Modelling of pile response in laterally spreading liquefiable ground

S Wang
Beca Consultants, Wellington, NZ (formerly University of Auckland, NZ)
Jackson.wang@beca.com

R P Orense
Department of Civil and Environmental Engineering, University of Auckland, NZ
r.orense@auckland.ac.nz (Corresponding author)

Keywords: earthquake, pile foundation, liquefaction, lateral spreading, finite element method

ABSTRACT

This paper aims at creating and assessing the feasibility of OpenSees-based finite element (FE) model used to simulate the response of a pile foundation in liquefying ground. A range of sources has been adopted in the definition of model components of which its validity is assessed against case studies presented in literature. A simple lateral load test is at first considered without the presence of liquefaction which confirmed that the backbone force density-displacement (*p-y*) curve simulating lateral pile response is of sufficient credibility for relatively shallow piles. With the application of lateral spreading, the FE model produces results in correlation with that of a relatively simplified equivalent linear procedure in terms of pile displacement and bending moment. The use of a bi-linear moment-curvature relationship in approximation of the typical tri-linear concrete M-Φ curve was able to adequately capture the response of a reinforced concrete pile subject to free-field ground displacements in liquefied ground with a degradation factor of 0.01.

1 INTRODUCTION

In order to increase the seismic resistance of engineering structures, pile foundations are commonly used. Past disastrous earthquakes have shown evidence that structures supported by pile foundations performed well even when the adjacent ground liquefied. On the other hand, a number of pile foundations have been found to be damaged during major earthquakes due to the effects of lateral forces caused by soil movement associated with liquefaction.

In this paper, a pseudo-static analysis procedure based on a beam on a nonlinear Winkler foundation (BNWF) concept implemented through the open source FE analysis package OpenSees (Open System for Earthquake Engineering Simulation) is discussed. OpenSees is operated by the University of California Berkeley and developed with sponsorship from the Pacific Earthquake Engineering Research Center. The package, which is a fully nonlinear object orientated computer code (Mazzoni et al. 2011), has been utilised in the simulation of pile foundations. Created models are validated via case studies involving both static loading and lateral spreading. Results indicate that such models are capable of adequately capturing pile responses within a range of ground conditions pertaining to soil properties and location of the liquefied layer.

2 MODEL DEFINITION

The finite element (FE) model implemented within OpenSees adopts a series of nodes adjoining pile and soil elements to represent pile behaviour. Displacement-based beam elements have been adopted to represent the pile while nonlinear horizontal and vertical springs act to simulate the soil behaviour. Zero-length spring components form the basis of *p-y*, *t-z* and *q-z* springs permitting translation of the pile in *x* and *z* directions. Horizontal responses are governed by *p-y* springs, while *t-z* springs act in simulating shaft friction. Additionally, the pile tip response is represented using a *q-z* spring. These springs are linked to slave nodes which are subsequently constrained to pile nodes via equal constraints. Forces or displacements can be imposed upon either the pile or fixed soil nodes emulating a range of loading scenarios.

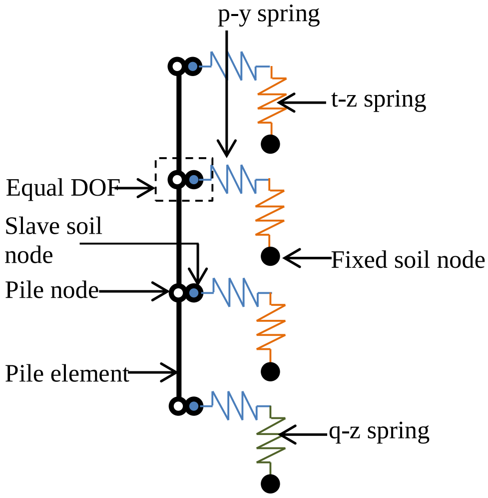

Figure 1. Definition of FE model implemented within OpenSees.

The various components of the model, which is based on the work by McGann et al. (2011), are outlined in Figure 1. Full details of the modelling and relevant equations are available in Wang and Orense (2013).

2.1 Modelling of *p-y* springs

p-y curves form the basis in simulating the horizontal response of the soil. OpenSees requires definition of both the ultimate lateral resistance (p_u) and the displacement at which 50% of the ultimate lateral resistance is mobilized in monotonic loading (y_{50}).

2.1.1 Definition of backbone resistance curve

The backbone curve is modelled on formulae established by Hansen and Christensen (1961). The resultant pressure e^D (passive minus active pressure) per front area of the pile governing the ultimate lateral resistance in terms of depth is generalized as:

$$e^D = \bar{q}K_q^D + cK_c^D \tag{1}$$

where $\bar{q}$ is the effective overburden pressure and c the cohesion. K_q^D and K_c^D are parameters relating to pressure at an arbitrary depth and are dependent solely on the angle of internal friction (ϕ). The distribution of initial stiffness (k_T) is necessary to compute the displacement y_{50}. For sands, this stiffness is dependent upon ϕ and on location of the soil with respect to the water table. The curves specified by the American Petroleum Institute (API, 1987) are used for this purpose. The continuous function of the *p-y* relation in the form of a hyperbolic tangent proposed by McGann et al. (2012) is employed.

2.1.2 Modification for influence of liquefaction

Although dramatic reductions in initial stiffness and ultimate lateral resistance occur within liquefied layers, McGann et al. (2012) has shown that non-liquefied soil at the layer interface will also experience significant reductions in shear strength. Therefore, the reduction in p_u and k_T for these regions must also be accounted for. This is achieved through modifying the *p-y* relationship through ratios which are functions of depth. These ratios when applied to p_u or k_T values in their homogenous state will capture its liquefied profile.

2.2 Modelling of *t-z* springs

The *t-z* relation defines the vertical response of the pile due to shaft friction. OpenSees requires definitions of the ultimate vertical resistance (t_u) and the displacement at which 50% of the ultimate resistance is mobilized in monotonic loading (z_{50}). For this purpose, the determination of t_u was based on formulae proposed by Kulhawy (1991) of which the horizontal stress down the pile (z) is expressed in terms of the vertical stress. Note that as OpenSees formulates the ultimate resistance as a force, the stress value must be converted via multiplying it by the tributary length of each pile node. Computations of z_{50} requires the initial tangents based on the friction angle (k_f). To simplify this, the tangents are founded on empirical data obtained by Mosher and Dawkins (2000).

2.3 Modelling of *q-z* springs

The *q-z* relation consists of a single spring located at the tip of the pile to simulate the tip resistance. Required definitions are the ultimate tip resistance (q_u) and the displacement at which 50% of the ultimate resistance is mobilized in monotonic loading (z_{50}). Meyerhof (1976) provides formulae for the extraction of q_u applicable to cohesionless soils through the rigidity index (I_r) of the soil and the bearing capacity factor (N_q).

In satisfying OpenSees requirements, the tip resistance is then computed as a force by multiplying the tip stress by the cross sectional area of the pile. Computation of z_{50} utilizes the *q-z* curve by Vijayvergiya (1977). The critical tip deflection (z_c) is evaluated as a percentage of the pile diameter; the deflection at 5% of the diameter has been adopted.

2.4 Modelling of pile foundation

Modelling of the physical pile can be achieved via elastic sections (sections defined by material and geometric constraints) consisting of nodes adjoining displacement-based nonlinear beam-column elements. The following properties are required for definition of the section:

- Young’s Modulus
- Cross-sectional area
- Second moment of inertia about the local *z* axis
- Second moment of inertia about the local *y* axis (not necessary for 2D analyses)
- Shear modulus (not necessary for 2D analyses)
- Torsional moment of inertia (not necessary for 2D analyses)

In reflecting a 2D analysis, pile nodes are only allowed to translate in the *x-z* plane while allowing rotation to occur about the *y* axis. Additionally, if yielding or cracking is expected to occur, a moment-curvature (*M-Φ*) relationship can be defined for an inelastic pile section in simulating its response to liquefaction effects.

3 MODEL VALIDATION

3.1 Vertical pile subject to inertial loading

Implementation of the FE model commences with the simulation of a vertical pile in saturated sand subjected to a lateral load at the pile head. This aims at validating the accuracy of the backbone *p-y* curve which will be crucial for subsequent simulations of liquefaction and lateral spreading.

Centrifuge tests conducted by Liu and Dobry (1995) on a steel pipe measuring 6.7m (22ft) length with a diameter of 0.381m (15in) form the basis of experimental testing. The pile was subjected to a lateral load ranging from 116kN (26kip) to 251kN (56.5kip) at the head.

The experiment was simulated for pile head loads of 116kN and 251kN within OpenSees. The soil was assumed a submerged unit weight of 10kN/m^3 with an angle of internal friction of 30^o. Figure 2 shows theoretical results produced from OpenSees. It can be seen that the overall shape of the bending moment curve is adequately captured in both cases with the location of the maximum negative moment being represented fairly accurately via the FE model. Moment magnitudes produced theoretically for the 116kN case are also consistent with that of its experimental counterpart. However, the model slightly overestimated the maximum negative moment for the 251kN case.

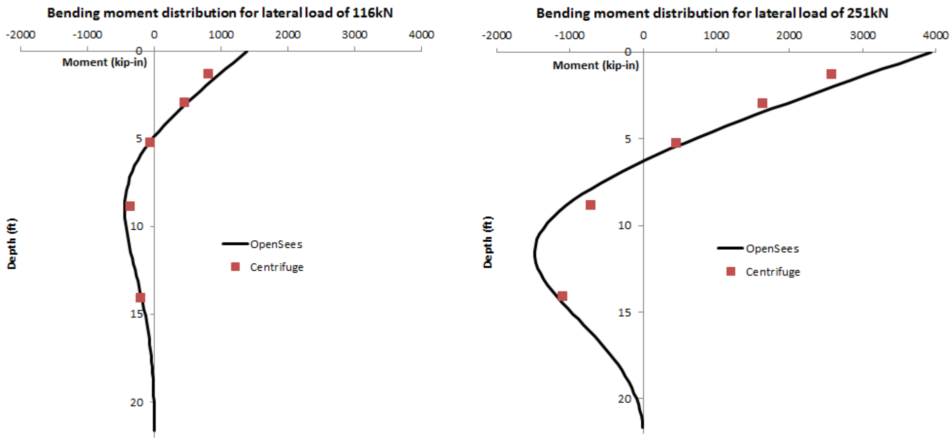

Figure 2. Bending moment distribution for (a) lateral load of 116kN (26kip); and (b) 251 kN (56.5 kip) computed via OpenSees. The experimental results were extracted from Liu and Dobry (1995).

3.2 Vertical pile subject to lateral spreading

The 1995 Kobe earthquake resulted in an oil storage tank supported by 69 concrete piles experiencing permanent displacements due to lateral spreading of the ground of up to 55cm. The piles measure 45cm in diameter and extend 23 meters into the ground. One such pile in particular located on the rim was displaced by 40cm of which was extensively studied by Ishihara and Cubrinovski (2004). Additionally, the pile response for a free-field displacement of 30cm has also been investigated. An equivalent linear analysis procedure was adopted by Ishihara and Cubrinovski (2004) in investigation of the pile giving displacements and bending moments along its depth for both ground displacement cases (Figure 3).

The soil profile consists of a 2.5m surface layer overlaying a deposit of liquefiable sand 11m thick. The response of the liquefied soil is characterized via an equivalent linear $p-\delta$ function subject to a degradation factor, β of 0.01 and 0.001. This pressure – displacement relationship is held constant throughout the liquefied layer. The distribution of lateral spreading is held constant within the unliquefiable crust while following that of a cosine curve throughout the liquefied layer.

The concrete pile section was represented via a tri-linear M-Φ relationship typical to that of a pre-stressed high-strength concrete pile.

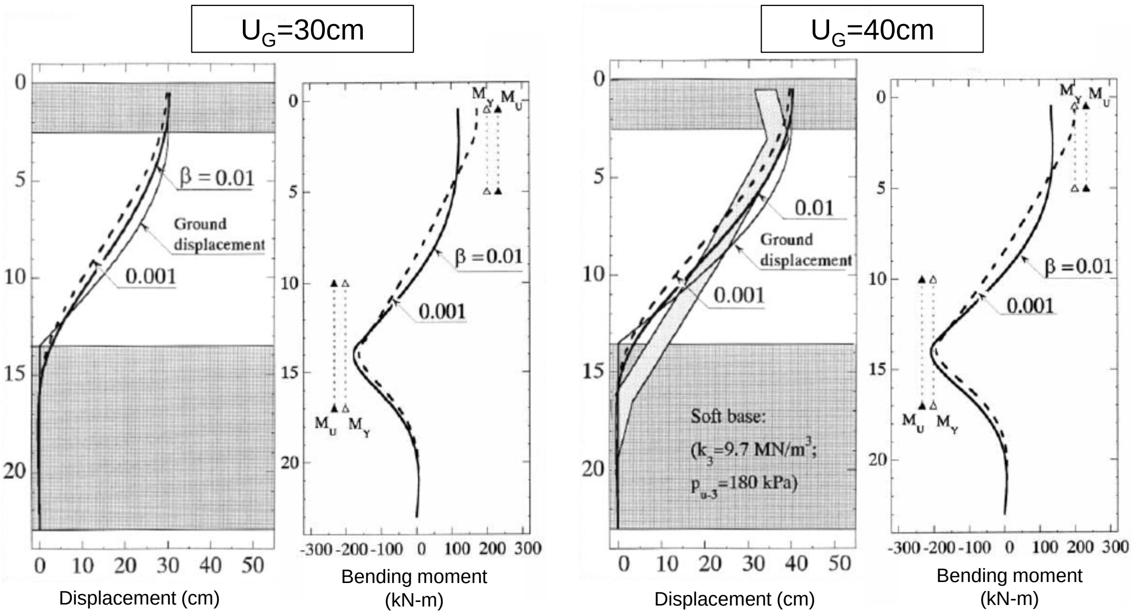

Figure 3. Displacement and bending moment distribution computed via an equivalent linear method for free-field soil deflections of 30cm and 40cm (extracted from Ishihara and Cubrinovski, 2004).

The manifestation of lateral spreading is implemented within the FE model via two different loading profiles. The first replicates that adopted by Finn and Fujita (2002) which constitutes of an upper unliquefiable layer subject to a uniform lateral displacement along its depth. This displacement then reduces linearly down to zero through the liquefiable layer underlying the top stratum. Additionally, a displacement profile following a cosine curve has also been implemented for the simulation of lateral spreading within the liquefied layer. This was the profile considered by Ishihara and Cubrinovski (2004) and also used in a similar study by Bowen and Cubrinovski (2008). The profiles are shown in Figure 4 which is compared to assess their relative significance on the overall pile response.

The strength of the liquefied soil was modified such that it corresponds to a degradation factor β=0.01, as obtained by Ishihara and Cubrinovski (2004). All soil deflections are applied to fixed soil nodes connecting onto *p-y* springs. The displacements were applied via single point constraints and incrementally increased across 20 stages for numerical stability in forming the static analysis.

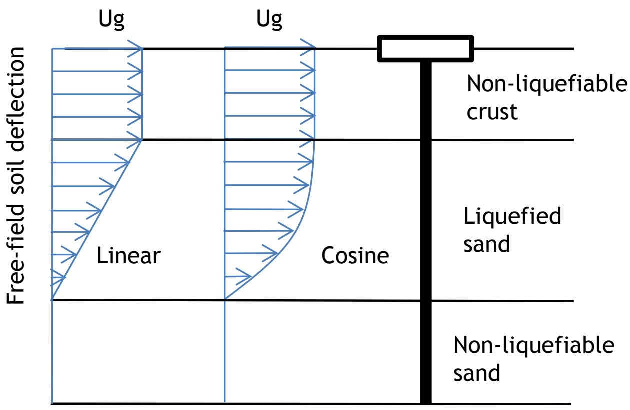

Figure 4. Lateral spreading free-field displacement profiles adopted for use within OpenSees.

As the maximum bending moment computed by Ishihara and Cubrinovski (2004) is approximately 200kNm, a bi-linear moment-curvature relationship was adopted in approximating the tri-linear curve. In this sense, only the first two linear sections of the *M-Φ* plot were considered. The unit weight of liquefied soil is approximated at roughly twice that of water (Tokimatsu, 2007). Deflection and moment distributions as computed through OpenSees for free-field displacements of 30cm and 40cm adopting both lateral spreading profiles are shown in Figure 5.

It can be seen that for both free-field ground displacement magnitudes for the cosine profile, the pile displacement and moment profiles match results obtained by Ishihara and Cubrinovski (2004) for a degradation factor of 0.01 relatively well. However, the response from OpenSees appears to see the moment return to zero near the base of the liquefied layer at a shallower depth than that computed via the equivalent linear method.

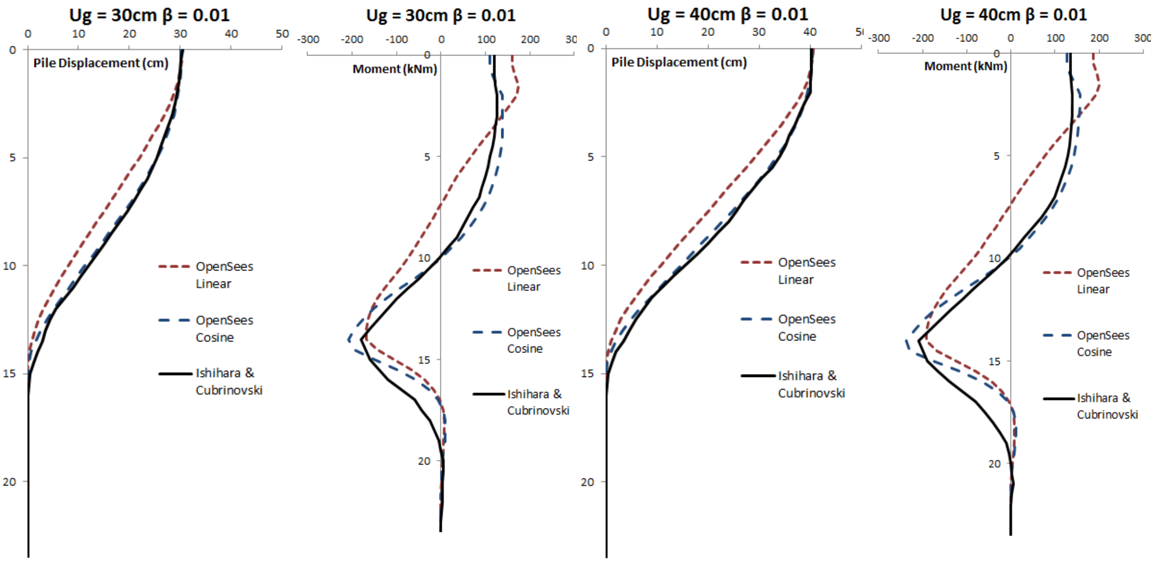

Figure 5. Displacement and bending moment distribution calculated via FE analysis using OpenSees for free-field soil deflections of 30cm and 40cm adopting both linear and cosine displacement profiles.

On the other hand, use of a linear lateral spreading displacement distribution produced significant differences in both pile displacement and bending moment. Displacement of the pile is mostly linear through the liquefied layer which is also of reduced magnitude relative to the cosine case. In terms of the bending moment, it appears that a linear spreading profile produces a higher positive moment at the pile head while giving a slightly lower maximum negative moment. It is also interesting to note that a linear profile generate displacements and bending moments more closely related to that produced via the equivalent linear method for a β = 0.01.

4 DISCUSSION

4.1 Model validation

From experimental data, the FE model has been shown to be reliable in predicting the response of vertical piles in non-liquefying ground. Adopting the Brinch-Hansen relation for definition of the ultimate lateral resistance along with corrected API values for initial stiffness thus proved adequate in defining a feasible backbone *p-y* curve representing soil behaviour.

When investigating pile response under lateral spreading, the model appears to give consistent results with that derived via the equivalent linear analysis procedure. Reasons for why the negative moment near the base of the liquefied layer reaches zero at a slightly shallower depth for the OpenSees model is likely attributed to the consideration of resistance and stiffness degradation about the boundaries of the liquefied layer. As the equivalent linear method does not take this into account, the resistance near the vicinity of the liquefied deposit is likely higher than that predicted using the FE model. The *p-y* relation defined according to the Brinch-Hansen and modified API procedure also observes an increase in the ultimate lateral resistance with depth. As this relation is effectively constant for the method adopted by Ishihara and Cubrinovski (2004) in any given soil layer, minor discrepancies between the two analyses are expected.

Use of a bi-linear *M-Φ* relation representing only up to the yield point of the concrete pile proved adequate for this case. This is most likely due to the fact that the moments experienced by the pile did not greatly exceed the yield range. Hence, it was not critical to define the third linear gradient leading to the ultimate moment. However, if free-field displacements of greater magnitude were to be analysed, the FE model would likely overestimate the bending moment as the pile curvature extends past its point of yielding; this case would indicate pile breakage.

4.2 Extension to raked piles

After full-validation, the model developed has been extended to analyse the response of inclined/raked piles. Analysis of centrifuge tests involving single inclined piles and actual case histories such as the Landing Road Bridge response during the 1987 Edgecumbe earthquake (Wang and Orense, 2013) successfully demonstrated excellent performance of the model.

4.3 Limitations

Several limitations surround the proposed FE model in evaluating the performance of pile foundations. Firstly, the validity of vertical springs has not been fully justified through tests involving purely axial loads.

5 CONCLUDING REMARKS

A 2D BNWF finite element model to be implemented via OpenSees has been created. This allows for the simulation of pile foundations subject to the effects of liquefaction. The model has been compared against experimental data and with an alternative equivalent linear analysis procedure. The model can be used to analyse different scenarios involving piles subjected to laterally spreading soils.

The Brinch Hansen p_u curve combined with the modified API distribution for k_T proved capable in simulating the behaviour of non-liquefied soil. The effects of lateral spreading due to soil liquefaction can be adequately incorporated using stiffness and resistance reduction factors. The free field soil deflection profile selected for the liquefied layer greatly affects the distribution of pile displacement and bending moment. Use of a linear profile produces lower deflections along the depth of the pile while giving larger moments at the pile head.

6 REFERENCES

American Petroleum Institute, A.P.I. (1987). Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms. *API Recommended Practice 2A (RP-2A)*. Washington D.C.

Bowen, H. J. & Cubrinovski, M. (2008). Pseudo-static analysis of piles in liquefiable soils: Parametric evaluation of liquefied layer properties. *Bulletin of the New Zealand Society for Earthquake Engineering*, 41.

Finn, W. D. L. & Fujita, N. (2002). Piles in liquefiable soils: Seismic analysis and design issues. *Soil Dynamics and Earthquake Engineering*, 22, 731-742.

Hansen, J. B. & Christensen, N. H. (1961). The ultimate resistance of rigid piles against transversal forces. *Bulletin No. 12*. Geoteknisk Institute, Copenhagen.

Ishihara, K. & Cubrinovski, M. (2004). Case studies of pile foundations undergoing lateral spreading in liquefied deposits. *Fifth International Conference on Case Histories in Geotechnical Engineering*. New York, NY.

Kulhawy, F. H. (1991). Drilled Shaft Foundations. In: FANG, H.-Y. (ed.) *Foundation Engineering Handbook,* 2nd Edition. Van Nostrand Reinhold, New York.

Liu, L. & Dobry, R. (1995). Effect of liquefaction on lateral response of piles by centrifuge model tests. *NCEER Bulletin*, January 1995.

Mazzoni, S., McKenna, F., & Fenves, G. L. (2011). *OpenSees getting started manual*. http://opensees.berkeley.edu/wiki/index.php/Getting_Started.

McGann, C. R., Arduino, P. & Mackenzie-Helnwein, P. (2011). Applicability of conventional p-y relations to the analysis of piles in laterally spreading soil. *Journal of Geotechnical and Geoenvironmental Engineering*, 137, 557-567.

McGann, C. R., Arduino, P. & Mackenzie-Helnwein, P. (2012). Simplified procedure to account for a weaker soil layer in lateral load analysis of single piles. *Journal of Geotechnical and Geoenvironmental Engineering*, 138, 1129-1137.

Meyerhof, G. G. (1976). Bearing capacity and settlement of pile foundations. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 102, 195-228.

Mosher, R. L. & Dawkins, W. P. (2000). *Theoretical Manual for Pile Foundations*. Washington D.C.: US Army Corps of Engineers, Engineer Research and Development Center.

Tokimatsu, K. (2007). Earthquake Resistant Bases and Foundations, Chap 4. In: Horikawa, K. (ed.) *Civil Engineering*, Vol 1. Encyclopedia of Life Support Systems (EOLSS).

Vijayvergiya, V. N. (1977). Load-movement characteristics of piles. *Proc., Ports 77 Conference*. New York

Wang, S. & Orense, R.P. (2013). Modelling of raked pile foundations in liquefiable ground. *Soil Dynamics and Earthquake Engineering* (submitted).