

**Reply to discussion by Zhang, Feng, Lie and Zhang on “An analytical approach for the prediction of single pile and pile group behaviour in clay” by Brian B. Sheil, and Bryan A. McCabe [Comput. Geotech. 75 (2016) 145-158]**

The authors are grateful for the discussers' interest in our work. The discussers have raised a number of important points that we are delighted to clarify and expand upon.

**Iterative process for loaded single pile**

The first question raised by the discussers relates to the process used to establish the local shear stress for each segment of a single pile. Combining Eqs. 1-3 from the original paper [1], we obtain the following expression:

$$w = \sum_r^{r_m} \frac{\tau}{G_0 \left( 1 - f \left( \frac{\tau}{\tau_f} \right)^g \right) \left( \frac{p'}{p'_0} \right)^n} \Delta r \quad (R1)$$

which relates  $w$ , the soil displacement and  $\tau$ , the shear stress, at a particular radial distance,  $r$ , from the pile, where  $G_0$  is the small-strain shear modulus,  $f$  and  $g$  are curve-fitting parameters,  $\tau_f$  is the maximum allowable soil shear stress and  $p'$  and  $p'_0$  are the current and far-field mean effective stresses, respectively.

As the discussers point out, the value of  $(P_{11})_{n1}$ , defined in the original paper, is required to calculate the pile compression,  $w_c$ , for that segment and is initially unknown. The process used in the original paper is similar to the method employed by Zhang and Zhang [2] except that  $(P_{11})_{n1}$  is initially set equal to  $(P_{11})_{n2}$  for the first trial instead of assuming zero compression strain. Once  $w_c$  is calculated using  $(P_{11})_{n2}$  as an average load over the segment, the settlement at the midpoint of the pile is obtained. The local shear stress,  $\tau$  can then be calculated from Eq. R1 using the GOAL SEEK function in EXCEL. Once a load distribution along the full length of the pile is obtained, further iterations are carried out using the updated values of  $(P_{11})_{n1}$  to obtain a more accurate value of the pile compression. Obviously Eq. 6 does not apply to a pile loaded in tension as the base load is simply set equal to zero and the same process described above is followed.

**Shaft resistance in compression and tension loading**

The second point raised by the discussers relates to the use of Eq. 4 for both compression and tension loading. It should first be noted that the maximum allowable shear stress in the soil at failure is of course equal to the undrained shear strength  $s_u$  and therefore Eq. 4

should read  $\tau_f = s_u$ . The shear strength at the pile-soil interface is defined by Eq. 5 and not by Eq. 4 as stated by the discussers. This involves factoring the undrained shear strength of the undisturbed soil mass by the interface strength reduction factor,  $R_{inter}$ . The value of  $R_{inter}$  can be deduced from site-specific interface tests and may also be used to account for a reduction in shaft resistance due to tension loading, if appropriate.

In case I, the authors compared predictions determined using the method under discussion to finite element calculations performed by Jardine et al. [3]. The aim was to replicate, as closely as possible, the modelling conducted by those authors. The difference between tension and compression shaft resistance has largely been attributed to (i) a loss in effective stresses immediately surrounding the pile and (ii) 'barrelling' of the pile. Given that the soil was modelled as fully undrained in that study and no interface elements were included, no explicit reduction in shaft resistance due to tension loading was adopted in our prediction of that case history. However, this should not be viewed as an endorsement of the use of the same shaft friction in compression and tension for any other case histories.

### **Pile cap behaviour**

A thorough examination of pile groups with intermediate cap stiffness necessitates the use of finite element modelling to capture the behaviour of the pile-soil-cap system properly [4]. This study is limited to free-standing pile groups with perfectly-rigid or perfectly-flexible pile caps; the influence of pile cap-soil interaction is therefore not considered. Single pile behaviour is extended to pile group behaviour using two-pile interaction factors coupled with the principle of superposition. As mentioned by the discussers, the response of a pile group with a perfectly-flexible pile cap can be obtained by imposing the same loads on all pile heads. A perfectly-rigid pile cap is analysed by equating the pile head settlement of all piles in the group and the total load on the group can be obtained by summing the individual pile head loads. Symmetry is employed to reduce the number of unknowns in the equations. An additional advantage associated with symmetry is that moment equilibrium is also enforced. The process of analysing either perfectly rigid or perfectly flexible pile groups using two-pile interaction factors has been widely documented in the literature and further discussion is available elsewhere [5].

### **Stress-strain relationship**

Another point the discussers raised was the determination of the stiffness degradation parameters  $f$  and  $g$ . This formulation was originally proposed by Fahey and Carter [6], and

has since been adopted by numerous authors including Lee and Salgado [7] and is not attributable to the authors of this paper. Parameter  $f$  controls the strain at which peak strength is reached and parameter  $g$  controls the curvature. The selection of these parameters can be obtained from curve-fitting to measured stress-strain or modulus degradation curve as shown in the original paper.

The values of OCR listed in Table 1 represent the extreme values for the particular profile considered. All four cases involved piles installed in soil with OCR values predominantly lower than 4; areas with  $OCR > 4$  had a negligible influence on the overall pile response and any differences erred on the side of conservatism. However, the authors agree that the OCR is an important parameter in the proposed method and piles installed in soils where high OCR values dominate should be considered explicitly. Fig. 4 provides an illustration of the results obtained from a numerical study [8] which was limited to  $OCR = 4$  due to convergence issues associated with modelling non-associated flow using an implicit integration scheme. For higher OCR soils, these results can easily be replaced with updated numerical predictions or field data.

With regards to the last point noted by the discussers, this is not an error in the paper. Note that Table 1 lists the pile fabrication specification of the Young's modulus which is 210 GPa for a steel pile. As mentioned in the paper, given that a pipe pile was used, the authors adopted a solid elastic cylinder with  $E = 28$  GPa in the modelling to give an equivalent axial stiffness as the steel pipe.

## Corrigenda

- On Fig. 9 in the original paper, the value of parameter  $g$  should read 0.3 as cited in the text and listed in Table 1.
- All instances of  $\tau_f = 0.5 \cdot s_u$  should read  $\tau_f = s_u$ . The limiting shear stress that can be maintained at the interface is therefore  $R_{inter} \cdot s_{u,soil}$

[1] Sheil BB, McCabe BA. An analytical approach for the prediction of single pile and pile group behaviour in clay. *Comput Geotech.* 2016;75(145-58.

[2] Zhang Q-Q, Zhang Z-M. Simplified calculation approach for settlement of single pile and pile groups. *Journal of Computing in Civil Engineering, ASCE.* 2012;26(6):750-8.

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[4] Sheil BB, McCabe BA. Numerical modelling of pile foundation angular distortion. *Soil Found.* 2015;55(3):614-25.

[5] McCabe BA, Sheil BB. Pile group settlement estimation: suitability of nonlinear interaction factors. *ASCE Int J Geomech.* 2015;15(3):04014056.

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