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Estimation of liquefaction induced settlement from computed seismic site response

L. Luo, J. Lu & A. Elgamal
Department of Structural Engineering, UC San Diego, La Jolla, CA, USA

M. Fraser
NAVFAC Southwest, San Diego, CA, USA

ABSTRACT: Post-liquefaction settlement of saturated soils can be estimated from peak shear strains using empirical methods. This paper presents a procedure for estimating settlement where the employed peak shear strains are obtained from Finite Element (FE) analyses. In this study, FE simulations were conducted using Cyclic1D, a PC-based graphical pre- and post-processor (user-interface) for execution of site response simulations including the analysis of liquefaction-induced lateral deformations (in a mildly inclined infinite slope). The liquefaction model employed in Cyclic1D is formulated within the framework of multi-yield-surface plasticity, where emphasis is placed on controlling the magnitude of cycle-by-cycle permanent shear strain accumulation in clean medium to dense sands. A parametric study was also undertaken to illustrate the effect of changing soil material properties on post-liquefaction settlement.

1 INTRODUCTION

Seismic site response analysis (Kramer 1996) is usually conducted with an equivalent linear model (e.g. Schnabel et al. 1972, Idriss and Sun 1992, Matasovic 1993), a cyclic nonlinear model, or an advanced constitutive model (e.g. Elgamal et al. 2006, Hashash et al. 2016). For liquefaction scenarios, an effective stress-based soil constitutive model is preferred (Kramer 1996, Yang et al. 2004 and Elgamal et al. 2006). However, in these modeling procedures, effort has been primarily focused on analysis of the cyclic stress-strain response; relatively little attention has been given so far to capture the associated liquefaction-induced settlement.

In this paper, a practical engineering procedure is presented to estimate liquefaction-induced site settlement. For that purpose, the approach of Ishihara and Yoshimine (1992) that relates peak shear strain to volume change was employed. The peak shear strains employed were obtained from ground response analyses conducted using the nonlinear Finite Element (FE) program Cyclic1D (http://www.soilquake.net/cyclic1d/).

It is noteworthy to mention that simplified analysis techniques (e.g. Idriss and Boulanger 2008), incorporated the use of the Ishihara and Yoshimine (1992) approach to estimate site settlement. In this regard, a Factor of Safety lower than 1.0 against liquefaction for any stratum is employed. However, this approach is conducted on a layer by layer basis, and thus does not include the impact of liquefaction of an underlying layer on potential reduction in shear energy in the overlaying strata (Kokusho 2013). As such, overly conservative results may be obtained (Martin et al. 1991), with potential expensive mitigation recommended, that might not be actually warranted.

In the following sections, the analysis framework of Cyclic1D is presented, followed by the settlement estimation procedure. Representative cases are presented to illustrate the influence of site stratification considerations on the resulting settlement. In particular, influence of a low shear strength seam within the soil stratum on the liquefaction-induced volume change profile and extent of the overall site settlement is highlighted.

2 COMPUTATIONAL FRAMEWORK

Cyclic1D is a nonlinear FE program for conducting one-dimensional (1D) lateral dynamic site-response simulations. The program operates in the time domain, allowing for linear and nonlinear studies (Hughes 1987). Nonlinearity is simulated by incremental plasticity models to allow for modeling permanent deformation and for generation of hysteretic damping. A solid-fluid (u-p) fully-coupled FE formulation (Chan 1988, Ziekiewicz et al. 1990) is employed for saturated soil analysis. Cyclic 1D is Readily available at http://www.soilquake.net/cyclic1d/.

Dry and/or saturated soil profiles may be studied. In saturated cohesionless soil strata, liquefaction and its effects on ground acceleration and permanent deformation are modeled. In this regard, the user may wish to explore the response of a level ground site, or conversely to investigate the response of a mildly-inclined infinite-slope site.

The liquefaction model (Figure 1) employed in Cyclic1D (Parra 1996, Yang 2000) is developed within the framework of multi-yield-surface plasticity (e.g., Prevost 1985). In this model, emphasis is placed on controlling the magnitude of cycle-by-cycle permanent shear strain accumulation in clean medium to dense sands (Parra 1996, Yang 2000, Elgamal et al. 2002, 2003, Yang et al. 2003a). Furthermore, appropriate loading-unloading flow rules were devised to reproduce the observed strong dilation tendency, resulting increase in cyclic shear stiffness and strength (the "Cyclic Mobility" mechanism). Model parameters controlling shear–volume coupling effects (dilatancy) were calibrated using the solid–fluid fully coupled FE program Cyclic1D in conjunction with an advanced numerical optimization code (Yang and Elgamal 2003b).

3 POST-SHAKING SETTLEMENT ESTIMATION PROCEDURE

Evaluation of post-liquefaction settlement is important for the analysis of seismic ground response. Previous research has been conducted to relate liquefaction-induced settlement to shear wave velocity (Ishihara and Yoshimine 1992; Yoshimine et al. 2006 and Yi 2010).

The procedure to estimate post-shaking settlement in Cyclic1D is as follows:

Step 1: Calculate the corrected shear wave velocity V_{s1}

Shear wave velocity V_s increases with increasing effective confining stress. Idriss and Boulanger (2008) introduced a corrected value that would be obtained in the identical sand if the initial vertical effective stress is 1 atm (100 kPa). The corrected shear wave velocity is:

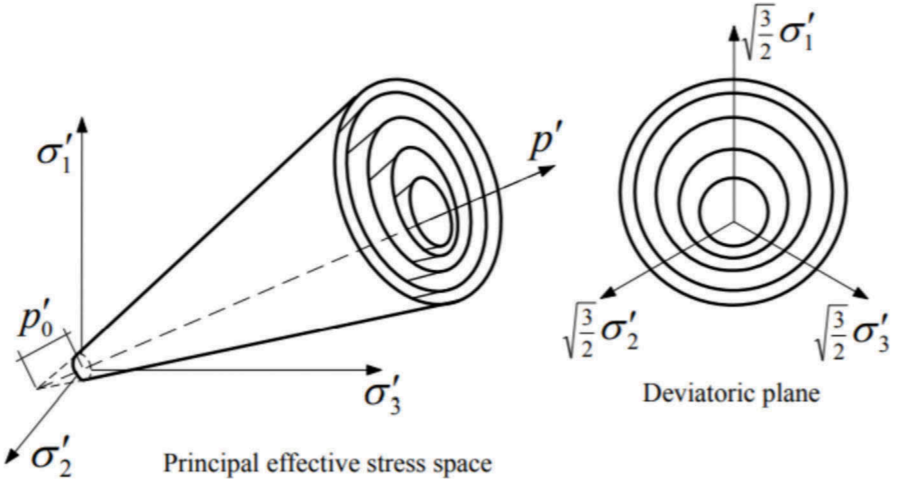

Figure 1. Multi-yield surfaces in principal stress space and deviatoric plane (after Prevost 1985, Elgamal et al. 2002, 2003, Yang et al. 2003a).

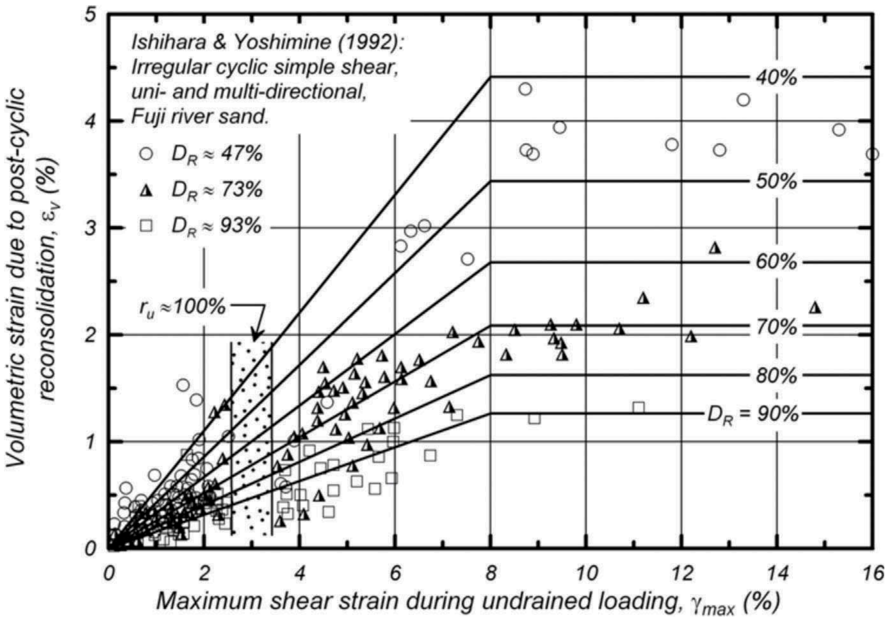

Figure 2. Relationship between post-liquefaction volumetric strain and the maximum shear strain induced during undrained cyclic loading of clean sand (Idriss and Boulanger 2008; after Ishihara and Yoshimine 1992)

$$V_{s1} = V_s\left(\frac{P_a}{\sigma'_{vc}}\right)^{0.25} \tag{1}$$

where P_a is the atmospheric pressure (100 kPa), σ'_{vc} is the initial effective vertical stress, and V_{s1} and V_s are in m/s.

Step 2: Calculate relative density D_R

Using V_{s1}, Yi (2010) proposed a relationship between D_R and$(V_{s1})_{cs}$ (V_{s1} for clean sand):

$$D_R = 17.974[(V_{s1})_{cs}/100]^{1.976}\ (\%) \tag{2}$$

where subscript *cs* is the abbreviation for clean sand (soil with 5% or less fines).

Step 3: Calculate volumetric strain ε_v

Ishihara and Yoshimine (1992) observed that the volumetric strain (ε_v) of clean sand that occurs during post-liquefaction reconsolidation was directly related to γ_{max} developed during undrained cyclic loading and also to the initial relative density D_R of the sand as shown in Figure 2.

Yoshimine et al. (2006) developed the following expression for the relationships shown in Figure 2:

$$\varepsilon_v = 1.5\exp(-2.5D_R)\cdot min(0.08, \gamma_{\max}) \tag{3}$$

in which both D_R and shear strain γ_{max} are in decimal.

4 REPRESENTATIVE NUMERICAL SIMULATIONS

Representative scenarios (Figure 3) are studied to illustrate the influence of a low shear strength (LSS) stratum existing in the middle of a soil profile on the resulting seismic response and volume change (Table 1). The low shear strength stratum is represented by loose sand in

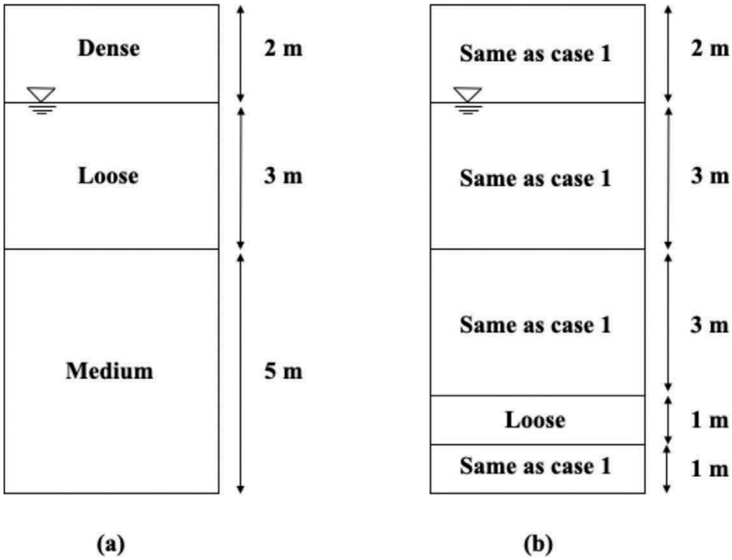

Figure 3. Employed soil profiles of study cases: (a) case 1; (b) case 2 (with a loose sand layer near the base)

Table 1. Soil properties

Soil	Friction angle (degrees)	Shear wave velocity at 10m depth[1] (m/s)	Mass Density (kg/m^3)	Permeability Coefficient (m/s)
Loose	29	185	1700	6.6×10^{-5}
Medium	35	205	1900	6.6×10^{-5}
Dense	40	225	2100	6.6×10^{-5}

this example to compare with a soil column without the low shear strength stratum. Case 1 is a 10 m soil column with 2 m dense dry sand on the top, followed by 3 m saturated loose sand layer and the bottom layer is 5 m saturated medium dense sand. Case 2 is identical to Case 1 except for a 1 m layer of loose sand (instead of medium sand) included near the base (as shown in Figure 3). Harmonic sine wave with amplitude of 0.2 g with 1 Hz was chosen as the base input motion as shown in Figure 4.

1. Shear wave velocity of cohesionless soils varies in proportion to $(p_m)^{1/4}$ where p_m is effective mean confinement.

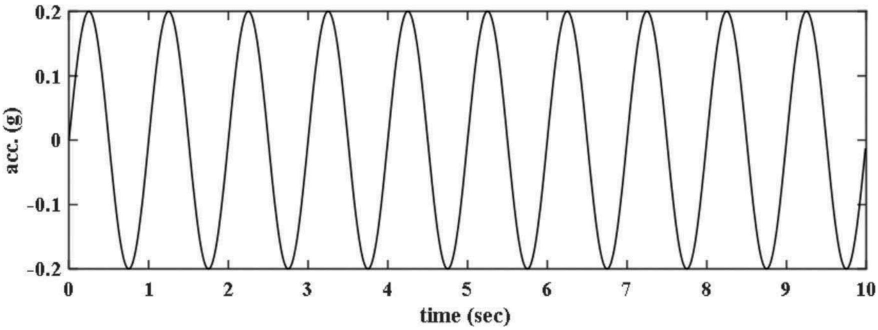

Figure 4. Base input motion

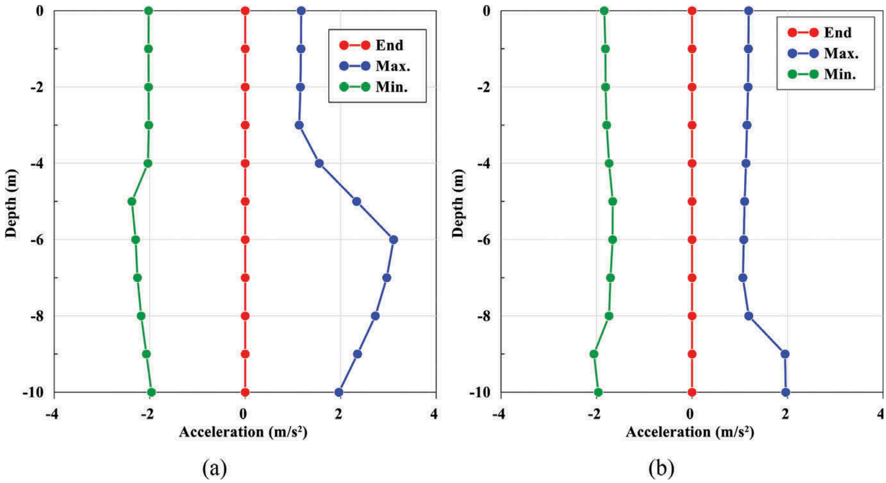

Figure 5. Lateral acceleration response profiles: (a) case 1; (b) case 2 (with a loose sand layer near the base)

Figure 5 shows the comparison of peak horizontal acceleration between the two cases. Presence of the loose sand layer near the base (Case 2) caused a reduction in the maximum and minimum horizontal acceleration along the soil profile. However, the corresponding horizontal displacement displays the opposite effect as shown in Figure 6. Presence of the loose sand stratum near the base (Case 2) increases the lateral displacement at the end of shaking. More importantly, it can be seen that the horizontal displacement was concentrated in the loose sand layer near the base.

Excess pore water pressure response profiles are shown in Figure 7. The soil column with a loose sand stratum clearly reduces generation of excess pore pressure for the layers above the LSS stratum.

Figures 8 and 9 show the shear strain and volumetric strain response profiles. Presence of the low shear strength stratum confines the volumetric strain and shear strain to the bottom layers and there is significant reduction of the volumetric strain and shear strain for the upper layers.

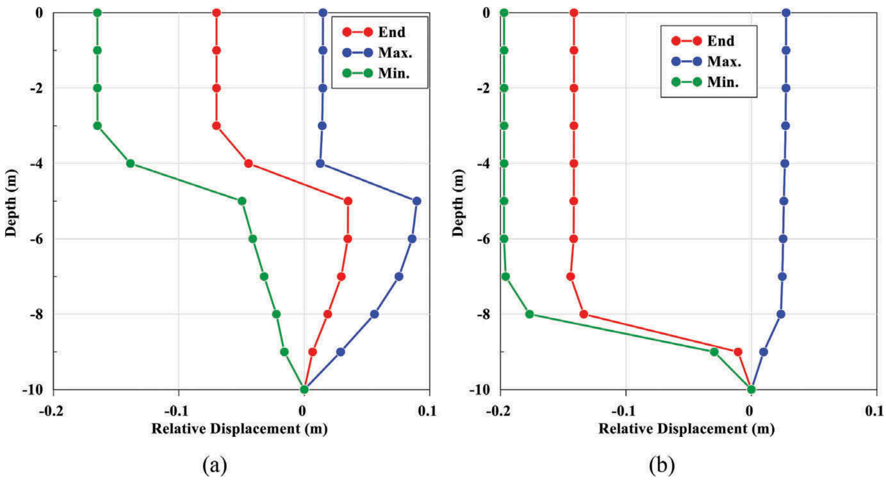

Figure 6. Lateral displacement response profiles: (a) case 1; (b) case 2 (with a loose sand layer near the base)

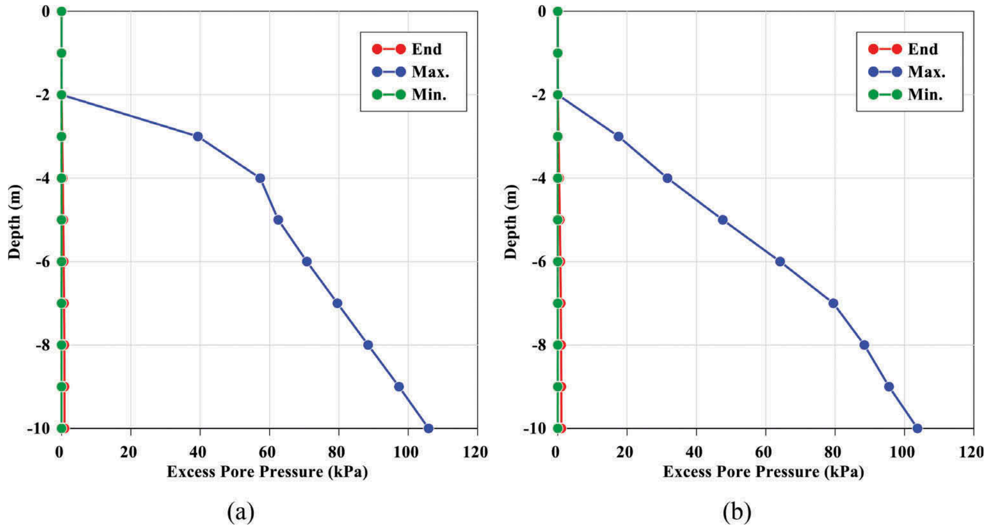

Figure 7. Excess pore pressure response profiles: (a) case 1; (b) case 2 (with a loose sand layer near the base)

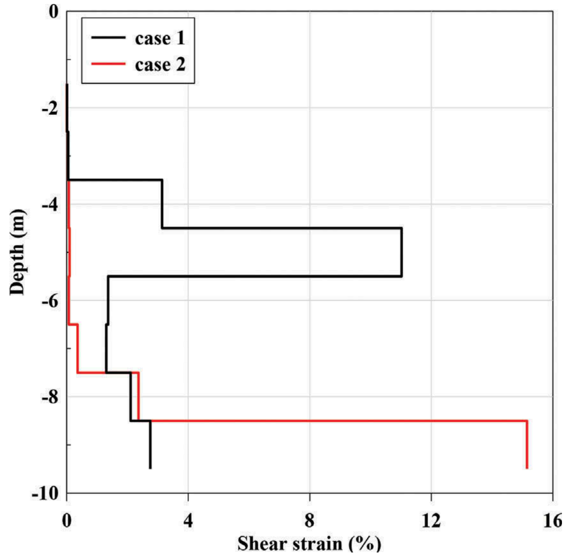

Figure 8. Shear strain response profile

Figure 10 displays the calculated settlement profiles of the two cases. Case 1 shows the distribution of the settlement along the depth as it accumulated along the stratum height. Case 2 shows a reduction in the ground surface settlement compared to case 1. It can also be observed that most settlement occurred in the lowermost strata.

5 SUMMARY AND CONCLUSIONS

A procedure for estimating liquefaction-induced settlement where the peak shear strains are obtained from a coupled formulation FE analyses was presented. The FE simulations were conducted using Cyclic1D, a PC-based graphical pre- and post-processor (user-interface) for

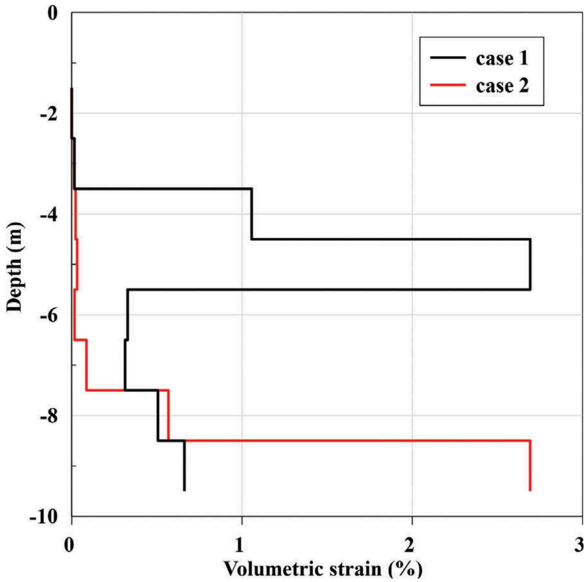

Figure 9. Volumetric strain response profiles

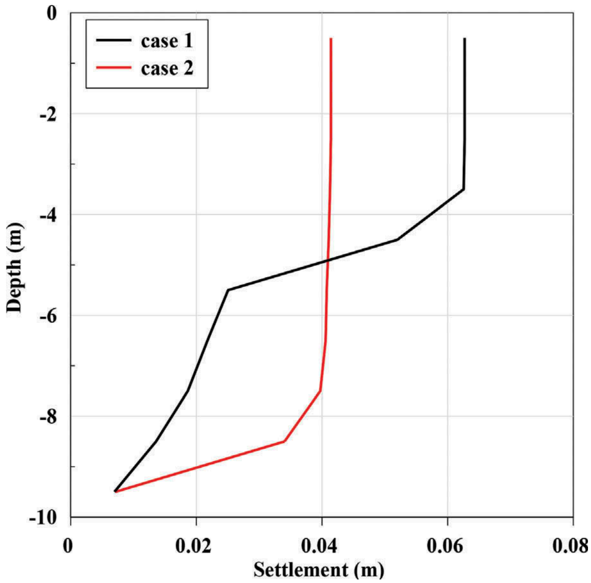

Figure 10. Calculated settlement profiles

execution of site response simulations including the analysis of liquefaction-induced lateral deformations.

A parametric study was undertaken to assess the effect of changing soil material properties on post-liquefaction settlement. From the computed results, it is clear that the introduced 1 m loose sand layer near the base caused: i) a major change in the volume change profile along the height, and ii) a significant reduction in the predicted overall site settlement. The employed program Cycilc1D, which now includes the presented site settlement calculations is available at http://www.soilquake.net/cyclic1d/.

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