Verification of numerically derived CPT based p-y curves for piles in sand

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ABSTRACT: This paper examines the potential of a numerically derived CPT-based formulation for *p-y* curves in sand (Suryasentana & Lehane 2014) to predict the response observed in lateral load tests conducted on six piles in four different sand deposits. A summary of the methodology employed in the derivation of this formulation is first described before presenting information related to each of the case histories examined. The lateral load displacement data measured in these case histories are shown to compare well with predictions obtained using the *p-y* formulation. This agreement should encourage further refinement of the formulation and ultimately the direct use of CPT q_c profiles for the analysis of laterally loaded piles in sand.

1 INTRODUCTION

The use of large diameter monopiles as foundations for onshore and offshore wind turbines has prompted renewed interest in the assessment of design methods that predict the response of piles to lateral load. There has been some doubt, for example, whether the American Petroleum Institute recommendations for piles in sand (API 2011) can also be applied to monopiles with diameters (D) that can be an order of magnitude larger than those used in support of the API recommendations (e.g. O'Neill & Murchison 1983). The growth of cone penetration testing (CPT) has also encouraged the search for prediction methods involving direct use of the CPT q_c value rather than use of inferred friction angles or relative densities in a given design method.

In response to the need for a rational CPT-based approach for laterally loaded piles in sand, Suryasentana & Lehane (2014) derived load transfer *p-y* curves for a wished-in-place pile in sand. The derivation was obtained via a regression analysis on a large series of 3D Finite Element computations that predicted the lateral pile response in a variety of different sands and a cavity expansion approximation using Finite Elements to predict corresponding CPTq_c profiles in each sand deposit.

The paper of Suryasentana & Lehane (2014) presents some examples that support the general form of the *p-y* formulation that they derived. This paper provides a brief summary of the steps followed in this derivation and then uses published case history data to examine the general suitability of the formulation.

2 NUMERICAL DERIVATION OF *P-Y* CURVES

The general procedure used by of Suryasentana & Lehane (2014) is summarised in Figure 1. Some notable features of the analyses performed are summarised as follows:

- *Plaxis 3D* Foundation (version 2.2) was used to predict the response to lateral load of the piles while Plaxis 2D (version 2012) was employed for the estimation of CPT q_c values.
- The sand was modelled in Plaxis 2D and 3D analyses using a non-linear elasto-plastic constitutive model, referred to as the Hardening Soil (*HS*) model (Schanz *et. al.* 1999). The small strain hardening soil model (HS-small), which includes a small strain overlay, was not used as it was not available in *Plaxis 3D Foundation* at the time of writing.
- The piles were modelled using solid elements with a Young's modulus comparable to that of uncracked concrete (30 GPa).
- No interface was assumed between the sand and the pile i.e. the analyses modelled a fully rough interface condition.
- The analyses employed dry sand throughout and hence the effective stress at depth, *z*, was given as the product of this depth and the specified dry unit weight (γ).
- q_c profiles corresponding to each laterally load pile analysis were derived using the procedure outlined in Xu & Lehane (2008), which involved calculation of the spherical cavity expansion limit pressures (p_{lim}) and use of the relationship between q_c and p_{lim} proposed by Randolph et al. (1994). A total of 10 Plaxis 2D analyses were ran, each taking 4 hours to process and extract p_{lim}.
- A total of 100 Plaxis 3D analyses were ran, each taking 8 hours to process and extract the bending moment (M) and displacement (y) data for a given lateral load, and subsequent derivation of net pressures following curve fitting of the *M* profiles.

Initial regression analyses of the collated *p* and *y* data yielded a format comparable to that proposed by Novello (1999), which was simply estimated from back-analyses of a small number of lateral pile tests. The best fit equation obtained was:

$$\frac{p}{\gamma z D} = 4.2 \left(\frac{q_c}{\gamma z}\right)^{0.68} \left(\frac{y}{D}\right)^{0.56} \qquad (1)$$

The similarity between the form and exponents of equation (1) with those of Novello (1999) was striking and provided some assurance to Suryasentana & Lehane (2014) of the applicability of the numerical approach employed.

Equation (1) gives a parabolic variation of net pressure (*p*) with displacement (*y*) and can be written in component form as follows to provide an indication of the relative importance of the CPT q_c value, the depth (z), average soil unit weight (γ) and pile diameter (D):

$$p = 4.2\ (q_c)^{0.68} (z)^{0.32} (y)^{0.56} (\gamma)^{0.32} (D)^{0.44} \qquad (2)$$

Equation (2) implies that a tenfold increase in pile diameter increases the stiffness of the *p-y* response by a factor of 2.75. This increase, which has important implications for monopiles referred to earlier, arises because of the lower strain levels in the sand mass around a larger pile for a given displacement (*y*). More careful examination of the predicted data indicated that equations (1) and (2) over-predicted pressures at large y/D values and therefore Suryasentana & Lehane (2014) proposed the following equation, which they demonstrated provided an improved match to their results:

$$\frac{p}{\gamma z D} = 2.4 \left(\frac{q_c}{\gamma z}\right)^{0.67} \left(\frac{z}{D}\right)^{0.75} \left(1 - exp\left(-6.2 \left(\frac{z}{D}\right)^{-1.2} \left(\frac{y}{D}\right)^{0.89}\right)\right) \qquad (3)$$

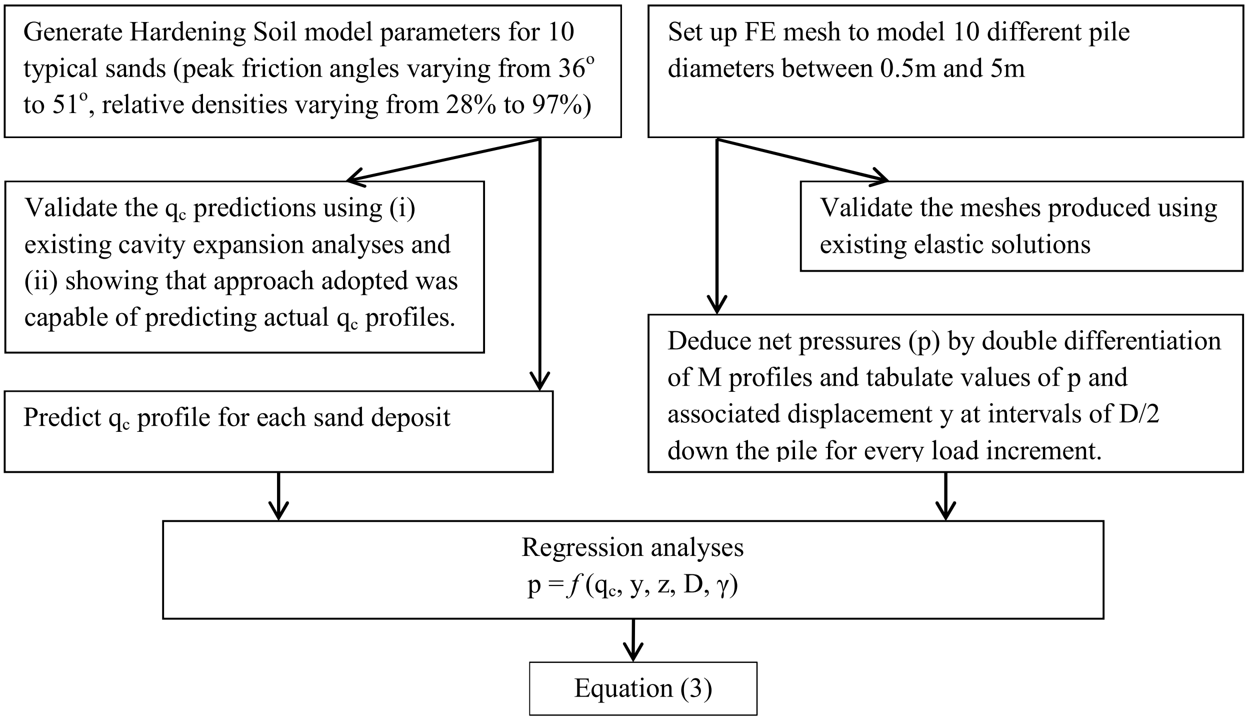

Figure 1. Methodology employed by Suryasentana & Lehane (2014)

3 CASE HISTORIES EXAMINED FOR VERIFICATION

The following case histories were examined to test the ability of equation (3) to predict the observed lateral pile load *vs*. head displacement responses.

1. Test piles reported by Pando *et al.* (2006) in a sand deposit at Hampton, Virginia, U.S.
2. Test piles reported by Luff (2007) for in a loose–medium dense dune sand deposit at Shenton Park, Perth, Australia
3. A centrifuge scale test pile reported by Ramadan *et al.* (2013) in dense sand at C-Core, Canada
4. Test piles provided by Venville (2004) in a sand deposit in North Perth, Australia

Case 1 has already been examined by Suryasentana & Lehane (2014) but is included in this paper for completeness.

3.1 *Case 1: Test Piles at Hampton (Pando et al. 2006)*

This case history involved full-scale lateral tests in medium dense silty sand on a prestressed concrete (PC) pile, a plastic pile (PP) and a fibre reinforced polymer (FRP) pile. The dimensions and flexural rigidity (EI) profile of each pile are described in detail in Suryasentana & Lehane (2014). The test setup and q_c profiles for the three piles are shown in Figure 2; no CPT profile is available for the PP pile but this is inferred to be similar to that at the location of the PC pile.

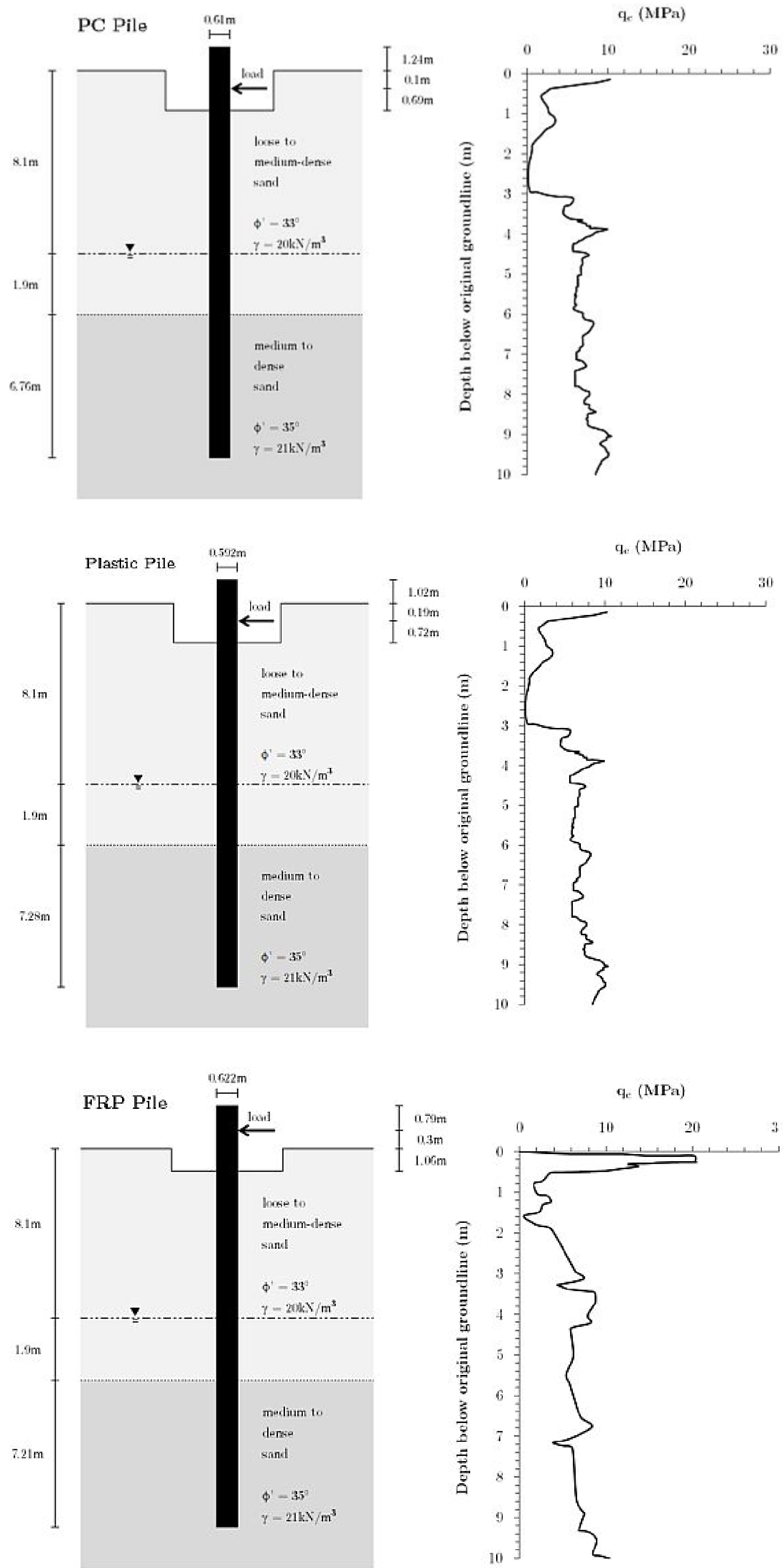

Figure 2. Case 1: Test setup and q_c profile

3.2 *Case 2: Test Piles at Shenton Park (Luff 2007)*

This case history involved jacking apart of two circular continuous flight auger (CFA) 225mm diameter grout piles tested in a loose to medium dry dense sand. The piles were installed at the University of Western Australia (UWA) test bed site at Shenton Park. Ground conditions at this site are described by Li & Lehane (2008) who also report a separate experiment (required for interpretation of a retaining wall experiment) that established the moment curvature relationship for the piles (which had an uncracked EI value of 2420kNm$^{2.}$). The setup and q_c profile at the test location are shown in Figure 3.

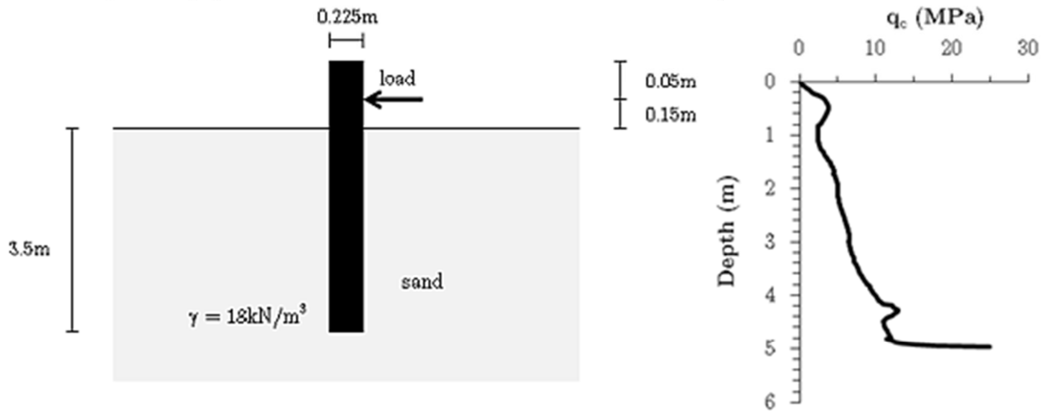

Figure 3. Case 2: Test setup and q_c profile

3.3 *Case 3: Centrifuge Piles tested at C-Core (Ramadan et al. 2013)*

This case history was performed at centrifuge-scale and involved lateral loading of an instrumented open-ended aluminium model pile in dense sand. The pile had a diameter of 18mm, wall thickness of 1.5mm and was 300mm in length. The pile was jacked into the sand bed and tested at a centrifuge acceleration of 70*g*. At prototype scale, the piles are 1.4m in outside diameter, 21m in length and have an EI of 4484 kNm^2. The setup and q_c profile of the test are shown in Figure 4. A key difference between this case and the other cases examined is that this case involved fully saturated sand while the others involved dry sand and therefore Equation (3) requires input of the buoyant density (γ') in place of γ.

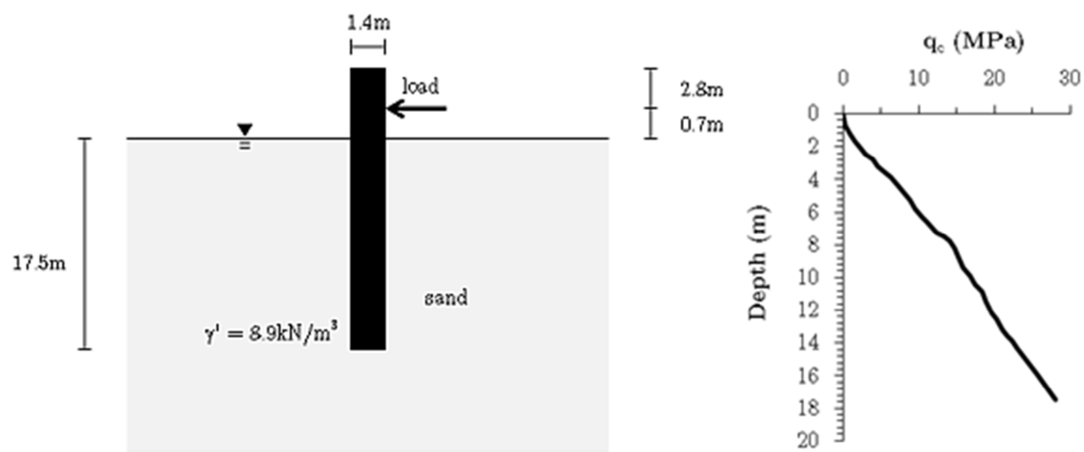

Figure 4. Case 3: Test setup and q_c profile (Prototype dimensions)

3.4 *Case 4:Test Piles in North Perth (Venville 2004)*

This case history is similar to Case 2 (which was also performed in Perth) except that the sand was significantly denser and a larger CFA pile diameter of 340 mm was employed. The two test piles were 6 m in length with an uncracked EI value of 11500 kNm^2, which dropped to 3500kNm^2 upon exceeding a bending moment of 15 kNm. The setup and q_c profile of the test location are shown in Figure 5. It is noted that there was an initial "seating load" of approximately 8kN for this test case.

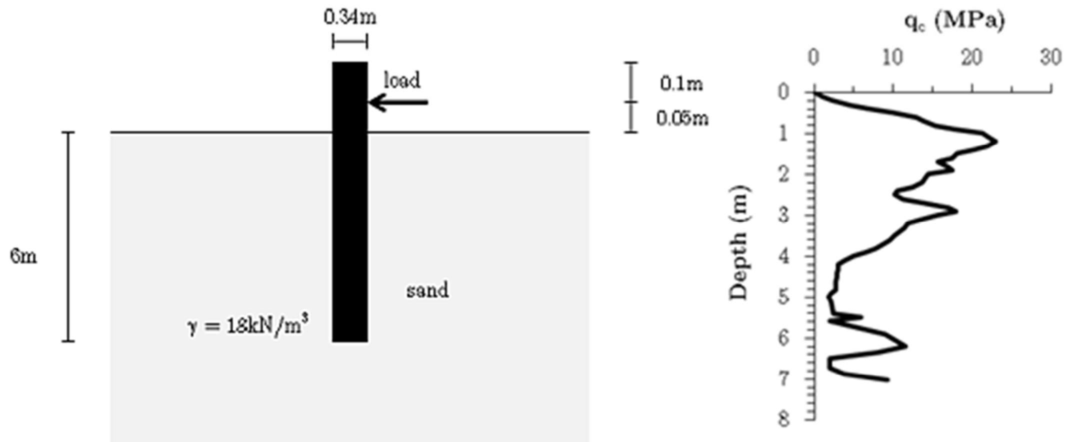

Figure 5. Test setup and q_c profile of Case 4

4 PREDICTED AND MEASURED PILE RESPONSES

p-y curves were derived at depth intervals of D/2 for each case history using the CPT q_c profiles shown on Figures 2 to 5. The Oasys ALP program (Oasys 2013) was used to conduct the analyses. This program, which is similar in form to many commercially available laterally loaded pile programs, represents the pile as a series of beam elements and the soil as a series of non-linear, non-interacting springs located between each beam element. The derived *p-y* curves were modeled by the program by ten carefully selected discrete lines that provided a good representation of the curves.

Figure 6 compares the measured and predicted lateral load – pile head displacement responses. The average measured load displacement response is plotted when two test piles were employed in any given case history. It is evident that the agreement between measurements and predictions is very good despite the relatively wide variety of pile types and ground conditions considered. Exact agreement should not be expected given the acknowledged limitations of the analyses of Suryasentana & Lehane (2004). For example, the very stiff response measured in Case 2 arises due to the slightly cemented nature of this material and the high small strain stiffness of the sand deposit relative to the q_c value (Lehane et al. 2008); this behaviour would not be captured using the H-S soil model used in the derivation of Equation (3). In addition, some errors in predictions can be expected due to difficulties in establishing precise EI variations during load tests on concrete piles; the modelling of such variations in the ALP analyses required an iterative procedure.

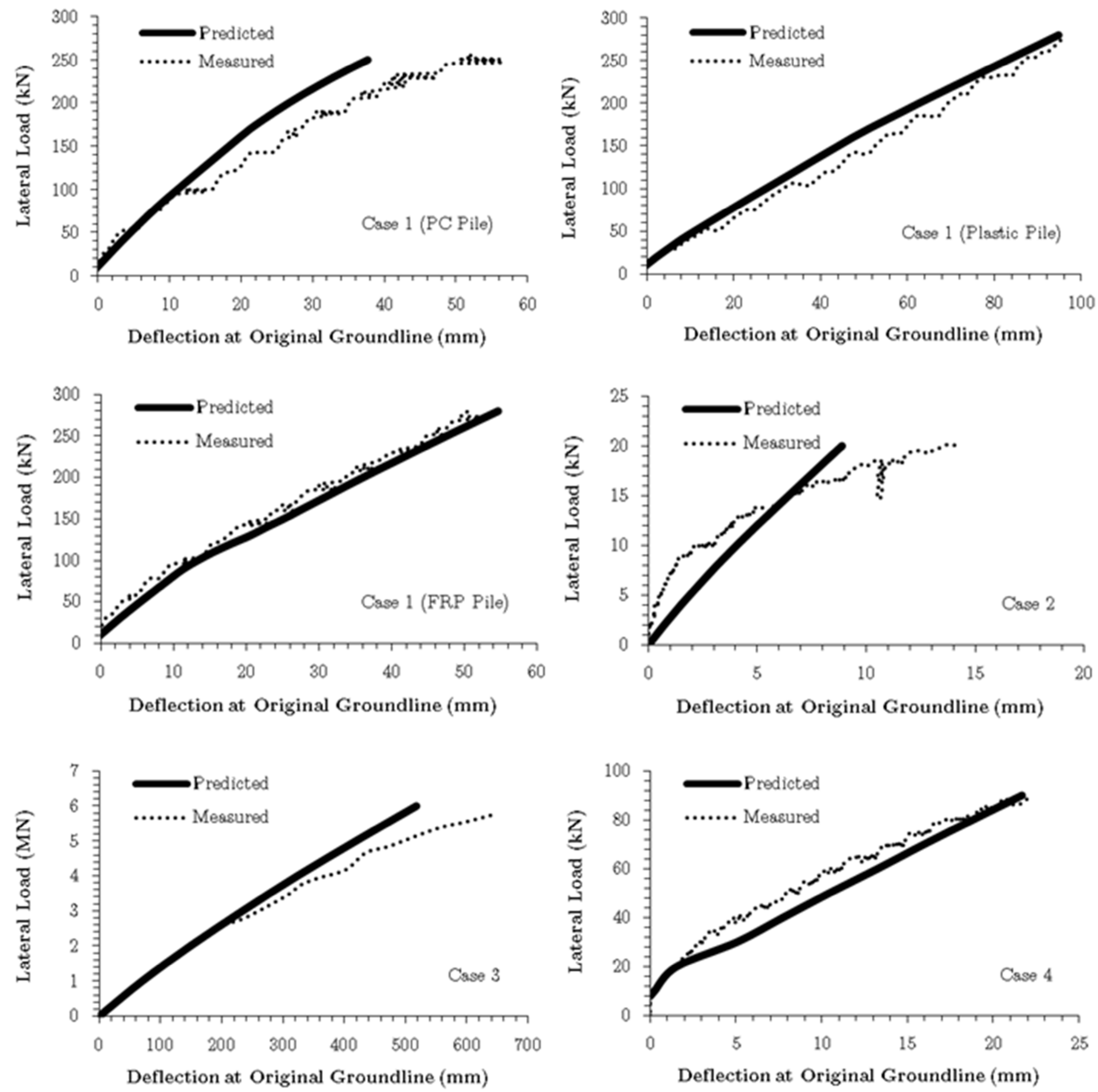

Figure 6. Comparison of predicted and measured responses for Case 1 to 4

5 CONCLUSION

This paper has examined a number of case histories to assess the ability of a numerically derived CPT-based *p-y* formulation for piles in sand to predict lateral pile response. The comparisons between predictions and measurements for 6 test piles at 4 sites have been encouraging and they provide strong evidence of the potential for CPT-based method for laterally loaded pile design. Further examinations of existing case histories will assist refinement of the basic equation (Equation 3).

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