Effective stress analysis of pile foundations in liquefiable soil

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ABSTRACT: An advanced dynamic analysis based on the effective stress principle is used to evaluate the seismic performance of foundation piles of a bridge pier in Christchurch. The employed method permits accurate simulation of the ground response in liquefying soils including the process of excess pore pressure build-up and associated highly-nonlinear stress-strain behaviour of soils.

In the analysis, complete liquefaction developed from 11 m to 17 m depth resulting in lateral ground displacements of about 28 cm and consequent damage to the pile at the pile head where the peak bending moment exceeded the yield level. Characteristics of the ground response and behaviour of piles are discussed using computed time histories and maximum values of accelerations, displacements, excess pore water pressures and pile bending moments. Results of the effective stress analysis are further examined through comparisons with a conventional analysis based on the pseudo-static approach.

1 INTRODUCTION

Soil liquefaction during earthquakes can cause very large loads on pile foundations including large lateral ground displacements and inertial loads from the superstructure. Since piles are primarily designed to carry axial loads, not resist bending forces, these large lateral loads make piles highly vulnerable to damage due to liquefaction. The extensive damage and failure of piles have caused numerous failures of bridges, buildings and storage tanks in the previous earthquakes (Hamada and O'Rourke 1992; Japanese Geotechnical Society 1998; Yasuda and Berrill 2000).

The seismic design philosophy of pile foundations in liquefied soil is based on two levels of treatment: a simplified approach based on empirical and pseudo-static methods, and detailed dynamic analysis using the time history or step-by-step procedure. The former approach is appropriate for preliminary assessment and design of piles, while the latter is suitable for performance based assessment of important structures. This paper discusses the application and relevance of detailed time history analysis based on the effective stress principle, with reference to a case study.

2 FITZGERALD AVENUE BRIDGE

The case study is of twin bridges crossing the Avon River in Christchurch, New Zealand. The bridge has been identified as an important lifeline, and a structural retrofit has been proposed by the Christchurch City Council to reduce the risk of failure in an anticipated earthquake event. Considered in this paper is the central pier of the east bridge, subject to shaking in the east-west (transverse) direction.

The existing bridge pier is founded on eight reinforced concrete piles, 0.3m in diameter and 9m long. It is proposed that in conjunction with the widening of the bridge, two steel encased reinforced concrete piles, 1.5m in diameter and 20m long, will be installed at each end of the pier, as shown in Figure 1. The soil profile at the site is quite complex and highly variable between the abutments and central pier both in stratification and strength of the soil. The weakest soil profile obtained from detailed in situ investigations was conservatively adopted for this analysis, as shown in Figure 1b. Here, the soil between 2.5m and 17.5m depth consists of liquefiable layers of sandy gravel and silty sand, with a relatively dense sand base layer below a depth of 17.5m.

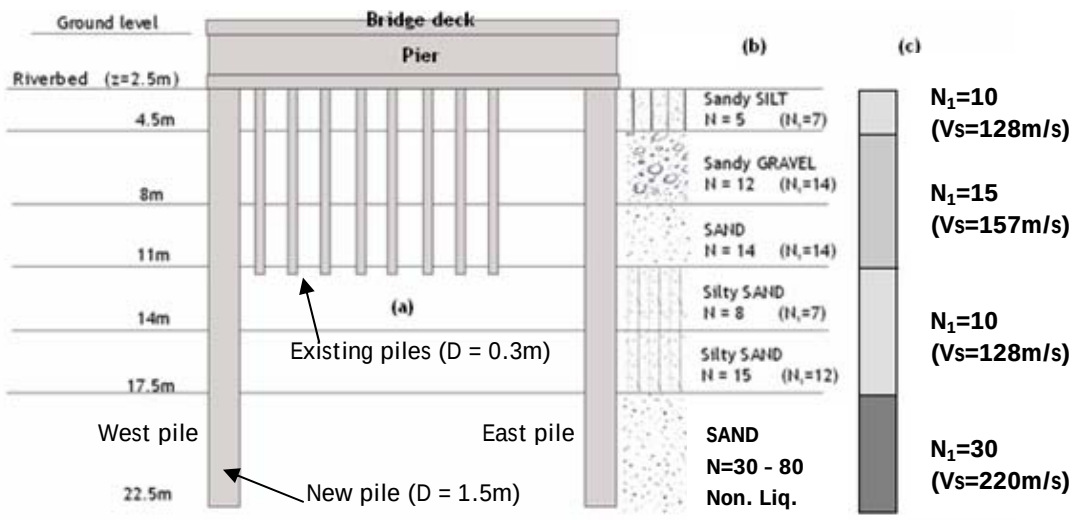

Figure 1. Central pier of bridge: (a) Foundation layout, (b) adopted soil profile, (c) soil properties used in analysis

3 EFFECTIVE STRESS ANALYSIS

A fully coupled effective stress method was used to analyze the soil-pile-bridge system. This is an advanced analysis that permits consideration of excess pore water pressure, flow of pore water through the soil and detailed modelling of the stress-strain behaviour of soils. The accuracy of the analysis has been extensively verified through case studies (Cubrinovski et al. 2001) and large-scale shake table tests (Cubrinovski et al. 2005). The 2-D numerical model used in this study is shown in Figure 2, where solid elements are employed for the soil and bridge superstructure, while beam elements are used for the piles and footing. The model is 160m x 30m in size. The piles were modelled as nonlinear members with a moment-curvature relationship approximated using the hyperbolic model. The footing, bridge deck and pier were all modelled as elastic materials with an appropriate tributary mass.

The soil elements were modelled as two phase solid elements using an advanced constitutive model, which uses an elasto-plastic deformation law for sandy soils (Cubrinovski and Ishihara 1998). This constitutive model requires a set of parameters to be determined regarding the initial stiffness, stress-strain relationship and liquefaction resistance of the soil. In the absence of detailed laboratory tests, however, input parameters for the model were determined by modifying the parameters of Toyoura sand in order to fit the liquefaction resistance at N_C = 20 cycles, as indicated with the symbols in Figure 3. This resistance was adopted using conventional empirical charts based on SPT blow count.

For the purposes of the analysis, the soil profile was further simplified, as shown in Figure 1c. Thus, the liquefaction resistance curves shown in Figure 3 correspond to the cyclic strength of the layers with normalized SPT blow count of N_1 = 10 and N_1 = 15, as simulated by the constitutive model. The non-liquefiable soil was modelled as equivalent linear soil with a shear modulus degraded to 30% of its initial value, a level chosen with regard to the density of the soil and its anticipated response.

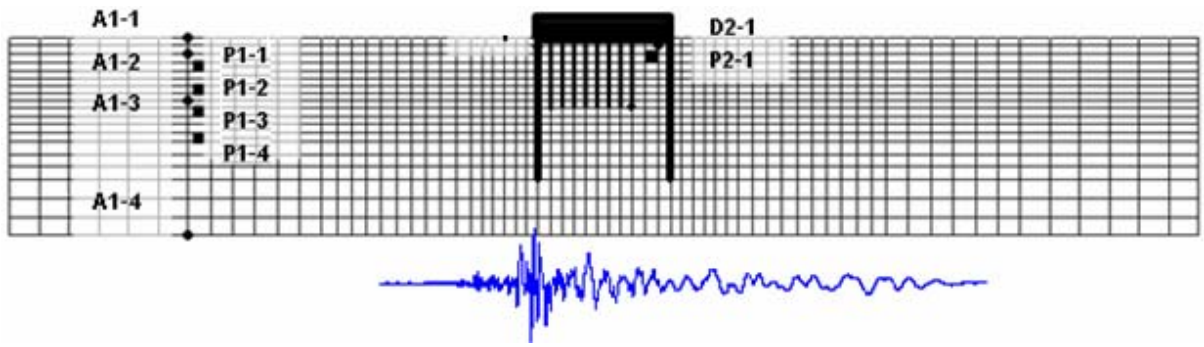

Figure 2. Numerical model and input motion used in the analysis

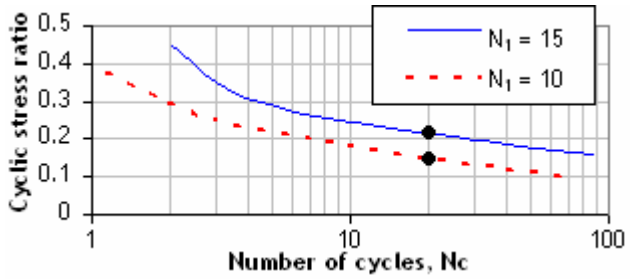

Figure 3. Liquefaction resistance curves for N_1 =10 and N_1 =15 layers.

Previous seismic hazard studies for Christchurch (Stirling et al. 2001) indicate that the most significant ground shaking would occur in a magnitude 7.2-7.4 earthquake located at a distance of about 40-60km away. Stirling et al. (2001) gives a peak ground acceleration value of 0.37g in a 475yr event and 0.47g in a 1000yr event; a peak ground acceleration of 0.44g was specified for the bridge using the loadings standard NZS1170.5. Having all of these in mind, a ground motion with similar attributes as above recorded at a depth of 25m in a downhole array during the 1995 Kobe earthquake (M=7.2) was used as a base input motion in the analysis, scaled to a peak acceleration of 0.4g.

4 RESULTS

4.1 Free-field response of the ground

Figure 4a shows computed excess pore water pressure time histories at different depths throughout the soil profile. It can be seen that although liquefaction eventually occurs in all layers, the pore pressure development is quite different for the two deeper layers with N_1 = 10 and N_1 = 15 respectively. In the weaker layer (N_1=10) liquefaction occurs straight after the first cycle of strong shaking while in the stronger layer (N_1=15) the excess pore pressures build up gradually. Figure 4b illustrates this feature, giving snapshots of the extent of liquefaction at different stages of the shaking. Note that here the pore pressure ratio of unity indicates complete liquefaction.

Effects of liquefaction on the ground response are evident in Figure 5 where acceleration time histories at three different depths are shown. Following the complete liquefaction in the mid layer at about 13-14 seconds, the accelerations above the liquefied layer decrease significantly and the ground motion at the surface shows elongation of the vibration period and loss of high frequencies. This diminished ground shaking and consequent reduction in the shear stress can explain the slower gradual build-up of the excess pore water pressure in the layers above the liquefied layer.

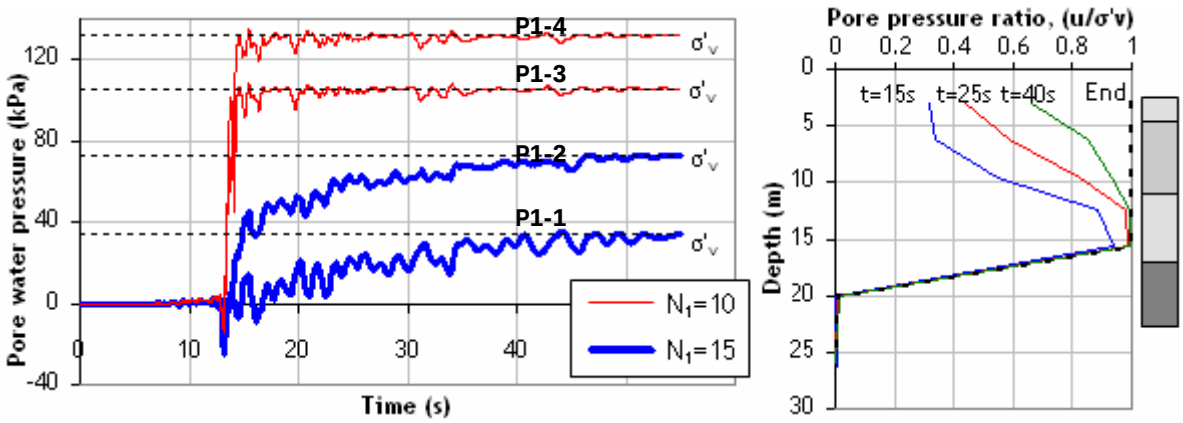

Figure 4. Free field excess pore water pressures; (a) time histories, showing the progression of liquefaction at different depths, (b) excess pore water pressure ratio as a function of depth at different times of the response

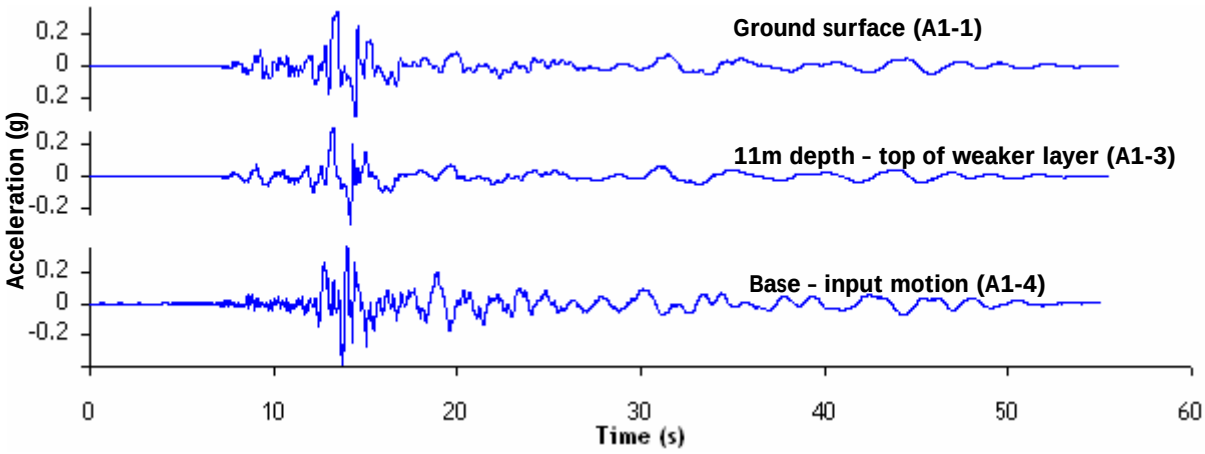

Figure 5. Acceleration time histories for different depths of the soil profile

The maximum values of the ground acceleration, shear strain and ground displacement, plotted in Figure 6, clearly display the effects of liquefaction on the free field ground response. Figure 6a shows an amplification of the acceleration between 17.5m and 30m depth, then a marked decrease within the mid liquefied layer. The amplification is partly due to the modelling of the base layer as an equivalent linear material with no hysteretic damping and relatively low numerical damping. The decrease in acceleration above the mid liquefied layer is expected, this phenomenon has been observed in downhole arrays during the 1995 Kobe earthquake and in many experimental tests. Figures 6b and 6c show that the majority of the ground deformation occurs in the mid layer with N_1=10, where the peak shear strains reach about 4-5%. The strains in the shallow part of the deposit are generally below 1%. This is reasonable as the effects of shaking above this layer have been diminished as previously described.

4.2 Response of the foundation soil

In general terms, the pile foundation provides a stiffening effect to the surrounding soil. This is illustrated in Figure 7, which compares the ground response at the free field and in between the piles. Despite large increases in pore water pressure, the soil retains some stiffness and the fluid-like behaviour of the free field with filtering effects is not observed in Figure 7b. The stiffening effect of the foundation on the response of the soil is further shown in Figure 8, with a large decrease in ground displacement for soil amongst the piles.

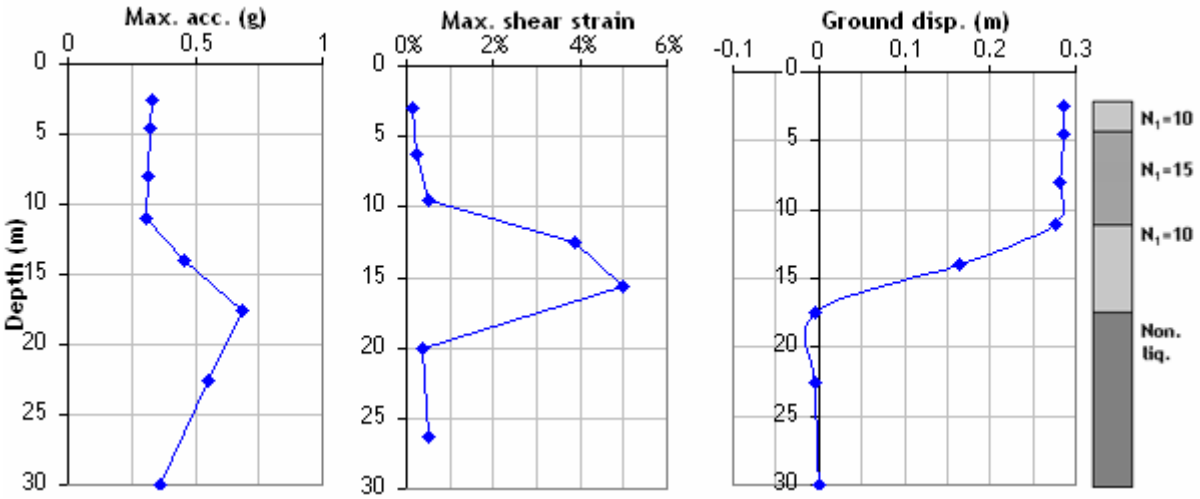

Figure 6. Maximum free field response: (a) accelerations, (b) shear strains, (c) ground displacements

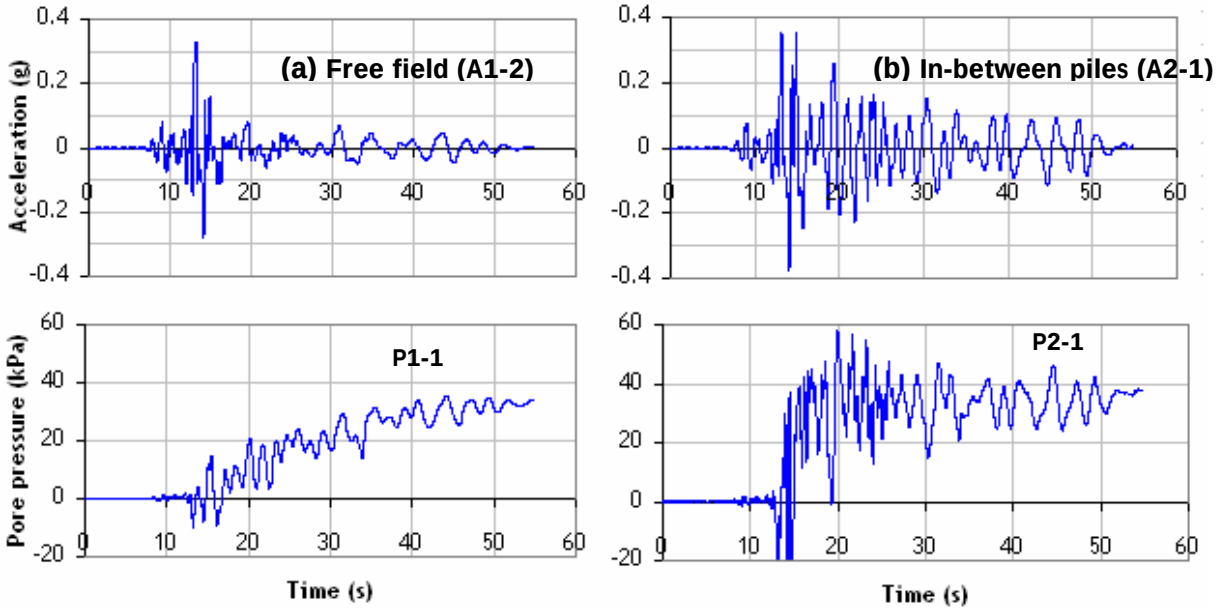

Figure 7. Time histories of acceleration and excess pore water pressure in the stronger liquefied soil (N1=15 at z =5m) for locations (a) in the free field; and (b) in between the piles

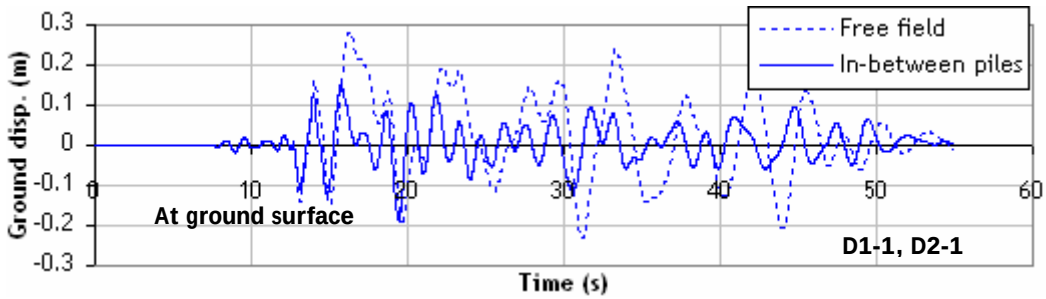

Figure 8. Comparison of the displacement at the ground surface for locations in the free field and in between the piles

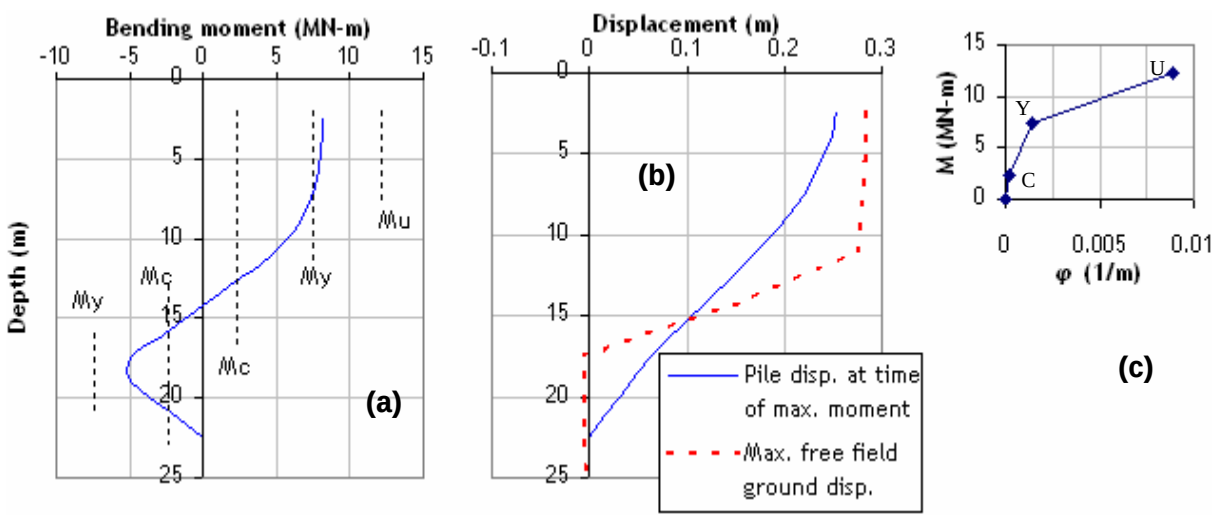

Figure 9. Computed pile response (a) Maximum pile bending moment, (b) pile displacement profile at the time of the maximum moment, (c) tri-linear M-ϕ relationship for the pile

4.3 Pile response

Figure 9a shows the bending moment distribution with depth for the west pile, plotted for the time when the maximum bending moment was reached at the pile head. The pile exhibits behaviour typical

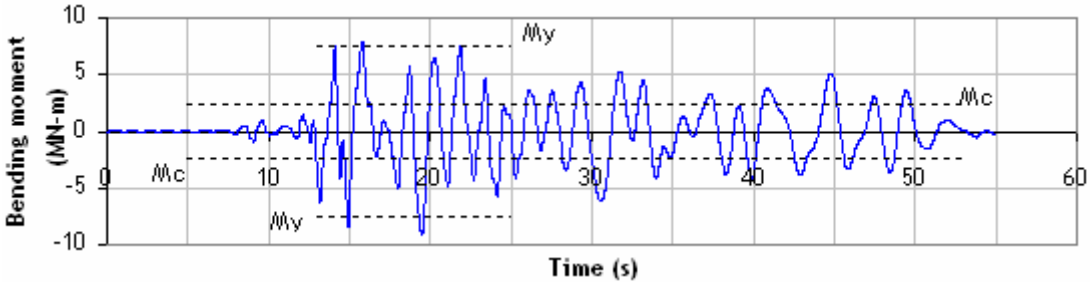

Figure 10. Bending moment time history at the pile head

of piles in liquefied soils, with the largest bending moments occurring at the pile head and at the interface between the liquefied and base layers. It can be seen that the bending moment exceeds the pile cracking moment along most of the pile length, with the largest damage and yielding occurring in the top 5m of the pile. Figure 9b shows the displacement of the pile relative to the free field ground displacement, indicating that below 15m depth the pile is pushing the soil, whereas above 15m the soil is pushing the pile in the direction of movement. Figure 10 shows a time history of the bending moment at the pile head for the west pile.

5 COMPARSION WITH PSUEDO STATIC ANALYSIS

It is interesting to compare the results of the effective stress analysis with those obtained using a more conventional pseudo-static approach. In the latter approach, the complex dynamic forces are approximated by two static loads. Kinematic loads from the soil movement are applied through free-field ground displacements acting on a series of soil springs, as illustrated in Figure 11. In addition, inertial loads from the superstructure are modelled with a lateral force applied to the pile head. The stiffness of the soil springs, ultimate pressure from the soil and the free field ground displacement are calculated using simple empirical methods based on SPT blow counts (e.g. Cubrinovski et al. 2007).

Figure 12 compares the free field ground response predicted by the effective stress analysis to that predicted by a simplified empirical method. In Figure 12a, the maximum cyclic shear strains from the effective stress analysis are compared to the values obtained from a simple correlation with SPT blow count (Tokimatsu and Asaka 1998). In the simplified procedure, the free field ground displacement that is applied to the pile is calculated by integrating the shear strains throughout the soil profile. Figure 12b shows that this approach is conservative when compared to the more rigorous effective stress analysis, predicting a much larger displacement at the pile head. In the effective stress analysis, large strains only occur in the mid liquefied layer with N_1 =10, so most of the ground deformation occurs in this layer. The simplified procedure is unable to capture these complex characteristics of the response.

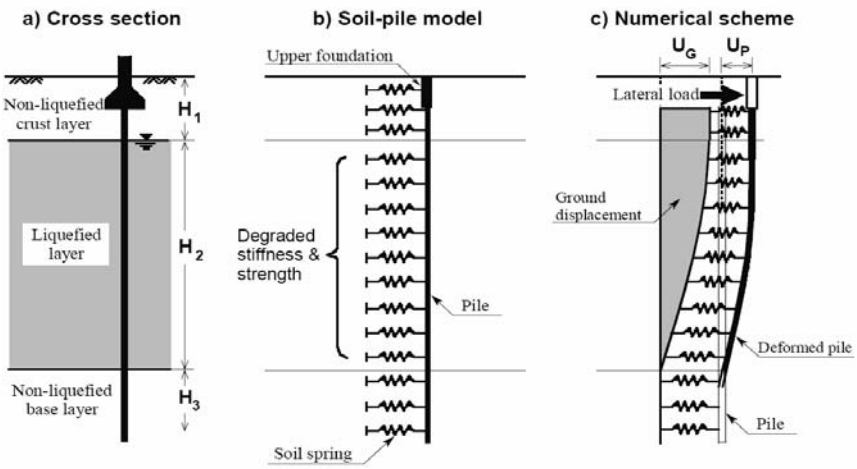

Figure 11. Simplified psuedo-static analytical model

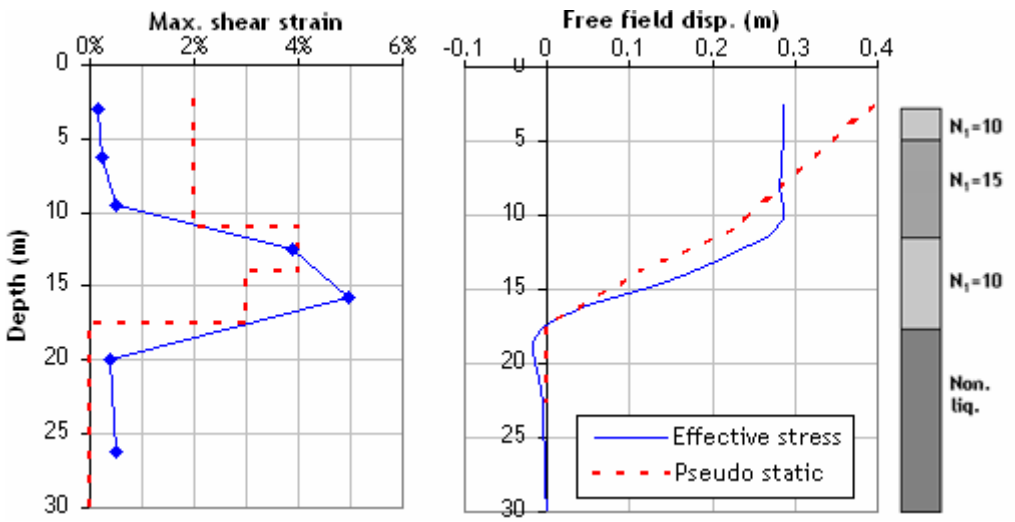

Figure 12. Comparison between the free field ground response calculated from the effective stress and psuedo static methods; (a) maximum cyclic shear strain, (b) ground displacement

Due to the uncertainty regarding the stiffness of liquefied soils, two simplified analyses were performed, one with a relatively stiff liquefied soil and one with relatively weak liquefied soil stiffness. The case with relatively stiff liquefied soil was analysed as having the stiffness in the liquefied layers degradation by a factor of 1/20, while the relatively weak liquefied soil case used a factor of 1/50. Both analyses used an inertial load corresponding to 0.44g ground acceleration.

The pile behaviour predicted by both simplified analysis cases and the effective stress analysis is compared in Figure 13. The maximum bending moment distributions are compared in Figure 13a, and it can be seen that the distribution predicted by the effective stress analysis shows different characteristics to the pseudo static results. The effective stress analysis results indicate that the bending moment at the interface between the liquefied and base soil layers is much lower, and the bending moment flattens out above the mid liquefied layer. The first observation is partly due to the use of an equivalent linear soil stiffness for the base layer, where the shear modulus was degraded to 30% of its initial value. Thus the base soil for the effective stress analysis has lower stiffness than in the pseudo static approach, resulting in less of a contrast in stiffness between the liquefied and base layers and hence lower bending moment. The second observation is also expected as the effects of widespread liquefaction occurring in the mid layer before the layers above cannot be captured in the pseudo static analysis.

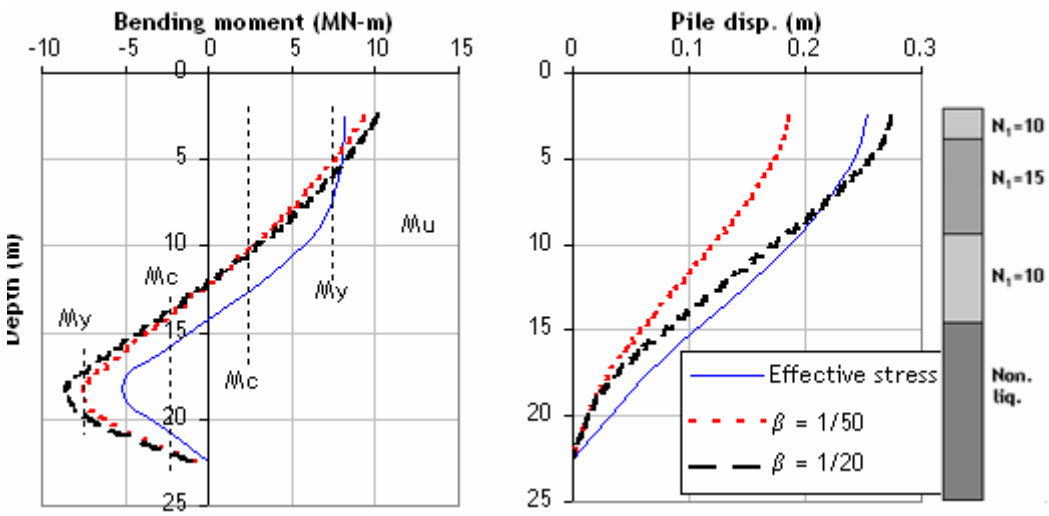

Figure 13. Comparison of pile behaviour between the two methods; (a) maximum bending moment distribution. (b) maximum pile displacement

Figure 13b shows that the pile displacement profile is different between the two methods. The pile is more flexible at the base in the effective stress analysis, due to the weaker base layer as described above. The displacement at the pile head predicted by the pseudo static varies considerably as the stiffness of the liquefied soil is varied. The more rigorous effective stress analysis predicts a displacement in between the upper and lower bounds predicted in the simplified analysis. By and large, the results of the effective stress analysis and pseudo-static analysis are in good agreement and consistent with the assumptions made and details of modelling.

6 CONCLUSIONS

An advanced dynamic analysis based on the effective stress principle has been performed to evaluate the seismic performance of foundation piles of a bridge pier founded in liquefiable soils. This case study demonstrated the capability of the effective stress analysis to capture important features of the complex soil-pile interaction in liquefying soils including:

- Detailed development of excess pore water pressure through time and space including effects of soil density and complex interaction between intensity of shaking, pore pressures and associated ground deformation. Typical effects of liquefaction on the ground motion such as loss of high-frequency content and elongation of the period were also observed.
- The soil-pile interaction significantly affected both the response of the foundation soil and piles. The presence of piles increased the stiffness of the foundation soil and consequently reduced its deformability as compared to the free field ground. The peak ground displacements were about 18 cm and 28 cm in the soil in-between piles and the free field soil respectively.
- The seismic performance of piles was rigorously evaluated by taking into account the highly complex dynamic nature of loads and soil-pile interaction. The horizontal displacement of the piles reached about 25 cm and bending moments reached yield level at the top of the pile. Hence, the analysis provided very detailed information on the performance of the piles including development, variation and duration of loads and consequent damage level to piles.

For the above reasons, the advanced effective stress analysis is suitable for a rigorous evaluation of the seismic performance of pile foundations of important structures. It can explain complex features of the response and verify design assumptions, and hence, it provides confidence in the design of the piles.

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