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Lateral response of piles in weak calcareous sandstone

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Abstract

Existing design methods for laterally loaded piles in soft rock have insufficient supporting case history data and consequently cannot be regarded as having a high level of reliability. This paper addresses the shortage of experimental data by describing the results from four instrumented lateral load tests on drilled and grouted piles installed in weak calcareous sandstone. The derived lateral load transfer (p - y) curves are first compared with existing approaches for weak rock and cemented soils. The paper then examines the potential of using CPT data directly for the design analysis of laterally loaded piles in weak rock. It is shown that equivalent elastic moduli of the rock controlling values of q_c at the test site are very similar to those controlling lateral pile response. This finding is used to formulate a simple bi-linear p - y approximation of the lateral response that requires q_c data and an estimate of the rock's effective stress strength parameters. It is shown that such an approach is likely to be sufficient for many practical purposes if the moment capacity of the pile section is incorporated in the analysis.

Keywords:

Pile; Lateral loading; Pile; Weak rock; Field Test

Introduction

Documented experiments investigating the response to lateral load of piles in rock are extremely rare. Given the shortage of available guidance, Reese (1997) sought to provide recommendations for practitioners but could only uncover the lateral pile tests reported by Nyman (1980) and Speer (unpublished 1992) to support his proposed load transfer p - y formulation. Reese et al. (2010) proposed the same formulation for rock without any additional supporting case history data and suggest that it should only be used with caution given the meager amount of data on which it is based. Moreover, the formulation assumes that the rock can be sampled to obtain a representative measure of the unconfined compressive strength and that cores exist from which an estimate of the rock quality designation (RQD) or fracture spacing can be established. The approach is therefore not suitable for very weak or highly fractured rocks for which core recovery is often very poor.

While the weak rock approach of Reese (1997) has been incorporated into many of the commercially available computer codes and has widespread use, other formulations have also been suggested. These include the continuum approach suitable for strong rocks proposed by Carter and Kulhawy (1992), and the load transfer approaches reported by Abbs (1983), Erbrich (2004) and Reese et al. (2010). Abbs (1983) attempted to capture the brittle nature of weak carbonate rocks, assuming that the pre-peak response could be simulated using a stiff clay model and the ultimate (residual) lateral resistance approached that of a loose sand. Erbrich (2004) developed another p - y approach for carbonate rocks which specifically allowed for the progressive ‘chipping’ of rock wedges close to the surface as lateral loads increased. Reese and Van Impe (2001) developed the method proposed by Evans and Duncan (1982) for cemented soils using a load transfer curve similar to that of sand but adjusted for the additional lateral resistance arising from cementation (the c' component of strength).

All existing methods for analysing laterally loaded piles in cemented soils and weak rock

have insufficient supporting case history data and consequently cannot be regarded as having a high level of reliability. This paper addresses the shortage of experimental data by describing the results from lateral load tests conducted on four instrumented drilled and grouted piles installed in weak calcareous sandstone. A drilled and grouted pile, which is essentially equivalent to a drilled shaft or bored pile, is the preferred pile type in many offshore environments where the carbonate content of the soil is high.

The derivation of p - y curves from the test pile responses is presented and shown to require careful interpretation of the structural response of the piles. These curves are first compared with those estimated using existing approaches and with the available laboratory data. As the weak rock at the test site could not be cored satisfactorily (although block samples were attained), attention is focused on establishing a general basis for how p - y curves for cemented soils and weak rocks can be related to the Cone Penetration Test (CPT) end resistance (q_c). A simple bi-linear p - y approximation of the lateral response that requires q_c data and an estimate of the rock's effective stress strength parameters is subsequently proposed.

Site and Soil Characterization

The site of the lateral pile tests was a limestone quarry in Pinjar, which is about 25 km north of Perth, Western Australia. The rock is mined as a "medium grade limestone" and is part of the Tamala Limestone formation. This formation comprises calcarenite and/or calcareous cemented, fine to very coarse, well graded sand that originated from wind-blown sediment and accumulated as coastal sand dunes during the Pleistocene era, and later lithified when the lime content dissolved to cement the grains together (Playford et al. 1976).

A test pit excavated at the site indicated that, as seen in Fig. 1, the deposit is characterized by distinct large-scale horizontal bedding with cemented band thicknesses varying from about 5 to 40 mm (typically about 25 mm) interlayered with discontinuous very poorly cemented

bands 2 to 10 mm in thickness. The carbonate and silica content of sampled materials were typically $40 \pm 25\%$ and $60 \pm 25\%$ respectively, indicating that the general deposit classifies as calcareous sandstone according to the Clark and Walker (1977) classification system. The in-situ water content is typically 4% and the degree of saturation is about 20%. In-situ sand cone replacement tests indicated a unit weight of $15.8 \pm 0.2 \text{ kN/m}^3$. The average grain size is 0.3 to 0.4 mm (measured after grinding the cemented materials using a mortar and pestle). The average point load indices ($I_{s,50}$) of the cemented materials tested in the vertical and horizontal direction were 0.26 MPa and 0.15 MPa respectively, but measurements of these indices showed a relatively large coefficient of variation of 0.7.

A total of six cone penetration tests (CPTs) and eight seismic cone penetration tests (SCPTs) were performed within 8 m of the test piles. The range and average of the end resistances (q_c) and friction ratios recorded are shown on Fig. 2. Friction ratios were consistently within the range 0.4% to 0.6% while q_c values between 1 and 2 m depth (i.e., approximately three pile diameters in the zone most critical for the lateral load tests) were typically $35 \pm 15 \text{ MPa}$. The traces show no clear evidence of the bedded and cemented nature of the deposit observed in the test pits (as seen in Fig. 1) and, without such knowledge, standard CPT soil behavior type charts (e.g., Robertson 2009) suggest the material is an overconsolidated very dense sand. The SCPT shear wave velocities (V_s) do, however, reveal the cemented nature of the deposit and are typically in the range 520 m/s to 1450 m/s with an average value of 950 m/s. These V_s values correspond to small strain shear moduli (G_0) shown on Fig. 2 with typical values of $1.4 \pm 0.7 \text{ GPa}$ in the area of interest for the lateral pile tests.

Direct shear tests were performed at a drained rate of displacement on ‘chunks’ or pieces of the material in a $60 \times 60 \text{ mm}$ shear box. These pieces, where were typically about 20-30 mm high and 40 to 50 mm long and wide, were fixed to the internal wall of the shear box with plaster using the technique described by Goodman (1989), and others. Strips of Teflon were

positioned at the interface between the plaster in the upper and lower boxes. Other shear box tests were performed on loose samples of reconstituted sand (achieved by grinding the cemented materials). The area of the sample on shearing plane was determined on the completion of each test and was typically about 60% of the full plan area of the shear box.

The peak shear stresses recorded in the direct shear tests are summarized on Fig. 3. The reconstituted samples indicated an ultimate (or critical state) friction angle (ϕ'_{cs}) of about 40° ; this angle is typical of angular coarse grained deposits with a high percentage of carbonates (e.g. Lehane et al. 2012). The intact samples show distinct evidence of a c' component of strength, where c' lies between 180 kPa and 300 kPa. All of these intact samples display a marked reduction in the shear stress that could be sustained following attainment of the peak values at a relative displacement of about 2 mm. Additional tests were performed under fully saturated conditions on intact samples (see Fig. 3) and these revealed comparable c' values to the other rock samples (tested with no water in the shear box bath) indicating that observed c' is not related to suction in the intact materials.

Drained triaxial tests on carefully carved, vertically orientated, (near) cylindrical samples of the deposit were also performed after saturation and isotropic consolidation to consolidation stresses (σ'_3) of 50, 100 and 200 kPa. The deviator stress (q) -axial strain (ϵ_a) relationships recorded are plotted on Fig. 4a. It is evident that, for each of the three samples, deviator stresses develop peak values of between about 1.5 and 1.75 MPa at axial strains of $1 \pm 0.5\%$ but these reduce significantly as axial straining progresses and cementation bonds are broken. The Young's moduli, defined at 50% of the peak strength, vary between 230 MPa and 320 MPa.

The stresses recorded at peak conditions are plotted in terms of the stress invariants t_f and s'_f on Fig. 4b (where t_f and s'_f are as defined on the figure and σ'_{1f} and σ'_{3f} are the major and minor principal effective stresses at peak), where they are compared with the strength envelope deduced from the shear box tests on Fig. 3. It is seen that the results for the (larger) triaxial

specimens are in general agreement with shear box data but plot at the lower end of this envelope range.

If the Mohr-Coulomb failure criterion is assumed, the unconfined compressive strength (q_u) of the triaxial samples can be estimated from:

$$[1] \quad q_u = 2c' \tan\left(45 + \frac{\phi'}{2}\right)$$

Assuming c' and ϕ' values of 180 kPa and 40° , as indicated in Fig. 4b, Eq. [1] implies unconfined compressive strength for vertical loading of between 1 and 1.7 MPa. This range is consistent with the average value of 1.1 MPa and standard deviation of 0.8 MPa indicated in 13 unconfined compressive strength tests on vertically orientated, carved prismatic block samples (Guo 2015). The corresponding mean ratio of q_u to the point load index $I_{s,50}$ value (with an average value of 0.26 MPa) is approximately 4 to 5 and a little less than a (typical) value of 7, quoted by Look (2014), and others, for rocks of this nature.

Experimental Details

Two 340 mm diameter and two 450 mm diameter, 5 m long, drilled and grouted test piles were constructed at Pinjar, using the continuous flight auger method. Full length steel pipes with a yield stress of 350 MPa were inserted in the grouted holes after removal of the auger. Steel pipes with a diameter of 219 mm and wall thickness of 4.8 mm were installed in the 340 mm piles while 356 mm diameter pipes with a wall thickness 6.4 mm were used for the 450 mm diameter piles. Each test pile was instrumented with 6 levels of strain gauges located on the tension and compression outer faces of the steel pipes. A plastic inclinometer access tube was inserted along the centerline of each pile.

The testing details are summarized in Table 1 and illustrated on Fig. 5. These involved jacking apart the two 340mm diameter piles followed by jacking apart the two 450mm

diameter piles. The respective locations of the 340mm piles (denoted, P340A and P340B) and 450mm piles (denoted, P450A and P450B) are shown in Fig. 5a. A 1 m deep pit, measuring 5m $\times$ 8 m, was excavated in the area of the test piles to facilitate application of the lateral loads and to avoid the influence on the test results of the upper (more variable) 1 m of the deposit. CPTs performed after lateral extension of the pit when the pile tests were completed indicated closely comparable q_c values to those shown in Fig. 2, apart from in the upper 0.1 to 0.15m before the cone reached a deep penetration mode.

The testing program initially involved application of a relatively small lateral load at 0.1 m below the head of each pile and about 0.9 m above the pit level, i.e., load eccentricity $e = 0.9$ m. The primary purpose of this phase of the testing was to provide on-site calibration of the moment-strain relationship at the strain gauge levels located above the base of the pit (i.e. at negative z values; see Fig. 5b). The initial lateral load applied to P340A and P340B was 40 kN, which on removal gave permanent lateral displacements of about 1.4 mm at the pile head and less than 0.2mm at ground level. The initial load applied to P450A and P450B was 140 kN and this led to permanent displacements of similar magnitude. Large scale cracks appeared in the grout annulus outside of the steel pipes during this testing phase indicating that this annulus made a negligible contribution to the piles' bending stiffness.

The outer grout was removed from the exposed sections of the piles and the lateral load eccentricity was reduced to approximately 0.32 m for the 340mm piles and 0.20 m for the 450mm piles. The flexural rigidities of the above ground sections of the 340mm and 450mm piles at this stage of loading was assessed from the preceding calibration exercise to be 4300 kNm² and 25300 kNm² respectively. Loading was applied incrementally with 10 minute waiting periods between each 10 kN load increment for the 340mm piles and 20 kN load increment for the 450mm piles.

Experimental Results

The variations of lateral pile head load H with lateral displacement, y_h (recorded at the level of applied load) when the point of load application was 0.26 ± 0.06 m above ground level are shown on Fig. 6 (noting that the maximum lateral load that could be applied was limited by the lower capacity pile in each pair). It is evident that piles P340B and P450B (both on the same side of the pit; see Fig. 5a) had lower capacities than their counterparts and begin to exhibit significant yielding once y_h reached between about 4 mm and 7 mm. Extrapolation of the H - y_h data for P450A suggests that this pile had an ultimate capacity about 50 kN greater than the ultimate value of 480 kN sustained by P450B. The observed capacity of P340B of 155 kN is noticeably less than that of P340A which only experienced a permanent displacement of 2.2 mm on unloading from 155 kN.

Displacement profiles derived from inclinometer measurements obtained during testing of P340A and P340B are shown on Fig. 7. Unfortunately no such data were obtained for the 450mm diameter piles as the inclinometer access tube melted due to high curing temperatures (on discovery, the melting point of the PVC tubes was found to be only 85°C). The data on Fig. 7 indicate that virtually no pile movement occurs below a depth of 1 m (or 3 pile diameters) and that the rock properties below this depth had no significant impact on the pile performance. The displacement profiles also indicate that the ultimate pile capacity was controlled by the structural capacity of the pile section and not by a geotechnical failure (for which rotation would take place close to the pile toe).

The derivation of net lateral stresses acting on the test piles requires accurate estimation of the bending moments in the piles and these moments were assessed using the moment-curvature relationship derived using the on-site calibration exercise referred to above. The results from this exercise are presented on Fig. 8 which plots the curvatures (κ) measured at each (above ground) strain gauge location for given applied moments (M); the curvature at

any given level was determined as the difference in strain recorded by the gauges on the compression and tension faces of the respective steel pipes divided by the pipe diameter; M was simply derived as the product of the applied load and the distance of this load from the strain gauge level. Reasonable agreement in the inferred M - κ relationships was obtained, noting that some of the gauges would lead to the inference of a slightly lower M - κ slope as they were marginally below ground level. These on-site calibrations provided the base data from which results could be extrapolated.

The computer program Oasys ADSEC (Oasys 2013) was used to extrapolate the measured M - κ relationships to large curvature levels and ultimate conditions. ADSEC is a program for determination of the bending characteristics of structural cross-sections and requires specification of the non-linear stress-strain properties (in tension and compression) of the materials within any particular cross-section; the geometries of the 340mm and 450mm cross-sections are provided in Table 1. The (un-factored) parabolic compression-strain curve with the form provided in the European structural engineering standard (EuroCode 2) was employed for the grout and the corresponding stress strain curve for the Grade 350 MPa steel tubes was used. Best fit average secant grout Young's moduli matching the on-site measurement of the moment-curvature relationships were lower than anticipated and approximately 4 GPa for both the 340mm and 450mm diameter piles. The compressive and tensile strengths adopted for the grout were 12 MPa and 1.5 MPa respectively and these are seen on Fig. 8 to provide a reasonable representation of the sections' bending capacities; the onset of significant yielding was calculated to occur at respective moments (M_y) of 67 kNm and 228 kNm for the 340mm and 450mm diameter piles.

The bending moments induced at the strain gauge levels of each test pile were derived using the M - κ relationships on Fig. 8. Best estimate moment profiles with depth (z) were then fitted to these data using the following relationship form (where a_i , $i = 0$ to n are empirical fitting

coefficients):

$$[2] \quad M = \left(1 - \frac{1}{e^{a_0 z} + 1}\right) \sum_{i=1}^n a_i z^{i-1}$$

These best-fit moment profiles are shown together with the estimated M data on Fig. 9. The net stress per meter run (p) and net pressure ($P = p/D$) on each pile at any depth, z , were then obtained by double differentiation of Eq. [2]. The displacements (y) at the corresponding depth and level of applied load were determined from Fig. 7 for the 340mm diameter piles. No displacement data were available for the 450mm piles but the moment profiles together with the H - δ_h trend for these piles could be used to obtain a reasonable estimate of the ultimate lateral net pressures (P_u). Displacements could not be estimated with accuracy from double integration of Eq. [2] due to the non-linear pile flexural rigidity coupled with the imprecise nature of the M profiles.

The net pressure (P) - displacement (y) curves that could be deduced with confidence (i.e. when lateral displacements measurements were deemed sufficiently accurate) for the 340mm diameter piles are shown on Fig. 10. As may be expected from the load-displacement response shown on Fig. 6, the P - y curves for P340A are stiffer than those of P340B. Peak (or ultimate) pressures (P_u) developed on both piles at a displacement of about $0.5 \pm 0.1\%$ of the pile diameter (D) and generally ranged from 1.3 MPa to 2.5 MPa. It is also evident that the P - y responses show inconsistent trends with the relative depth (z/D) and no clear tendency for ultimate/maximum pressures to increase with z/D over the (limited) range investigated. More brittleness is in evidence for P340A (which developed higher pressures) but, in general, the level of brittleness is relatively mild compared to that observed in the triaxial tests (Fig. 4).

Discussion

Ultimate lateral pressures

The net stresses for the 450mm diameter piles were derived from the pile bending moments but extrapolation of these to peak lateral pressures was more approximate because of the absence of inclinometer data; the lateral movements at depth used for this extrapolation were assessed using the pile head displacement data (see Fig. 6). These piles displayed significant signs of yielding when the applied lateral load was 480 kN; the maximum moment in the piles at this stage was about 250 kNm and occurred at a depth between 0.4m and 0.5 m (equivalent to one diameter).

The estimated maximum pressures (P_u) for the 340mm and 450mm piles are plotted on Fig. 11 and have a mean and standard deviation of 2 MPa and 0.7 MPa respectively. It is evident that P_u does not vary systematically with z/D . The coefficient of variation of P_u is broadly compatible with that of the CPT q_c data shown on Fig. 2.

These P_u values are compared on the same figure with predictions obtained using the Reese (1997) formulation for weak rock and the Brinch-Hansen (1961) and Reese and Van Impe (2001) formulations for cemented (c - ϕ') soils. The calculations assumed $\phi'=40^\circ$ and adopted the best fit c' value of 180 kPa seen in the triaxial tests (Fig. 4), which is equivalent to the lowerbound c' value obtained in shear box tests (Fig. 3).

Brinch-Hansen (1961) predicts P_u as a function of c' and ϕ' using the following expression:

$$[3] \quad P_u = K_q \sigma'_v + K_c c'$$

Where K_q and K_c are passive resistance coefficients and prescribed functions of c' , ϕ' and normalised depths (z/D) and can be found in Brinch-Hansen (1961) and textbooks such as

Tomlinson and Woodward (2007). The Reese (1997) formulation relates P_u with the unconfined compressive strength (q_u) as:

$$[4a] \quad P_u = \alpha q_u \left(1 + 1.4 \frac{z}{D} \right); \quad z/D \leq 3$$

$$[4b] \quad P_u = 5.2 \alpha q_u; \quad z/D > 3$$

Where α = strength reduction factor = 1.0 (assumed in this study) and, in the absence of q_u data, q_u can be approximated from c' and ϕ' using Eq. [1].

Using a concept initially proposed by Evans and Duncan (1982), Reese and Van Impe (2001) represent the ultimate resistance as the sum of the cementation/ cohesive component (P_{uc}) and frictional component ($P_{u\phi}$), where the latter is multiplied by an empirical adjustment factor (A), i.e.

$$[5] \quad P_u = A P_{u\phi} + P_{uc}$$

The comparisons on Fig. 10 reveal some significant differences between estimates made using Eqs. [3], [4] and [5]. The Brinch Hansen (1961) predictions for ultimate pressures are evidently non-conservative for the Pinjar material whereas those obtained using the Reese and Van Impe (2001) formulation are significant under-estimates at shallow depths. The Reese (1997) formulation with $\alpha = 1$ gives pressures that are in better agreement but are at the lower end of the range of measured resistances. All methods predict a stronger dependence on z/D than indicated by the measurements.

P-y response

The $P-y$ data from both 340mm diameter test piles are combined in Fig. 12, which shows bi-linear (linear-elastic perfectly plastic) approximations to the curves for average, lowerbound (LB) and upperbound (UB) cases. It is assumed for the purposes of this plot that

these respective cases correspond to the in-situ mean, lowerbound and upperbound CPT q_c values of 35 MPa, 20 MPa and 50 MPa; see Fig. 2. Following Poulos (1971), the ratio of P to y for a laterally load pile is approximately $0.8E_s/D$, where E_s is the equivalent linear elastic modulus (E_s) of the soil. E_s moduli determined in this way are seen on Fig. 12 to be approximately 12 times the corresponding q_c value for each of the average, lowerbound and upperbound cases. Maximum pressures (P_u) are close to being developed at a lateral displacement of 1.5 to 2 mm or y/D value about 0.5% and these are approximately equal to 0.045 times the CPT q_c value at all z/D values considered ($z/D \leq 1.5$).

The mean measured P - y trend is shown in normalized form on Fig. 13, where each P value is normalized by the maximum pressure recorded at the depth (P_u). This trend is compared with the bounds of P/P_u vs y/D proposed for weak rock by Reese (1997) and with the recommendation for a c' - ϕ' soil given by Reese and Van Impe (2001), assuming $c'=180$ kPa and $\phi'=40^\circ$; see Fig. 4). It is evident that the Reese and Van Impe formulation provides a closer match to the average field curve, despite P_u values for this method being under-predicted (see Fig. 11). The normalized curves given by the weak rock model are well above those observed in the field over the early stages of loading.

Prediction of pile response using a CPT-based bi-linear p - y curve

The simple bi-linear approximation to the p - y curves indicated on Fig. 12, assuming $E_s/q_c=12$ and $P_u/q_c=0.045$, is now examined to assess the ability of this simplification to provide reasonable predictions of the lateral response of the Pinjar piles, including the 450mm diameter piles for which p - y data were not obtained. A program referred to as LAP (Doherty 2014) was used for the calculations. This program represents the pile as a series of beam elements and the soil as a series of non-linear, non-interacting p - y springs located between each

beam element; the program is identical to many commercially available laterally loaded pile programs and its accuracy was verified by a parallel series of calculations for the Pinjar piles performed by ALP (Oasys 2013), which is a very similar program but does not incorporate pile yielding.

An important element of the observed response was structural yielding of the Pinjar piles. The analyses assigned a simple bi-linear moment-curvature ($M-\kappa$) approximation to the curves on Fig. 8, with yield moments (M_y) assumed to be those corresponding to the onset of significant non-linearity in the $M-\kappa$ relationship. The calculated pile head load-displacement ($H-y_h$) responses, which were performed for the mean, upperbound and lowerbound q_c values are compared with the measured data on Fig. 14.

It is seen that the upper and lower bound calculations bracket the measured $H-y_h$ data for all piles, apart from P450A. Calculations using the mean $p-y$ response (corresponding to $q_c=35$ MPa) provide matches to the field behavior which is sufficient for most practical purposes. A more general correlation between $p-y$ responses and CPT data is explored in the following.

Correlation with CPT q_c

The assessment of in-situ stiffness and strength from weak rocks such as those at Pinjar is problematic because of the immense difficulties obtaining undisturbed core samples. Given the tendency for the $p-y$ responses to reflect CPT q_c values and their variability (Fig. 12), a spherical cavity expansion analogy for CPT penetration is now examined to further investigate the potential of a CPT based method to predict lateral pile response in weak rocks.

The spherical expansion approach to predict q_c has found widespread application and is described by Yu and Houlsby (1991) and applied in sands by Suryasentana and Lehane (2014) and in clays by Xu and Lehane (2008). The drained spherical cavity expansion limit pressure

(p_{lim}) is related to q_c (for a cone with a 60° apex angle) by the following expression proposed by Randolph et al. (1994):

$$[6] \quad q_c = p_{lim} (1 + \tan \varphi' \tan 60^\circ)$$

Values of p_{lim} can be estimated using the Yu and Houlsby (1991) solution for cavity expansion in a linear elastic Mohr Coulomb soil, for which the controlling parameters are the c' and φ' values, the dilation angle (ψ), the stress level (or depth) and the equivalent linear elastic modulus (E_s) for the material. CPT q_c resistances using these p_{lim} values and Equation (6) were calculated assuming a fixed friction angle (φ') of 40° (a typical value for carbonate rich soils/rocks) and a dilation angle of zero. For these calculations, combinations of c' and E_s values were deduced that gave calculated q_c values equal to either the lowerbound, mean or upperbound q_c resistances of 20 MPa, 35 MPa and 50 MPa measured at Pinjar. These analyses held the vertical effective stress constant at 10 kPa, which was representative of the area of interest for the Pinjar piles. Other analyses indicated that the derived c' values, which were generally in the observed ranged of 50 kPa to 300 kPa, did not exhibit high sensitivity to the assumed stress level or to the assumption of a zero dilation angle because of the relatively high values of c' involved.

Ratios of q_c/c' determined in this way are plotted on Fig. 15a against the E_s/q_c ratios required to give the three target CPT q_c resistances. It is evident that, for the range of E_s/q_c values shown by the 340mm diameter piles at Pinjar (deduced from Fig. 12), q_c/c' is typically about 160 and hence c' values are approximately 125 kPa, 220 kPa and 310 kPa for the respective lowerbound, mean and upperbound q_c resistances of 20 MPa, 35 MPa and 50 MPa. These c' values are in good agreement with the range of values inferred from shear box and triaxial tests (Figs. 3 and 4), which implies that the operational E_s values controlling cavity expansion and cone penetration (as deduced using Yu and Houlsby, 1991) are very similar to those controlling lateral pile response. Guo (2015) also showed that specification of these

operational E_s values in elastic 3D Finite Element analyses of the Pinjar tests provided almost identical predictions of the $H-y_h$ responses predicted by the LAP program in Fig. 14; this agreement provides justification for the use of the Poulos (1971) correlation between E_s and the linear elastic component of the $p-y$ curve.

Support for the estimated q_c/c' ratio of 160 at Pinjar is provided in Fig. 15b, which plots q_c/c' against the ratio of the equivalent soil/rock modulus (E_s) to the unconfined compressive strength (q_u), where q_u is calculated from Eq. [1] with $\phi' = 40^\circ$. It is seen that q_c/c' is also about 160 for the mean of the E_s/q_u range of 200 to 580 reported by Goodman (1989) for sandstones and limestones.

The ultimate pressures recorded on the 340mm piles were approximately $0.045q_c$ whereas higher values of $0.07q_c$ were indicated by the tests on the 450mm piles (which explains the under-prediction for these piles on Fig. 14b). Therefore for an average P_u/q_c ratio of 0.06 and with $q_c/c' = 160$, P_u is approximately $10c'$ or $2.3q_u$ for the z/D range from which data could be extracted with accuracy ($z/D \leq 1.5$). Finite Element analyses of laterally loaded piles in a Tresca soil (e.g. Truong and Lehane 2014) suggest that likely upperbound P_u/c' or P_u/q_u ratios at large z/D values are about 15 and 3.5 respectively.

The foregoing analyses indicate that if a CPT has been performed at any given weak rock site and if c' and ϕ' can be measured or estimated, the operational equivalent linear elastic modulus (E_s) can be determined using Eq. [6] and the spherical cavity expansion solution of Yu and Houlsby (1991); an estimate of E_s can also be made using Fig. 15a. A simple bi-linear approximation to the rock's $p-y$ characteristic, of the form indicated on Fig. 12 (but noting $p=PD$), can then be constructed with an initial (elastic) slope equal to $0.8E_s$ extending to a maximum p value ($p_u = P_u D$) of approximately $0.06 q_c D$ (as stated above, this multiple of q_c may be conservative at large z/D values). The lateral pile analysis can be performed using

readily available commercial software with these p - y characteristics and specification of the pile's flexural rigidity and yield moment capacities.

Conclusions

Instrumented lateral pile load tests in weak calcareous sandstone have shown that the lateral response of this material is not well predicted by either the weak rock model of Reese (1997) or the cemented soil model of Reese and Van Impe (2001). This paper shows that the lateral behavior of piles in weak rocks and cemented soils is better related directly to the CPT q_c value with the equivalent elastic moduli of the rock controlling values of q_c at the test site being very similar to those controlling lateral pile response. A simple bi-linear p - y approximation of the lateral response is proposed requiring q_c data and an estimate of the rock's effective stress strength parameters. It is shown that standard lateral pile analysis with such p - y curves is likely to be sufficient for many practical purposes if the moment capacity of the pile section is incorporated in the analysis.

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Notation

The following symbols are used in this paper:

c'	=	effective cohesion;
D	=	pile diameter;
e	=	load eccentricity above ground level;
E_s	=	Equivalent operational linear Young's modulus;
G_o	=	small-strain shear modulus;
H	=	lateral load applied at pile head;
$I_{s,50}$	=	Point load index (corrected to 50mm diameter);
K_c	=	Brinch-Hansen c' pressure coefficient;
K_q	=	Brinch-Hansen vertical stress pressure coefficient;
M	=	pile bending moment;
M_y	=	yield moment of pile
P	=	lateral pressure;
p	=	lateral resistance per meter length;
p_{lim}	=	limiting pressure of cavity expansion;
P_u	=	ultimate soil/rock pressure;
q_c	=	cone tip resistance;
q_u	=	unconfined compressive strength;
t	=	wall thickness of steel pipe;
V_s	=	velocity of shear wave;
y	=	lateral displacement of soil/pile;
z	=	depth below ground surface;
A	=	strength reduction factor
y_h	=	deflection of pile head;
κ	=	pile curvature;
N	=	Poisson's ratio of soil or rock mass;
σ'_v	=	effective overburden stress; and
ϕ'	=	friction angle.

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Table 1. Test pile summary

Pile Nos.	External pile diameter D (mm)	Steel pipe diameter (mm)	Pipe Wall thickness t (mm)	Eccentricity of lateral load e (m)	E_{grout}^* (GPa)	Tensile & Comp. strength* (MPa)	Flexural rigidity (prior to cracking of external annulus) kNm^2	Moment at first yield M_y (kNm)	Plastic capacity M_p (kNm)
P340A & P340B	340	219	4.8	0.32	4	1.5 & 12	7200	67	90
P450A & P450B	450	356	6.4	0.2	4	1.5 & 12	31750	228	300

* Backfigured from on-site calibration test