Response mechanisms of liquefiable deposits and their influence on surface liquefaction manifestation

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ABSTRACT

Results from a series of effective stress analyses are used to identify and discuss response mechanisms of Christchurch soil deposits with varying liquefaction performance and surface liquefaction manifestation during the 2010–2011 Canterbury Earthquake Sequence. In particular, three representative soil profiles of varying characteristics, belonging to sites that did not manifest surface evidence of liquefaction in the 4 September 2010 earthquake, but did manifest liquefaction in the 22 February 2011 earthquake are analyzed using input ground motions representative of these two main events. Simplified (soil–column) models for effective stress analyses are developed primarily based on data from in-situ cone penetration tests, and an advanced state-concept based constitutive model is used to simulate the complex dynamic response of liquefiable soils. The study shows that system-response processes involving interactions between different layers in the dynamic response, and through excess pore water pressure redistribution and water flow, play a key role in the overall performance of the deposit and the severity of liquefaction manifestation at the ground surface.

Keywords: earthquake, effective stress analysis, liquefaction, system response

INTRODUCTION

Liquefaction assessments (i.e. evaluation of liquefaction triggering and consequences) are routinely carried out by geotechnical engineers using empirical methods which are largely based on observations from case histories (e.g. Boulanger & Idriss, 2014; Idriss & Boulanger, 2008; Robertson & Wride, 1998; Youd et al, 2001, among others). These methods consider each layer in isolation (i.e. separately from any other layer in the deposit), and a factor of safety against liquefaction triggering, maximum shear and volumetric strains are estimated separately for each layer. In other words, interaction between different layers in the dynamic response or pore-water pressure re-distribution and water flow are ignored in these methods. Initial observations from the 2010–2011 Canterbury, New Zealand earthquakes indicated that such system response effects were a significant factor for both manifestation of liquefaction and severity of liquefaction–induced damage. In the initial screening, clear anomalies were identified in the predictions by the simplified methods over specific areas, and for certain types of soils and stratification of deposits (Cubrinovski et al, 2018).

To elucidate some of these effects and their relation to the severity of surface liquefaction manifestation, Cubrinovski et al. (2018) scrutinized the soil profile characteristics and responses of 55 well-documented liquefaction (and no-liquefaction) case histories (free–field level ground sites) from Christchurch. At each of

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these sites, detailed assessment of land damage was conducted by Tonkin+Taylor engineers who classified the severity of liquefaction manifestation as: minor, moderate, major, severe and very severe, following the classification scheme adopted by Russell & van Ballegooij (2015). Also, comprehensive geotechnical investigations were performed at each site including cone penetration tests (CPTs), borehole logs, piezometer tests, and high resolution compression wave (Vp) and shear wave velocity (Vs) measurements (at 200 mm intervals) using direct-push cross-hole technique (Cox et al., 2017).

Focusing on the two main events in the 2010–2011 Canterbury Earthquake Sequence (CES), Cubrinovski et al. (2018) considered three groups of sites: (i) sites that manifested liquefaction (sand boils) in both the 4 September 2010 and 22 February 2011 earthquakes (Yes/Yes sites), (ii) sites that did not manifest liquefaction in the September event but manifested liquefaction in the February earthquake (No/Yes sites); and, (iii) sites that did not manifest liquefaction in either event (No/No sites). The Yes/Yes and No/No sites were shown to have practically identical critical layer characteristics, with low penetration resistance and shallow location of the critical layer. However, significant differences between the Yes/Yes and No/No sites were identified in regard to their deposit characteristics and soil stratigraphy. Indicatively, as illustrated in Fig. 1, the No/No sites are characterized by vertical discontinuity of the liquefiable layers, that is, low permeability non-liquefiable layers interchangeably sequencing relatively thin liquefiable layers of low resistance. Results of advanced effective stress analysis show that this type of deposits is affected by dynamic cross-interaction between layers, where liquefaction in a loose deeper layer substantially reduces the demand for all soils above that depth, thereby and in conjunction with partial saturation effects preventing liquefaction of the critical layer at shallow depth (Fig. 1b). On the other hand, the typical Yes/Yes site is comprised of a thick critical zone at shallow depth which is liquefied first, but it is also subjected to additional disturbance by water seeping upwards from deeper soils toward the critical zone, hence creating a thick and continuous zone of unconstrained strong upward flow of water towards the ground surface (Fig. 1a). Depending on which mechanism is activated in each case, the system–response may intensify (as in the case of the Yes/Yes sites) or mitigate (as in the case of No/No sites) the effects of liquefaction on the severity of surface manifestation and associated damage.

![Figure 1. Schematic illustration of representative soil profiles and key system–response mechanisms for (a) Yes/Yes and (b) No/No sites (modified after Cubrinovski et al., 2018)](image)

The current study is a follow-on to the Cubrinovski et al. (2018) paper and presents an attempt to interpret the response of representative No/Yes sites (i.e. sites that did not manifest liquefaction in the September earthquake but manifested liquefaction in the February event) which were left out from the initial study. Note that some results from seismic effective stress analyses of characteristic No/Yes case histories (among others) were also presented in Ntritsos et al. (2018); the present paper however focuses exclusively on the No/Yes sites and includes a more comprehensive discussion on the predicted system–response mechanisms and their relation to the observed liquefaction manifestation at the ground surface.

**REPRESENTATIVE SOIL PROFILES**

Christchurch is largely founded on deep alluvial soils of the Canterbury plains, a fan deposit resulted from numerous rivers flowing eastward from the foothills of the Southern Alps (Brown and Weeber, 1992). These soils comprise gravels, sands, silts, peat and their mixtures, and are highly variable both vertically and
horizontally over relatively short distances. Not surprisingly, the soil profile characteristics within the No/Yes group of sites, a sub-group of the 55 sites, are also highly variable, as they cover a broad range of sites with properties roughly somewhere between those of the No/No and Yes/Yes sites. In this paper we are examining three typical No/Yes sites from different parts of the city which can be assumed, in general terms, to be representative of the vast majority of the No/Yes sites.

The top 10 m of the soil profiles for the three select No/Yes sites (NY-1, NY-2 and NY-3), in terms of soil behavior type index \( I_c \) and equivalent clean sand cone tip resistance \( q_{c1Ncs} \) (calculated from the CPT data using the Boulanger and Idriss, 2014 liquefaction triggering procedure), are presented in Fig. 2. The measured \( V_p \) profile for each site is also shown in this figure to indicate the degree of saturation throughout the deposit. Identified critical layers are marked with the salmon shading in the \( q_{c1Ncs} \) columns. The latter represent the layers (typically low tip resistance shallow layers) which are most likely to manifest liquefaction at the ground surface (if liquefied).

![Figure 2](image)

**Figure 2.** \( q_{c1Ncs}, I_c \) and \( V_p \) profiles for the three representative No/Yes sites used in this study: (a) NY-1; (b) NY-2; (c) NY-3. The solid black lines overlying the original CPT traces (grey lines) indicate the average values used in the calibration of the constitutive model (see “METHODOLOGY” section). The following abbreviations are used in this figure: \( Cs = \) clean sand, \( 1.3 < I_c \leq 1.8 \); \( SwF = \) sand with small amount of fines, \( 1.8 < I_c \leq 2.1 \); \( NPs = \) sandy silt and non-plastic silt, \( 2.1 < I_c \leq 2.6 \); \( PLs = \) plastic silt or clay, \( I_c > 2.6 \).

Key features of the representative soil profiles presented in Fig. 2 are summarized in the following:

1) The NY-1 profile has multiple liquefiable layers (at various depths) with relatively low \( q_{c1Ncs} \), and \( I_c \) indicative of clean sand or silty sand behavior. The two shallowest of these layers, at depths 1.4 and 2.1 m, are the most critical layers for liquefaction manifestation at the ground surface. It is worth noticing that soils as deep as 6 m from the depth of the ground water table are partially saturated (i.e. \( V_p < 1500 \text{ m/s} \)). Therefore, some increase in the liquefaction resistance of these soils should be expected. This is particularly true for the layers at depth from 3.7 to 4.7 m which have the lowest \( V_p \) value (~ 850 m/s); for the remaining layers with \( V_p > 1000 \text{ m/s} \), this increase in the liquefaction resistance will be relatively small (e.g. Tsukamoto et al, 2002; Hossain et al, 2013). Lastly, multiple non-liquefiable layers are also present at various depths (similarly to the
typical No/No profile, see Fig. 1b) hindering the vertical communication of excess pore water pressures developed in the liquefiable layers.

2) The NY-2 profile has characteristics similar to the Yes/Yes soil profiles (see Fig. 1a), these being the thick critical zone of low \( q_{c1Ncs} \) and the underlying clean sand type soils of higher resistance. The major difference between typical Yes/Yes and NY-2 profiles lies in the location of the critical zone, which in the NY-2 profile starts at 3 m depth from the ground surface, about 1 m deeper than in the typical Yes/Yes profile. Also worth noting is the presence of a thin non-liquefiable layer within the critical zone, which breaks the continuity of the latter and consequently, the ease with which excess pore water pressures are transferred from one layer to the other.

3) The NY-3 profile is largely comprised of fully saturated clean sands below the water table, and has a 0.6-m thick critical layer of lower resistance at depth 3.6 m from the ground surface. Thin layers of lower resistance are also present at depths about 5.5 and 7.3 m.

**METHODOLOGY**

A series of fully coupled nonlinear effective stress analyses were carried out to investigate the characteristics of the free-field response at the three representative No/Yes sites. Simplified one-dimensional (1D) “soil–column” models (i.e. 1D vertical wave propagation with two-dimensional quadratic elements constrained to deform in simple shear) were developed based on the CPT data by identifying depth intervals over which \( q_c \) and \( I_c \) can be approximated by constant values (see black solid lines in Fig. 2).

An elastic–plastic constitutive model tailored for liquefaction problems was employed in the effective stress analyses (Cubrinovski & Ishihara, 1998a; 1998b). The so-called Stress–Density model (S-D model) is a state-concept based model that accounts for the combined effects of density and confining stress on sand behavior through the state-concept framework. The S-D model parameters were determined through a combined use of empirical relationships and generic data for sandy soils. More specifically, Christchurch sands were modelled using model parameters established from laboratory tests on Toyoura sand as a basis (Cubrinovski & Ishihara, 1998a; 1998b). The dilatancy parameters of the model were then slightly modified to simulate target liquefaction resistance curves (LRCs) established using the CPT–based liquefaction triggering procedure of Boulanger & Idriss (2014). In the development of the target LRCs, partial saturation effects on liquefaction resistance were also taken into account by modifying the obtained curves for \( V_p \)’s lower than 1400 m/s according to the model proposed by Hossain et al. (2013). For a given \( V_p \), the same set of model parameters was used to simulate the target LRCs across all different density states of liquefiable soil layers with the initial void ratio being the only parameter to change as a function of \( q_{c1Ncs} \).

Non-liquefiable layers were also modelled with the S-D model except that the pore pressure generation feature was turned off for these layers. The initial shear modulus in these layers was defined based on the shear wave velocity measured in the high resolution cross–hole testing. The shear stress – shear strain relationships were defined based on model simulations of the generic stiffness degradation and damping ratio curves proposed by Darendeli (2001) and modified for strength compatibility at large strains according to the procedure described in Yee et al. (2013).

The 1D soil–column models used in the effective stress analyses represented the top 20 m of the deposit, and input motions were applied at the base of the model, at 20 m depth. Using several deeper CPTs, characteristic layers from 10 to 20 m depth were identified, and these layers were modelled following the procedure described above for non-liquefiable soils. Hence, all three representative soil profiles (NY-1, NY-2 and NY-3) had identical soil profiles and model parameters from 10 m to 20 m depth.

Analyses were performed under drained conditions allowing for pore water pressure redistribution and vertical water flow through and between layers. Soil permeability values were estimated for each layer based on the corresponding \( I_c \) values, following the recommendations of Robertson and Cabal (2015).

Two input ground motions were used as base excitations of the soil–column models for each of the three representative No/Yes soil profiles, resulting in a total of six effective stress analyses. Both ground motions
were obtained from deconvolution of surface motions recorded during the 4 September 2010 and 22 February 2011 at select “reference” strong motion station sites (Nritisos et al, 2019). The deconvoluted motions were then scaled to 0.20g and 0.40g to approximately represent the average levels of intensity experienced in Christchurch City during the September and February earthquake, respectively. Acceleration time histories and acceleration response spectra at 5% damping for the two input ground motions are presented in Fig. 3. Clearly, for the majority of vibration periods of engineering interest, the spectral acceleration amplitudes are larger for the 22Feb2011 earthquake motion, with the exception of long vibration periods (i.e. T > 2 s) due to both the longer duration of shaking and forward directivity effects in the September earthquake (Bradley & Cubrinovski, 2011).

Figure 3. Acceleration time histories and acceleration response spectra at 5% damping for the two input motions, 22Feb2011 and 04Sep2010, used as base excitations of the soil–column models

RESULTS

Figs 4, 5 and 6 comparatively show the results of the effective stress analyses at each representative NY model for the two input motions, 04Sep2010 and 22Feb2011, in terms of excess pore water pressures Δu at specific time sections (columns d and e) and maximum shear strains γ max (column f) for the top 10 m of the deposit. Also shown in Figs 4, 5 and 6 are the profiles of “magnitude–corrected” peak ground acceleration a max, M=7.5 (column c) which roughly indicate the level of seismic demand imposed on throughout the depth of the profile. a max, M=7.5 was taken as the maximum acceleration computed from the effective stress analysis at each node, divided by a magnitude scaling factor (MSF) derived as follows. At each node the number of (constant acceleration amplitude) equivalent cycles N eq was computed from the acceleration time history using the “peak-between-mean crossing count” method (Dowling, 1972) and the weighting scheme outlined in Appendix A.2 of Boulanger & Idriss (2014). The resulted N eq was then used as input to equation A.4 of Boulanger & Idriss (2014) to obtain MSF. Because the MSF calculated using this procedure is dependent on q c1N cs through the parameter b (i.e. the slope of the LRC), two MSF-values were computed at each node, one for each element sharing the node; hence, the discontinuity of the a max, M=7.5 profile in Figs 4, 5 and 6.

In the following, key response features and mechanisms of the three representative No/Yes profiles are discussed, in particular with regard to liquefaction development and surface liquefaction manifestation.

NY-1 analyses

From a liquefaction manifestation viewpoint, the NY-1 profile has a critical zone from 1.4 to 3.2 m depth (encompassing two critical layers separated by a thin non-liquefiable layer), but all low resistance liquefiable
layers (i.e. including the deeper layers) are relevant for liquefaction triggering and its consequent effects on the dynamic response of the deposit.

Figure 4. NY-1 deposit: (a) simplified soil profile; (b) equivalent clean sand cone tip resistance, \( q_{c1Ncs} \); (c) magnitude-corrected peak acceleration, \( a_{\text{max}, M=7.5} \); (d-e) evolution of excess pore water pressures with time, \( \Delta u \); (f) maximum shear strain, \( \gamma_{\text{max}} \), across the top 10 m of the deposit.

Under the 04Sep2010 base excitation, the liquefiable layers of the deposit respond with gradual build-up of excess pore water pressures (indicated in Fig. 4d) and full liquefaction is being triggered in a layer from 4.4 to 5.6 m depth at time \( t \approx 20 \) s. At that time, the excess pore water pressures that have been developed within the critical layer at 2.1 m depth are less than or about equal to half of the initial vertical effective stress at this depth, whereas no excess pore water pressures have been developed in the shallowest critical layer (i.e. the layer at 1.4 m depth). Similarly, the deeper liquefiable layers are also far from a zero effective stress state at \( t = 20 \) s. Liquefaction of the layer at 4.4 m depth reduces the seismic demand for all soils above that depth, as indicated by the drop in \( a_{\text{max}, M=7.5} \) in Fig. 4c. As a result, the remaining level of loading (after \( t = 20 \) s) is not sufficient to induce liquefaction of the shallow critical layers. This effectively results in a non-liquefiable crust from the ground surface to 4.4 depth. Under these conditions, liquefaction of the layer at 4.4 depth alone would be unlikely to manifest at the ground surface for the relatively moderate amplitudes of shaking caused by this event.

The response of the NY-1 profile is crucially different in the case of the 22Feb2011 excitation. Although liquefaction first occurs within the same layer at \( t \approx 13 \) s, and in particular in the upper part of this layer from 4.4 to 5 m depth, the excess pore water pressures (at that time) within the shallower critical layer are now much closer to the initial overburden effective stress, particularly at the bottom sublayer of the critical layer which is nearly liquefied. The reduction in the accelerations caused by liquefaction of the deeper layer is now not sufficient to prevent liquefaction of the critical layer which quickly liquefies from bottom-up within the next few seconds. Some moderate liquefaction manifestation at the ground surface would be expected in this case.

It is also quite informative to look at the distribution of maximum strains for each earthquake. In the case of the 04Sep2010 earthquake, large strain development is mostly concentrated on the (early) liquefied layer at 4.4 m depth whereas noticeable strains (but of much smaller magnitude) also appear in the deeper layer at 7.4
m depth which also liquefied but at a rather later time. Importantly, the strains in the shallow critical layer are relatively small, less than 1%. On the other hand, in the case of the 22Feb2011 earthquake, relatively large strains develop at four different depths in the profile. The deep layer at 7.4 m, although of higher q_{c1Ncs} and hence, higher density than the two shallower liquefied layers, and also despite the fact that liquefies few seconds later than the two shallower layers, exhibits the largest strains (~ 5%), most likely due to the fact that it is under higher confining stress (i.e. slightly more contractive behavior) and that it is also subjected to higher seismic demand compared to the shallower layers, since liquefaction effects on weakening the seismic demand are added up as we move toward the ground surface and the cumulative thickness of liquefied soils increases. Clearly, apart from the density state of the soil itself, the seismic demand (or energy of loading) that remains after liquefaction has been triggered in a layer, is crucial for the magnitude of resulting strains.

NY-2 analyses

The NY-2 profile has a critical zone of low q_{c1Ncs} from 3.0 to 5.8 m depth, overlying clean sands of higher resistance. In both the 04Sep2010 and 22Feb2011 excitation cases, the largest part of the critical zone liquefies at a relatively early stage of shaking, at times t ≈ 20 s and t ≈ 14 s for 04Sep2010 and 22Feb2011 respectively, in both cases with significant energy still present in the remaining (post-triggering) part of the input ground motion (see Fig. 3). This results in severe liquefaction and large strain accumulation at depths between 4.2 and 5.8 m. Despite the almost identical responses of the critical zone for the two excitation cases suggested by the effective stress analyses (i.e. similar level of maximum strains with the exception of the shallower sublayers of the critical zone), evidence of liquefaction at the ground surface was observed at this site only after the 22 February 2011 earthquake.

![Figure 5. NY-2 deposit: (a) simplified soil profile; (b) equivalent clean sand cone tip resistance, q_{c1Ncs}; (c) magnitude-corrected peak acceleration, a_{max, M=7.5}; (d-e) evolution of excess pore water pressures with time, Δu; (f) maximum shear strain, γ_{max}, across the top 10 m of the deposit.](image)

An important difference in the responses of the NY-2 deposit lies, in fact, in the deeper soils immediately beneath the critical zone, which did not liquefy in both cases, but built up much higher excess pore water pressures in the case of the 22Feb2011 earthquake. In this case, the excess pore water pressures in the deeper layers are equal or higher than those in the critical zone, activating the mechanism 2 depicted in Fig 1a with
substantial upward flow of water into the critical zone from underlying layers between 5.8 m and 10 m depth. This will cause additional disturbance including prolonged liquefaction and more severe fluidization of the already liquefied soils in the critical zone as well as a strong upward flow towards the ground surface with contributions from a large part of the deposit. This strong seepage flow seems now capable of breaking the integrity of the (over 3 m) thick surface crust and manifesting liquefaction at the ground surface in the form of sand boils.

NY-3 analyses

The NY-3 profile has a 0.6 m – thick critical layer with $q_{c1Ncs} \approx 100$ at depth 3.6 m from the ground surface, surrounded by clean sands of higher resistance. In a simplified analysis context, that is, if not accounting for system response effects (e.g. redistribution of excess pore water pressures and water flow), liquefaction of the relatively high resistance and thin critical layer alone would predict either minor liquefaction manifestation or no manifestation.

![Figure 6. NY-3 deposit: (a) simplified soil profile; (b) equivalent clean sand cone tip resistance, $q_{c1Ncs}$; (c) magnitude-corrected peak acceleration, $a_{max, M=7.5}$; (d-e) evolution of excess pore water pressures with time, $\Delta u$; (f) maximum shear strain, $\gamma_{max}$, across the top 10 m of the deposit.](image)

Effective stress analysis with the 04Sep2010 input motion suggests that liquefaction does not occur at any depth in the deposit. In fact, due to the relatively high resistance of the clean sands composing the NY-3 profile, the developed excess pore water pressures remain far below the level of initial vertical effective stress all throughout the deposit. Also, no sign of softening–induced reduction of the seismic demand is evident in $a_{max, M=7.5}$ in accordance with the low magnitude of predicted maximum strains in Fig. 6f.

In contrast, in the 22Feb2011 excitation liquefaction does occur in the critical layer at $t = 16.2$ s. Clearly, as can be seen in the $a_{max, M=7.5}$ profile, the 22Feb2011 excitation subjects the soils to higher seismic demand, resulting not only in liquefaction of the critical layer but also in a development of high excess pore water pressures in the deeper soils below the critical layer. In fact, the excess pore water pressures at these deeper soils are about equal to the excess pore water pressures in the overlying critical layer. This implies that the only way for these excess pore water pressures to dissipate is through upward water flow towards the ground surface. Indeed, a gradual increase in the excess pore water pressures is clearly seen in the top part of the
deposit, in the two layers immediately below and also above the water table. The excess pore water pressures in these layers are partially shear–induced (due to the shaking itself) but a large part of them is seepage–induced as a result of the mechanism described above. To further illustrate these response features, excess pore water pressure (EPWP) ratio time histories at characteristic depths, below and within the critical layer are shown in Fig. 7. The progressive increase in EPWP in the upper soils due to water seeping from the critical layer can clearly be seen after \( t \approx 20 \) s. Under these conditions of strong upward flow of water towards the ground surface, surface liquefaction manifestation is highly probable, even for this case of a thin and not severely liquefied layer.

**Figure 7.** Time-histories of computed excess pore water pressure ratios in the NY-3 model using the 22Feb2011 input motion: within (z = 3.9 m), above (2.4 m) and below (4.6 m) the liquefied critical layer

With respect to the predicted maximum strain, it is important here to note that apart from the relatively high density of the liquefied layer, the short duration over which the layer stays at a nearly zero effective stress state, that is, less than 5 s according to Fig. 7, must have also contributed to the low level of maximum strain predicted by the analysis. Therefore, it can be concluded that in addition to the soil’s stress–density state and the timing of liquefaction and its consequences on the seismic demand discussed in the previous paragraphs, the flow of water into and from the liquefied layer including the extent of the water flow effects also play a crucial role in strain development and consequences of liquefaction.

**CONCLUSIONS**

Results from a series of effective stress analyses were used to identify and discuss response mechanisms of Christchurch soil deposits with varying liquefaction performance and surface manifestation during the 2010–2011 Canterbury Earthquake Sequence. Key findings from this study are summarized in the following:

1) Three predominant types of soil profiles were identified within the No/Yes group of sites (sites that did not manifest liquefaction in the 4 September 2010 event but did manifest liquefaction in the 22 February 2011 earthquake), represented by the three soil profiles analyzed in this study, NY-1, NY-2 and NY-3. The NY-1 type of sites are highly stratified deposits of low-resistance liquefiable and non-liquefiable soils, similarly to the No/No sites (sites that did not manifest liquefaction in any event). The NY-2 type of sites are characterized by relatively thick critical zones of low liquefaction resistance at shallow depth, underlain by clean sands of higher liquefaction resistance at larger depths. The main difference between the NY-2 and Yes/Yes sites (sites that manifested liquefaction in both earthquakes) lies in the properties of the surface crust which is often thicker in the case of the NY-2 type of sites and may also include non-liquefiable, plastic soil layers. The NY-3 type of sites are sites largely composed of clean sands of relatively higher liquefaction resistance, occasionally interrupted by thin, lower strength, and often fines–containing soils creating critical layers at various depths.

2) System–response mechanisms involving interactions between different layers in the dynamic response and through pore water redistribution and water flow are always present in the seismic response of natural soil deposits and often play a key role in the occurrence and severity of surface manifestation of liquefaction. Excess pore water pressure redistribution and water flow from deeper soils toward the liquefied critical zone and from the liquefied critical layer toward the ground surface seem to be the key elements that led to surface liquefaction manifestation in the 22 February 2011 for NY-2 and NY-3 sites, respectively. The early softening and liquefaction of a layer at 4.4 m depth in the 04Sep2010 analysis of the NY-1 model is what hindered
further increase in the seismic demand at shallow soils, preventing liquefaction of the critical layer at 2.1 m depth, which would likely cause surface manifestations.

3) Particular emphasis should be placed on the timing and the location(s) of first occurrence of liquefaction. The timing of the onset of liquefaction is important because the consequences of liquefaction (i.e. deformations and associated damage), are better correlated to the damage potential of the portion of the ground motion that remains after triggering of liquefaction (e.g. Kramer et al. 2016). Then, after the initiation of liquefaction at a particular depth in the deposit, the (remaining) seismic demand imposed on all layers above that depth is largely affected by liquefaction–induced isolation effects. Therefore, the response of a deposit and consequently the severity of surface liquefaction manifestation and associated damage are strongly influenced by whether liquefaction is triggered early or late in a particular ground motion and also by the depth at which it is firstly triggered.

4) Strain development of a soil layer at a given stress–density state, is strongly affected not only by the timing of liquefaction as discussed in item 3, but also by changes in the drainage conditions. Earthquake shaking and related liquefaction induce pore water gradients and water flow that result in contraction of the soil skeleton with outflow of pore water in some areas, and expansion with inflow of pore water, in other areas. Under such conditions, the soil’s strain potential when sheared may significantly differ from that evaluated assuming fully undrained conditions (e.g. Sivathayalan & Logeswaran, 2007).

The effective stress analyses presented herein emphasize the need to consider system response of deposits when evaluating liquefaction and associated damage. In fact, they demonstrate the governing influence of system response effects on liquefaction manifestation and emphasize the need to incorporate such considerations in the simplified liquefaction evaluation procedures, in which, currently, each layer is considered in isolation, and system processes are not rigorously quantified.

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