A Simplified Procedure For Evaluating Post-Seismic Settlements In Liquefiable Soils

Anna Chiaradonna, Ph.D.,1 Emilio Bilotta, Ph.D.,2 Anna d’Onofrio, Ph.D.,3 Alessandro Flora, Ph.D.,4 and Francesco Silvestri, Ph.D.5

1Department of Civil, Architectural and Environmental Engineering, University of Napoli Federico II, 80125, Napoli, via Claudio 21; e-mail: anna.chiaradonna@unina.it
2Department of Civil, Architectural and Environmental Engineering, University of Napoli Federico II, 80125, Napoli, via Claudio 21; e-mail: bilotta@unina.it
3Department of Civil, Architectural and Environmental Engineering, University of Napoli Federico II, 80125, Napoli, via Claudio 21; e-mail: donofrio@unina.it
4Department of Civil, Architectural and Environmental Engineering, University of Napoli Federico II, 80125, Napoli, via Claudio 21; e-mail: flora@unina.it
5Department of Civil, Architectural and Environmental Engineering, University of Napoli Federico II, 80125, Napoli, via Claudio 21; e-mail: francesco.silvestri@unina.it

ABSTRACT

The importance of predicting ground deformation in loose, saturated granular soils has been widely recognized for a reliable evaluation of liquefaction damage. A procedure is proposed in this paper for the evaluation of post-cyclic consolidation settlements, as a result of volumetric strains induced by the dissipation of excess pore pressure. A stress-based model is first adopted for generating the excess pore water pressure in 1D free-field conditions, allowing for an effective stress analysis according to a loosely coupled approach. Then, the post-cyclic settlement is simply calculated integrating the vertical strains. To this aim, by considering a well-documented case history in which an extremely small settlement was observed upon seismic excitation, soil stiffness is estimated on the basis of either CPT data or shear stiffness decay curve, to show the effect of modelling hypothesis on the results. Both approaches result into a value of the settlement close to the observed one and much lower than that calculated using a well-established empirical procedure.

INTRODUCTION

Liquefaction-induced ground deformations have caused significant damage to engineered structures and lifelines during past earthquakes (e.g., 1999 Kocaeli, Turkey; 2011 Christchurch,
New Zealand; 2012 Emilia, Italy earthquakes). Such experiences once more showed that the evaluation of ground deformation on loose, saturated granular soils is a key factor for a reliable assessment of liquefaction-induced damage.

Recently, Bray and Macedo (2017) suggested computing the liquefaction-induced settlement of building as the sum of three different components: (i) settlement due to the formation of sediment ejecta; (ii) shear-induced settlement during seismic shaking caused by the preexisting stress field below the building; (iii) post-liquefaction settlement induced by volumetric consolidation. The relative importance of these three components depends on a number of factors, the most critical being the existence of a low permeability thin crust, the thickness of the loose sand layer being liquefied, the geometrical features and the loads of the building. Often, the distortional component is the most critical one.

Nevertheless, the evaluation of the volumetric post-seismic component is obviously important. This paper presents a simple, sound method to estimate it. The state-of-the-practice for estimating post-liquefaction volumetric-induced building settlements is largely based on empirical procedures developed in 1D free-field conditions, i.e. without considering the effects of structures (e.g., Ishihara and Yoshimine 1992; Zhang et al. 2002). These procedures usually use charts correlating the volumetric post-cyclic strains to the soil relative density as well as to a liquefaction safety factor. Such correlations stem from laboratory investigations on specific sands, and may lack of the ability to represent cases referred to different soils. Simplified 1D dynamic analyses, needing a limited computational effort, may overcome some of the limitations of these empirical approaches, providing a reasonable compromise between the need to have an expeditious tool and that to have a reliable result.

The method proposed in this paper is based on simple, non-linear, effective stress numerical 1D simulations, used to evaluate excess pore pressures generated by the seismic event (Chiaradonna et al., 2018a). The post-cyclic settlement is then calculated as the integral of consolidation volumetric deformations, as described in the following section.

The proposed approach was applied to a river dike struck by the seismic sequence occurred in Emilia plain (Italy) in May, 2012. The embankment was damaged by some minor slope instability problems, but no detectable settlement was observed. Considering the large extent of the embankment compared to the thickness of the liquefiable ground layer, 1D conditions could be reliably adopted in the calculations. In order to have a reference result, the settlement was estimated also using the well-known empirical procedure proposed by Zhang et al. (2002).

**ESTIMATION OF VOLUMETRIC-INDUCED GROUND SETTLEMENT**

The dissipation of excess pore pressures induces a post-cyclic consolidation settlement that can be computed as follows:
\[ w = \sum_{i}^{N} \frac{\Delta \sigma_z}{E_{oed,i}} \Delta z_i \]  

(1)

where \( \Delta \sigma_z \) is the increment of vertical effective stress arising from the dissipation of the excess pore pressure, \( \Delta u \), \( E_{oed,i} \) is the constrained modulus and \( \Delta z_i \) is the thickness of the generic layer \( i \) that results from the discretization of the soil profile into \( N \) layers. The constrained modulus must be estimated on available data, which can be in situ tests (for instance CPT results), simple non-linear equivalent elastic models or more sophisticated laboratory results. With the previous approach (CPT results), the Young modulus can be estimated as:

\[ E' = \alpha_E \cdot q_c \]  

(2)

where \( q_c \) is the CPT tip resistance and \( \alpha_E \) is an empirical coefficient. Then, assuming that the solid skeleton may be treated as an elastic medium with Young’s moduli \( E' \) and \( G \) and Poisson’s ratio, \( \nu' \), it follows that:

\[ E_{oed} = E' \frac{1-\nu'}{(1+\nu')(1-2\nu')} \]  

(3)

Alternatively, considering the equivalent non-linear elastic approach, a shear stiffness \( G \) can be calculated as a function of the effective stress and of the shear strain level on a stiffness decay curve (as inferred from the literature or from laboratory testing). Then, the constrained modulus can be expressed as:

\[ E_{oed} = \frac{2 \cdot G \cdot (1-\nu')}{(1-2\nu')} \]  

(4)

This approach has the advantage of being based on the value of \( G \) related to the effective stress state upon seismic excitation, and should thus lead to the most reliable estimate of the settlement.

**APPLICATION TO A CASE STUDY**

The above described procedure was used to estimate the post-seismic volumetric settlement likely induced by the Emilia earthquake in Italy (20.V.2012, \( M_w=6.1 \)) in the sandy soil deposits constituting a river bank at the site of Scortichino, where significant evidences of soil deformation and building damages were observed after the main event (Tonni et al., 2015). To this aim, an effective stress analysis was carried out by the 1D non-linear code SCOSSA (Tropeano et al., 2016) on a reference soil column of the river embankment. Since the 2012 seismic sequence affected an alluvial plain with a significant depth of the seismic bedrock, no acceleration records of the mainshock on rock outcrop sites were available. To overcome this
problem, the deconvolved outcrop motion at the site of Scortichino was considered in the analysis (Figure 1c), as detailed in Chiaradonna et al. (2018b).

An extensive in-situ and laboratory geotechnical investigation carried out after the earthquake allowed for defining an accurate model for the dynamic analyses (Tonni et al., 2015). Figure 1a shows the soil layering and the related shear wave velocity profile, as obtained by combining the results of borehole and surface geophysical tests. The core of the dike (AR) and its foundation soil (B) consist of a silty sand, while a thick formation of alluvial sands (A), interbedded by clay (C), overlies an alternation of both materials (AL) and the bedrock.

Figure 1. (a) Soil profile, (b) stiffness and damping vs strain, and (c) reference input motion.

Figure 1b shows the normalized shear modulus and damping ratio curves, obtained from resonant column and cyclic simple shear tests, adopted to simulate the non-linear soil behaviour. The build-up of excess pore water pressure in the saturated soils has been predicted through a simplified model based on the cyclic accumulation of shear stress (Chiaradonna et al., 2018a). Such a model enables to compare the irregular time-history of shear stress induced by earthquake with the soil liquefaction resistance, as measured in cyclic stress-controlled laboratory tests. Figure 2 shows the best-fit curves predicted by the pore pressure model based on the data of cyclic simple shear tests on undisturbed samples of silty sand and clean sand (Tonni et al., 2015).

Figure 3 reports the results of the 1D effective stress analysis in fully undrained condition, expressed in terms of profiles of maximum acceleration, shear strain, shear stress and excess pore pressure ratio. Liquefaction is not attained, but a maximum excess pore pressure ratio of 0.6 is shown in the silty sand layer. The acceleration profile is characterized by a
significant reduction in the dike; this effect can be attributed to the lower stiffness and higher damping mobilized in the underlying silty sand layer, which dampers the upwards waves propagation, acting as an isolator at the base of the dike.

**Figure 2.** Cyclic resistance curves (a) and pore water pressure relationship (b) for clean sand and silty sand.

**Figure 3.** Results of the effective stress analysis in terms of profiles of peak (a) acceleration, (b) shear strain, (c) shear stress and (d) pore pressure ratio.

**Prediction of settlements according to Zhang et al. (2002)**

Based on the results of the dynamic analysis, the post-seismic consolidation settlement was estimated as described in the following. The approach proposed by Zhang et al. (2002) combines an existing CPT-based method to estimate liquefaction resistance with laboratory test results on clean sand, in order to evaluate the liquefaction-induced volumetric strains for sandy and silty
soils. More in detail, the CPT-based liquefaction potential analysis proposed by Robertson and Wride (1998) is adopted for evaluating a vertical profile of the factor of safety, FS. FS was defined as the ratio between the cyclic resistance ratio (CRR) based on the normalized cone tip resistance, \( q_{c1N} \) \( c_s \) (Robertson and Wride, 1998), and the cyclic stress ratio (CSR), computed through the maximum shear stress profile (Figure 3c). Figure 4a reports the CPT data for the soil column, as reported by Tonni et al. (2015).

![CPT profiles](image)

**Figure 4.** CPT profiles of (a) cone tip resistance, (b) sleeve friction, (c) soil behaviour type index.

The four key plots for estimating liquefaction-induced ground settlements by the CPT-based method proposed by Zhang et al. (2002) are presented in Figure 5. Data in Figures 5a and 5b are the results of the liquefaction potential analysis. Data in Figure 5c are calculated from the relationship between volumetric strain and equivalent clean sand normalized CPT tip resistance, for different factors of safety (FS) (see Figure 3 in Zhang et al., 2002). The settlement profile shown in Figure 5d is obtained integrating the deformations along the soil column, and the surface value is equal to 8 cm.

**Prediction of settlements from CPT results**

The calculation of the surface settlement using eqs. (1), (2) and (3) was based on the smoothed \( q_c \) profile (blue line) reported in Figure 6a. The Young’s modulus was estimated using eq. (2). Robertson and Cabal (2010) suggest that the coefficient \( \alpha_E \) can be estimated for uncemented sandy soils as function of the soil behaviour type index, \( I_c \), as follows:
\[ \alpha_E = 0.015 \left[ 10^{0.55I_c} + 1.68 \right] \]  

(5)

Robertson and Cabal (2010) also specify that the values of \( \alpha_E \) calculated using eq. (5) are related to medium strain amplitudes (about 0.1\%). For higher strain levels, lower values of \( \alpha_E \) should be adopted. Applying equation (5) on the \( I_c \) data reported in Figure 4c, a mean value \( \alpha_E = 16 \) was obtained for the silty sand, and \( \alpha_E = 10 \) for the sandy soil layer. Looking at the maximum shear strain profile (Figure 3b), it is clear that strain amplitudes higher than 0.1% are attained in the silty sand layer and in the upper part of the sand deposit, while smaller values are reached in its lower part. As a consequence, the coefficient \( \alpha_E = 10 \) was assumed for the sand deposit since the strain level attained on average in the layer is about 0.1%, while a reduced value of \( \alpha_E \) ranging from 10 to 5 was adopted for the silty sand. The considered range, which is part of a sensitivity analysis, takes into account the high level of uncertainty on the definition of \( \alpha_E \).

The Poisson’s ratio, \( \nu' \), needed in eq. (3) was computed from the coefficient of earth pressure at rest, \( K_0 \), measured through SDMT tests. The values of \( \nu' \) are equal to 0.31 for the silty sands (B) and 0.29 for the alluvial sand deposits (A), respectively. Figure 6b shows the profile of constrained modulus based on the CPT data, computed through equations (2) and (3).

For \( \alpha_E \) equal to 10 and 5, an overall settlement (Figure 6d) ranging from 2.5 to 3.5 cm was respectively computed with eq. (1). Such values are two to three times smaller than that predicted with the Zhang et al. (2002) approach previously adopted.

Figure 5. Settlement predicted through the empirical method by Zhang et al. (2002): vertical profiles of (a) cone tip resistance, (b) factor of safety, (c) volumetric strain and (d) induced ground settlement.
Prediction of settlements from shear modulus at the end of shaking

During ground shaking, the degradation of the tangent shear modulus was correlated to the excess pore pressure through the relationship proposed by Matasovic and Vucetic (1993). As an example, Figure 7 shows the time histories of the excess pore pressure and tangent shear modulus at 7.2 m under the ground surface. It can be pointed out that most of the degradation of the soil stiffness is induced by the excess pore pressure build-up. The final value of the shear modulus, \( G_{degr} \), was assumed for estimating the constrained modulus, \( E_{oed} \), (eq. 4).

Figure 8a reports the vertical profile of the mobilized shear modulus at the end of shaking in the uppermost 40 m. It can be observed that the lowest values pertain to the silty sand layer under the ground water table, where the maximum shear strain levels are attained (Figure 3b).

![Figure 6. Settlement predicted through the CPT-based approach: vertical profiles of (a) cone tip resistance, (b) constrained modulus, (c) excess pore pressure ratio and (d) induced ground settlement.]

![Figure 7. Time histories of excess pore pressure ratio and tangent shear modulus at 7.2 m under the ground surface.]

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An overall settlement (Figure 8d) of about 1.2 cm was computed from the application of eqs. (1) and (4). Such a value is smaller than that calculated using CPT results, and much smaller than the one estimated with Zhang et al. (2002) method. It is also the closest to the observed settlement that was lower than 1 cm in the analysed section.

![Figure 8](image.png)

**Figure 8.** Vertical profiles of (a) degraded shear modulus, (b) constrained modulus, (c) excess pore pressure ratio and (d) induced ground settlement.

**CONCLUSIONS**

The post-seismic consolidation settlement can be calculated by simply integrating the vertical strains induced by the dissipation of excess pore water pressures. The quality of the prediction obviously depends on the ability to evaluate the soil constrained stiffness. To show the effect of different modelling hypothesis, two different strategies where adopted, using respectively CPT results and a non-linear equivalent elastic approach. The previous relates soil stiffness to tip resistance with an empirical correlation, while the latter has the great advantage of allowing to take into account the variation of effective stresses on the mobilized stiffness. With both approaches, realistic values of surface settlement were calculated for the well-documented case history of a dike in Emilia Romagna, struck by the 2012 earthquake, consistent with the observed damage pattern reported by Tonni et al. (2015). Further applications of the proposed procedure can be found in Chiaradonna et al. (2018c), where the effectiveness of the proposed procedure has been demonstrated on three other well-documented case histories.
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REFERENCES