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A *p-y* Approach to Predict the Lateral Load Capacity of Piles Socketed into Melbourne Mudstone

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ABSTRACT

Rocks are a highly variable entity where the disparity in composition, depositional history and weathering condition cause them to have different strength and deformation behaviour. For laterally-loaded rock-socketed piles, these factors are crucial in determining the lateral capacity of the pile. In this paper, extensive numerical modelling was undertaken for pile socketed into mudstone rock using a three-dimensional discrete element package, *3DEC*. The numerical model was based on a calibrated model established by using a full-scale lateral pile load test conducted in mudstone. The benchmark model was extended to investigate several pre-determined influential rock and pile parameters and the results were incorporated to form a new *p-y* criterion for Melbourne mudstone. The proposed *p-y* method was subsequently validated using another full-scale pile test in mudstone and good agreement was observed.

Keywords: Numerical modelling, *p-y*, mudstone, lateral capacity

1 INTRODUCTION

Many civil engineering infrastructures (e.g., tall buildings, bridges, retaining walls, transmission towers} are designed to resist high lateral loads generated by wind, waves and earthquakes. The design methods for laterally loaded piles embedded in soils are well established and calibrated by extensive full-scale field tests (Meyerhof 1976; Vesic 1977). However, there is a lack of research and field test data on laterally loaded piles socketed into rocks. Recent advancements have focussed on developing methods for predicting the lateral load behaviour of rock socketed piles using elastic continuum and subgrade reaction (*p-y*) methods (Reese 1997; Yang 2006). A few notable analytical methods (Reese 1997; Yang 2006) have been devised for specific rocks but significant errors were observed when these methods were applied on different types of rocks (Chong 2011). This is due to the substantial variation in strength and deformation behaviour of rocks caused by variations in composition, depositional history as well as weathering condition. To the authors' knowledge, there has not been sufficient study conducted or analytical method proposed to predict the lateral capacity of piles socketed into the local Melbourne mudstone (Chong et al 2008; Chong 2011). Therefore, for more efficient design solutions, it is imperative to develop an analytical method capable of capturing the local mudstone's lateral behaviour.

In this study, three-dimensional distinct element code, *3DEC* was employed to investigate numerically the lateral behaviour of piles socketed into mudstone rock. The numerical model was based on a calibrated model established for laterally-loaded piles socketed into Melbourne mudstone (Chong et al 2011a; Chong 2011). In this paper, the benchmark model was employed to examine the effect of several influential rock and pile parameters on the *p-y* and the corresponding load-deflection behaviour of piles socketed into mudstone. A new *p-y* criterion was devised by incorporating the results obtained from the parametric study. The proposed *p-y* criterion adopts a hyperbolic form, as shown in Equation 1, where the curve is governed by K_i (initial subgrade reaction modulus) and p_{ult} (ultimate rock resistance). The *p-y* criterion was subsequently input into computer program, PYGMY (Stewart 2000), to estimate the pile head load-deflection behaviour of another full-scale pile test. The predicted pile head response was observed to match the test results closely.

$$p = \frac{y}{\frac{1}{K_i} + \frac{y}{p_{ult}}} \qquad (1)$$

Where, p = resultant rock mass resistance per unit pile length (kN/m); y = corresponding ; pile deflection (m); K_i = initial subgrade reaction modulus (kN/m^2); p_{ult} = ultimate resistance of rock (kN/m)

2 NUMERICAL MODELLING

The numerical model established for laterally-loaded piles socketed into Melbourne mudstone (Chong et al 2011a; Chong 2011) was employed to carry out parametric study on a host of rock and pile properties. The effect of these parameters on the initial subgrade reaction modulus (K_i) and the ultimate rock resistance (p_{ult}) of the *p-y* curve was subsequently analysed to establish a new *p-y* criterion for laterally-loaded piles socketed into Melbourne mudstone.

2.1 Initial Subgrade Reactiong Modulus (Ki)

The initial sub-grade reaction modulus for mudstone, K_i, is dependent on a host of rock and pile parameters. It has been shown by Chong et al (2011b) using laboratory-scale modelling that rock modulus, pile diameter and pile bending stiffness are the key parameters influencing the load-deflection response of piles socketed into mudstone. Therefore, this study carried out more detailed parametric study on these influential parameters. K_i can be mathematically expressed as shown in Equation 2. The effects of the five parameters were investigated in more detail using the *3DEC* model established based on full-scale pile-load tests.

$$K_i = f(E_r, D, v, E_p I_p) \tag{2}$$

Where, K_i = initial subgrade reaction modulus for homogenous mudstone (kN/m^2); E_r = rock modulus (kPa); D = pile diameter (m), v = Poisson's ratio of rock; E_pI_p = pile bending stiffness (kNm^2)

<u>*Effect of Mudstone Modulus (E_r)*</u>

Four different mudstone modulus values (62, 220, 540 and 1320MPa) with other parameters held constant were simulated and the respective *p-y* behaviour obtained as shown in Figure 1a. The figure shows the *p-y* behaviour becomes stiffer with higher rock modulus while the ultimate rock resistance approaches a similar value. With this, the K_i for each case was then calculated. Figure 1b depicts the variation of K_i with different rock modulus and shows a strong linear relationship (R^2=0.99). Similar linear behaviour was also reported by Yang (2006) in his FEM modelling.

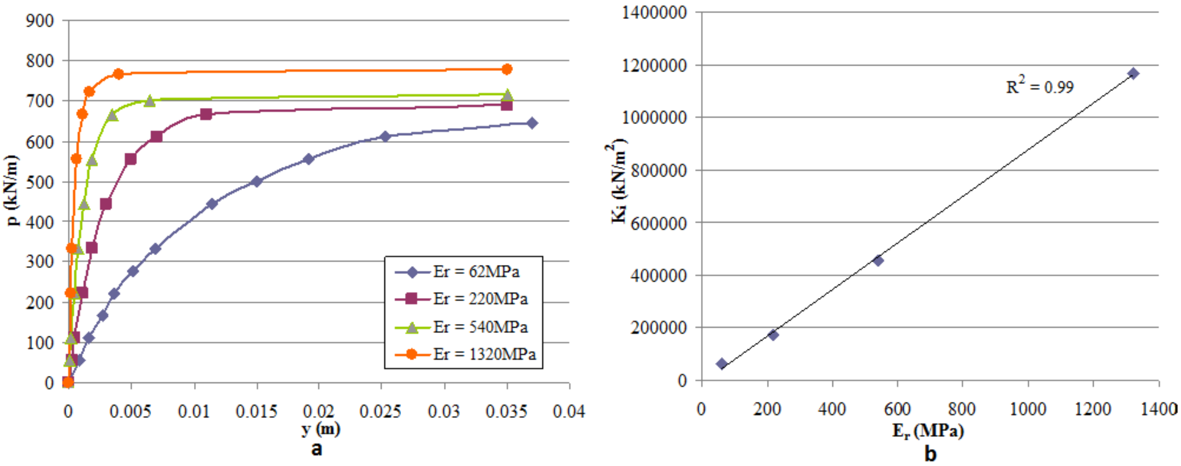

Figure 1 a) *p-y* behaviour and b) Variation of K_i for different mudstone modulus

<u>*Effect of Pile Diameter (D)*</u>

A number of researchers have found contradictory results for the effect of pile diameter on K_i. Terzaghi (1955) and Vesic (1961) suggested that pile diameter can be omitted when estimating K_i. However, Carter (1984) observed from his field test results that pile diameter has a substantial effect on K_i. Yang (2006) also concluded in his FEM studies on the effect of pile diameter that it has a linear relationship with K_i. Therefore, an investigation was carried out to study the effect of pile diameter on K_i for mudstone. The full-scale *3DEC* model was extended to simulate three different pile diameters of 0.3, 0.6 and 1.2m while keeping the other parameters fixed. The respective *p-y* behaviour is plotted in Figure 3a.

As can be seen from this figure, the *p-y* response becomes stiffer in both the linear and non-linear regions with larger pile diameter. The variation of K_i for different pile diameters is illustrated in Figure 3b. It can be seen that pile diameter has a strong linear relationship with K_i (R^2 = 0.99). Similar linear behaviour was also reported by Yang (2006) in his FEM study based on relatively stronger rocks (shale and sandstone).

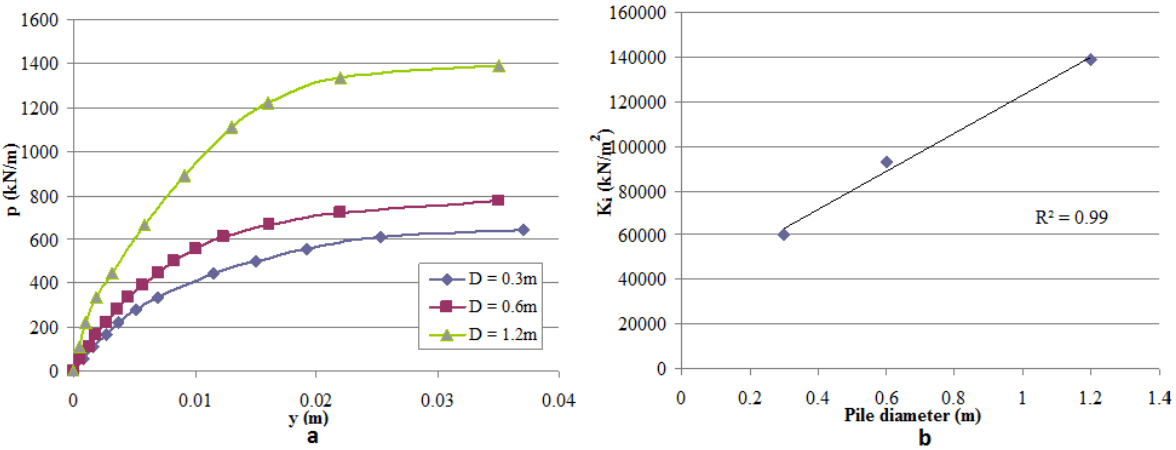

Figure 3 a) *p-y* behaviour and b) Variation of K_i for different pile diameters

<u>Effect of Poisson's ratio of Mudstone (ν)</u>

The effect of Poisson's ratio on K_i was examined by varying the values from 0.2 to 0.4 based on the full-scale *3DEC* model. Figure 4a depicts the average *p-y* curves for different Poisson's ratios simulated. As can be seen from this figure, the initial linear part of the average *p-y* curves is identical for all the Poisson's ratios investigated. It has also been found by Williams (1980) and Chiu (1981) that the Poisson's ratio of the mudstone falls consistently in the range of 0.25 – 0.30 regardless of the weathering conditions. Therefore, the effect of Poisson's ratio can be disregarded in the subsequent derivation of K_i for mudstone.

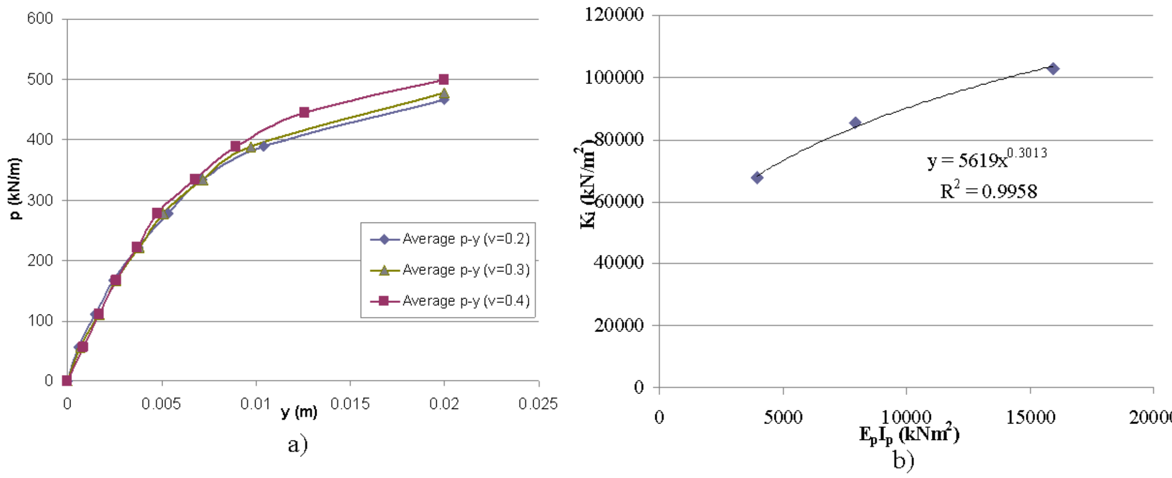

Figure 4 a) Average *p-y* curves for different Poisson's ratios; b) Variation of K_i with different pile bending stiffness, E_pI_p

<u>Effect of Pile Bending Stiffness (E_pI_p)</u>

Three values of pile bending stiffness (E_pI_p) of 3980, 7960, 11940 and 15920 kNm^2 representing pile modulus of 10, 20 and 40 GPa were simulated while maintaining other parameters constant. The K_i values for different pile bending stiffnesses were calculated and plotted in Figure 4b. It can be seen from this figure that a power relationship (R^2=0.99) exists between K_i and E_pI_p. This finding is consistent with that of Vesic (1961) as observed for soils and Yang (2006) for relatively hard rocks.

The detailed study conducted on the key influential parameters given in Equation 2 found that mudstone modulus, pile diameters and pile bending stiffnesses have significant effects on K_i and their combined effects on K_i can be mathematically expressed by Equations 3-4. A constant (*d* = 0.3m), which is the same as the pile diameter tested in this study, is required to normalise the effect of pile diameter. In Equation 4, the mudstone modulus can be estimated using the well-established water content relationship (Chiu, 1981) which offers this method a distinct advantage over the existing *p-y* criteria (Gabr et al., 2002; Yang, 2006) that require modulus to be calculated using highly subjective rock classification systems such as RMR and GSI.

$$K_i = \eta(E_r)\eta(D)\eta\left(\frac{E_pI_p}{E_rD^4}\right) \quad (3) \qquad K_i = \mathrm{A}E_r(\frac{D}{d})\left(\frac{E_pI_p}{E_rD^4}\right)^{0.3} \quad (4)$$

Where, A = proportionality constant based on combined effects; *d* = normalisation constant (0.3m); E_pI_p = pile bending stiffness (kNm^2)

Proportionality constant, A, takes into account the combined effect of all the key parameters shown in Equation 4. The predicted K_i values using Equation 4 are plotted against the K_i derived from *3DEC* models in Figure 5. As can be seen from this figure, the relationship between both initial sub-grade reaction moduli is linear (R^2 = 0.99) with a slope of 1:1. This implies a proportionality value of 1 is obtained for the given mudstone.

2.2 Ultimate Rock Resistance (p_u)

The ultimate rock resistance, p_u, for laterally-loaded piles socketed into rock has been investigated by a number of researchers (Gabr et al., 2002; Yang, 2006). Rock classification systems such as Rock Mass Rating (RMR) (Bieniawski, 1989) or Geological Strength Index (GSI) (Hoek and Brown 1988) have been adopted to account for the secondary structures in the rock mass when computing its ultimate rock resistance. The distinct disadvantage of using these classification systems is the inconsistency in assigning a rating value for a given rock condition. As it is based on the user's experience and judgement, very large discrepancies may occur between different users. Therefore, to diverge from the unreliable nature of the rock classification systems which have been shown to produce large under-prediction of the ultimate rock resistance for the local mudstone (Chong 2011), a different approach is adopted in this study to develop a more practical and reliable method to compute the ultimate rock resistance, p_u.

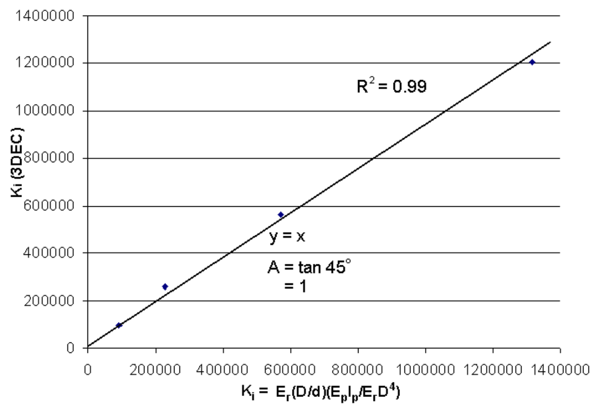

Figure 5 Comparison of *3DEC* and predicted (Equation 4) initial subgrade reaction modulus

This study explored the use of an empirical formula to estimate the ultimate rock resistance, p_u, for laterally-loaded piles socketed into mudstone by taking into consideration the wealth of knowledge available from previous studies carried out at Monash University on rock-socketed piles in mudstone. Based on field tests carried out on axially-loaded piles socketed into mudstone, Pells (1980) suggested the use of a bearing capacity factor with the unconfined compressive strength (UCS) of the mudstone to estimate the ultimate bearing capacity of the rock. UCS is a well-established property of mudstone and is widely used in many empirical equations developed for foundation engineering. Pells (1980) proposed bearing capacity factors of 6 and 12 for mudstone at surface and deeper depth, respectively.

In the context of laterally-loaded piles socketed into mudstone, the same approach was employed for predicting the ultimate lateral rock resistance, p_u. A series of different bearing capacity factors was investigated and it was found that a bearing capacity factor of 7 produced the closest and the most consistent estimation of the ultimate mudstone resistances for different water contents. Therefore, Equation 5 was adopted for predicting the ultimate rock resistance.

$$p_u = 7q_u D \quad (5)$$

The two relationships (Equations 4 and 5) proposed for determining K_i and p_{ult}, respectively, were subsequently incorporated into the general hyperbolic equation (Equation 1) to produce the complete *p-y* curve for laterally-loaded piles socketed into mudstone rock. The *p-y* predictions based on the two proposed equations are compared with the *p-y* curves of different mudstone water contents (Chong 2011), as shown in Figure 6a. Figure 6b presents the corresponding pile-head load-deflection responses for different water contents of the mudstone.

In Figure 6a, it can be seen that the new hyperbolic *p-y* criterion proposed in this study is able to capture quite closely the variations encountered in the *p-y* curves for different water contents of mudstone. Both the linear and non-linear regions of the computed *p-y* curves for water contents ranging from 5% to 20% match very closely the simulated results of *3DEC* modelling. Similar good agreement can be observed in the prediction of corresponding pile-head load-deflection responses for different mudstone water contents as shown in Figure 6b. In addition to more accurate predictions, the proposed equation is more practical due to the requirement of only basic rock properties such as modulus and UCS, which are functions of a single variable, the water content of mudstone, as opposed to the use of cumbersome correlations with GSI or RMR. The proposed *p-y* criterion is further validated against another full-scale pile-load tests conducted in Melbourne mudstone.

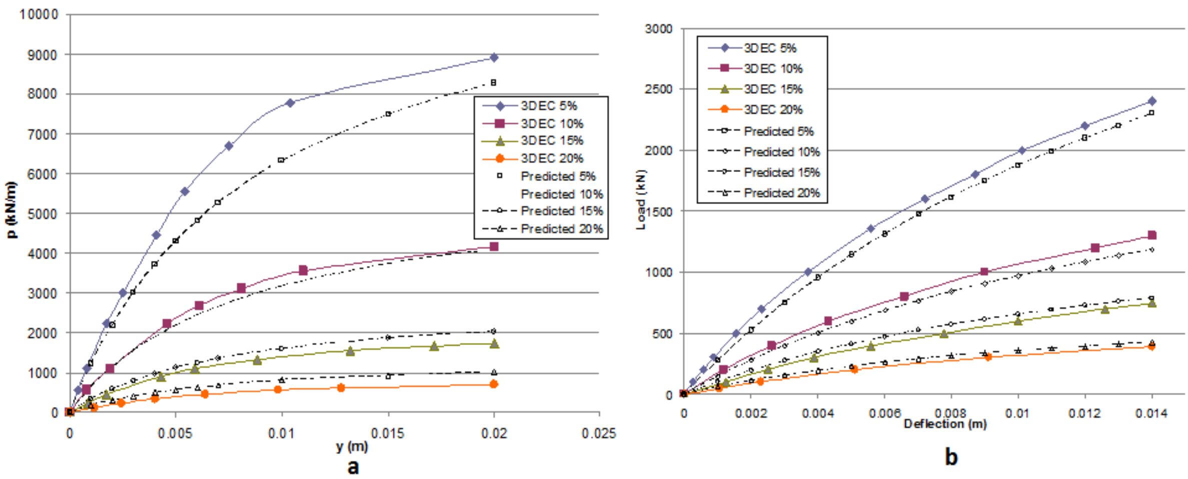

Figure 6 a) *p-y* curves and b) Pile head load-deflection predictions using the proposed hyperbolic *p-y* criterion

2.3 Validation of Proposed Method with Full-Scale Pile Load Tests

The results of another full-scale field pile-load tests was utilised to validate the proposed *p-y* criterion. The pile test was conducted at the same site as described in Chong et al (2011a) and the water content of the mudstone was approximated to be 20%. The pile tested had a socket lengths of 1.2m (*L/D* = 4). The *p-y* curves estimated for the given mudstone condition were input into PYGMY (Stewart 2000) to generate the corresponding pile-head load-deflection responses as shown in Figures 8. The computed load-deflection curves using the proposed *p-y* criterion show acceptable agreement with the pile test results. The close predictions reinforce the feasibility and effectiveness of the proposed *p-y* criterion to predict the load-deflection behaviour of laterally-loaded piles socketed into mudstone.

3 CONCLUSION

The analysis for laterally-loaded piles socketed into rock is more complex than its soil counterpart due to the substantial variation in the nature of rocks having different composition, depositional history and weathering condition which cause the variation in strength and deformation behaviour. Existing analytical methods for laterally-loaded rock-socketed piles were derived specifically for a given rock condition and their application on other rock types are questionable and must be treated with caution. To date, there have not been sufficient studies carried out to develop an analytical tool to determine the lateral capacity of pile socketed into the local Melbourne mudstone. This study pioneered the detailed investigation on the lateral behaviour of piles socketed into Melbourne mudstone. A new subgrade reaction (*p-y* approach) has been proposed in this paper that can be used in conjunction with computer package such as PYGMY, to generate the pile head load-deflection response. Through extensive parametric study, the effects of influential parameters such as rock modulus, UCS, pile diameter and pile bending stiffness have been incorporated into the proposed *p-y* criterion. A full-scale lateral pile load test has been employed to validate the proposed *p-y* criterion and close agreement with the predicted pile head load-deflection response has been achieved.

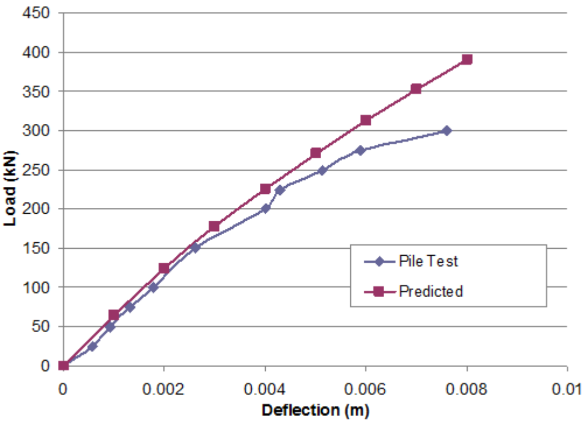

Figure 8 Comparison of pile head load-deflection of predicted pile test

4 ACKNOWLEDGEMENTS

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