investigate if the observations made in respect of the absence of the influence of base shear and moment on the 273- and 457-mm-diameter piles also apply for larger diameter piles used offshore.

An example of an FE model employed using the Plaxis 3D code [31] is shown in Fig. 8, which was used to simulate the lateral response of the 273-mm-diameter, 1-m-long test pile. Mesh and model dimension sensitivity analyses were carried out first to ensure accurate results that were obtained with minimal boundary effects. The lateral boundaries were constrained in the normal direction, while the base boundary was fully fixed. The soil was modelled using ten-node tetrahedral elements, and the pipe pile was modelled as a cylindrical steel plate with six-node plate elements. The sand-pile shaft interface was assigned a strength reduction factor of 0.7 (Liu et al. [26]). The sand plug within the pile and sand below the pile tip was deactivated to simulate the case of the lateral test which removed the plug before testing. Elastic elements were used locally in the vicinity of the pile tip excavation for numerical stability. In this case, no load transfer to the sand from the base of the annular steel pile was permitted.
Fig. 5 Derived profiles of the 0.273-m-diameter and 1.5-m-long pile: a bending moment; b shear force; c soil reaction; d displacement
The elasto-plastic hardening soil (HS) model available was employed to model the Shenton Park sand and has been used successfully for analysing a variety of boundary value problems in this material due to its ability to capture the stress and strain level dependency of soil stiffness. A detailed description of the model and parameters can be found in Schanz et al. [31]. The model requires specification of peak friction and dilation angles $\varphi$ and $\psi$ and stiffness parameters defined at a reference stress $p_{\text{ref}}$ of $E_{50}^{\text{ref}}$ (secant stiffness in triaxial conditions at 50% mobilised strength) for deviatoric loading; $E_{\text{oned}}^{\text{ref}}$ (the one-dimensional tangent stiffness) for compressive loading; and $E_{\text{ut}}^{\text{ref}}$ and $\nu_{\text{ur}}$ (the triaxial unloading–reloading stiffness and Poisson’s ratio) for unloading–reloading. A power law parameter $m$ is used to describe the variation of stiffness with stress level according to a power function. The parameters selected for the analyses presented here are listed in Table 2 and are consistent with those proposed by Mathew and Lehane [27] for the sand. The sand is modelled as fully dry in keeping with the insensitivity, shown by Lehane et al. [20], of the sand’s stiffness and strength to the small levels of in situ suction at the Shenton Park site.

The measured and computed load–deflection responses for the 1.0-m-long, 273- and 457-mm-diameter piles are presented in Fig. 9. It is seen that the calculated pile responses match the measured pile responses very well over the full displacement range. Importantly, as seen in the field tests, the FE model predicts almost identical responses for the piles with and without sand plugs. Therefore, based on these agreements, it can be inferred that the FE model provides a credible means of extrapolating findings to larger diameter pipe piles.

FE analyses were therefore repeated using the same HS parameters but for piles with diameters of 6, 8 and 10 m and a length of 30 m (corresponding $L/D$ ratios between 3 and 6). A constant loading eccentricity of 10 m was used in all the simulations. The analyses were conducted for piles with and without sand plugs. The wall thickness of both types of piles was 90 mm.

The computed load–deflection responses of the piles are plotted in Fig. 10. It is seen that the load–deflection response for the 6-m-diameter pile is almost completely independent of the contribution of the pile base, i.e. a pile with no sand plug and therefore, no capacity for load transfer to the sand beneath the pile base has the same load–displacement response. Figure 10 indicates slightly higher capacity for the 10-m-diameter pile with a base contribution. However, there is no significant effect of the base on the initial stiffness for any pile diameter up to a lateral ground deflection of 200 mm. The largest relative...
difference of the 10-m-diameter monopile is still less than 5% even for a ground deflection of 1800 mm. This is also consistent with the numerical observation in Byrne et al. [8] where limited contribution of base moment and base shear force was found in sand even for a large diameter short pile with $D = 10$ m and $L/D = 2$.

Figure 11 presents the response of the embedded part of the three different diameter piles, including the profiles of deflection, bending moment, soil resistance and soil pressure at a ground displacement of 10%$D$. As shown in Fig. 11a, all the three prototype monopiles demonstrate a rigid pile response under lateral loading with nonzero displacement at the pile tip. Consistent with the observations in the tests and reported in the literature (e.g. Wang et al. [38]; Klinkvort and Hededal [16]), the position of the rotation centre is at around 0.75$L$ below the ground surface and not affected by the pile diameter. However, despite the reverse displacement and rotation at the pile base, the bending moment profiles in Fig. 11b show that very little moment is generated at the pile base (although this increases somewhat with pile diameter). As shown in the figure, the influence of removing soil plug is mainly on the peak bending moment (around 10% reduction for 10-m-diameter pile). It is believed that this difference is the combined result from the pile base and the shaft friction along the pile.

Figure 11c presents the mobilized soil resistance along the piles: removing the pile base has little influence on lateral pile–soil interaction. In addition, the lateral soil pressure for different diameter piles (as shown in Fig. 11d) is not affected significantly by the pile aspect ratio. This is

![Fig. 8 Mesh of finite element model involving a pile with $D = 0.273$ m and $L = 1.0$ m](image)

![Fig. 9 Comparison between measured and computed load–deflection response](image)

![Fig. 10 Load–deflection response at ground surface for piles with and without soil plug removed](image)

### Table 2 Hardening soil model parameters for Shenton Park sand

| Sym.      | Parameters                           | Value  |
|-----------|--------------------------------------|--------|
| $\gamma$  | Unit weight of soil (kN/m$^3$)       | 16.4   |
| $\rho_{ref}$ | Reference stress for normalized stiffness parameters (kPa) | 100    |
| $E_{50}^{ref}$ | Reference secant stiffness at 50% of the failure load in isotropic triaxial compression (MPa) | 60     |
| $E_{oed}^{ref}$ | Reference tangent stiffness for oedometer loading (MPa) | 60     |
| $E_{ur}^{ref}$ | Reference unload reload stiffness (MPa) | 180    |
| $m$       | Exponent for stress dependent stiffness | 0.5    |
| $c$       | Cohesion intercept (kPa)             | 2      |
| $\varphi$ | Peak friction angle                   | 38     |
| $\psi$    | Dilation angle                       | 6      |
| $v_{ur}$  | Poisson’s ratio for unload–reload conditions | 0.2    |
Fig. 11 Profiles of pile response with and without soil plug removed at a lateral pile displacement of 0.1D at ground level: a pile deflection, b bending moment, c soil resistance, d soil pressure
also consistent with the experimental observation in this study as discussed in Figs. 6 and 7.

Therefore, it is clear that the both experimental and numerical results have proved the limited influence of pile base on the lateral response of piles and that the p–y load transfer method can be applied to piles with low aspect ratios without losing significant accuracy.

5 Conclusions

This study presents results from series of lateral load tests performed in the field and additional numerical analyses with the objective of examining the contribution of the pile base to the response of rigid piles under lateral load. The study has shown that, contrary to expectations of a variety of researchers, the relative contribution of the base to predictions of lateral pile response in sand is small for piles of all diameters. In addition, the p–y load transfer approach, with no additional springs at the pile base, has been shown to be capable of providing reasonable predictions of the lateral response of piles with low aspect ratios.

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