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# POST-LIQUEFACTION SETTLEMENTS OF STRUCTURES

**Winfried Schepers[1], Stavros A. Savidis[2,3], and Norbert J. Krutzik[4]**

[1] Project Manager, GuD Geotechnik und Dynamik Consult GmbH, Berlin, Germany
[2] Senior Partner, GuD Geotechnik und Dynamik Consult GmbH, Berlin, Germany
[3] Professor, Chair of Soil Mechanics and Foundation Engineering,
Berlin Institute of Technology, Germany
[4] Director, NJK Consulting, Frankfurt, Germany

## ABSTRACT

Seismic liquefaction is a phenomenon which even occurs in regions of low seismic risk, and which occurred even in recent decades, see Nieuwenhuis (1994). It is therefore a load case which has to be taken into account in the design of safety related structures. If a site is susceptible to liquefaction, then either costly deep foundations have to be designed, or the site is to be classified as not eligible for the planned structure.

In practice, occasionally a liquefaction potential analysis must be performed even for an existing building. If that site turns out to be susceptible to liquefaction, then either the potential impact of liquefaction induced soil deformations on the building has to be evaluated, or geotechnical measures to mitigate that impact must be designed and executed.

This paper starts with a summary of methods and procedures to incorporate the favorable and unfavorable contributions of buildings on shallow foundations to the liquefaction potential. Then, a procedure to evaluate the impact of liquefaction induced settlements in terms of soil densification of the subground, which is currently implemented in practice, is outlined. Finally, some applicable geotechnical liquefaction mitigation methods are presented.

## INTRODUCTION

Current geotechnical seismic design codes generally require that the liquefaction susceptibility of a site is investigated during the design stage, and that appropriate measures are undertaken to mitigate the liquefaction hazard. Occasionally, however, a liquefaction hazard assessment has to be performed for existing buildings. The reason might be that new findings emerged about the actual seismic hazard of the site, or that the lifetime of a structure is to be extended above the planned lifetime and hence stronger seismic events must be considered, amongst other reasons.

Here we restrict our considerations to buildings on shallow foundations. Potential liquefaction induced failure modes of buildings on shallow foundations have been summarized by Bray and Dashti (2012). They distinguish volumetric-induced displacements

- localized volumetric strain due to partial drainage
- displacements due to sedimentation after liquefaction
- consolidation-induced volumetric strain due to excess pore pressure dissipation after shaking

and shear-induced displacements

- punching settlements and tilting under the static load due to strength loss
- soil-structure interaction induced cumulative ratcheting due to cyclic loading of foundation

triggered by liquefaction.

At the cases at hand only buildings with a ratio height/width $\ll 1$ founded on large baseplates were involved. Hence, shear induced displacements are of significantly less concern than volumetric displacements. Furthermore, sedimentation, that is sand boils and similar phenomena, as well as partial drainage is of little concern, too, due to the large horizontal extent of the baseplates. What remains of interest are the displacements due to consolidation-induced volumetric strain, also known as post-liquefaction settlements.

Simplified methods to assess the liquefaction potential of free-field sites are available since the early 1970s. The seminal work was published by Seed and Idriss (1971), and has been updated and extended continuously since then. Those methods are developed from statistical evaluations of liquefaction case histories, and have meanwhile been incorporated in design codes in one or the other form, e. g. NRC RG 1.198 , EN 1998-5, and KTA 2201.2.

The simplified methods are generally applicable to level ground conditions only. The main reason is that the liquefaction case histories from which the methods have been developed are actually level-ground cases, without buildings or other topographic irregularities. Furthermore, the impact a building has on the liquefaction potential is very complex.

Regulatory provisions provide very little guidance on the estimation of settlements due to liquefaction. For example, NUREG/CR-574, as the technical basis for NRC RG 1.198, refers with one paragraph of text only to the procedure of Tokimatsu and Seed (1987) as being appropriate for liquefaction settlement hazard screening purposes, while, for example, EN 1998-5 and KTA 2201.2 provide no guidance about liquefaction induced settlements.

For the numerical simulation of liquefaction phenomena by means of finite element computations the mechanical properties of the soil skeleton as well as the interaction with the pore water must be properly modelled. That requires a constitutive model for the soil skeleton on the one hand, and a coupled fluid-solid finite element for saturated soil on the other hand. Significant progress can be observed on this subject in recent years, see e. g. Yang et al. (2004), Savidis et al. (2013), and Savidis et al. (2014), and it can be observed from the literature that it is possible to simulate centrifuge experiments with reasonable accuracy, see Chakrabortty and Popescu (2012). For real-world liquefaction problems, however, fully numerical methods are still not applicable, despite the huge advances in computer technology. Hence empirical methods have to be applied.

According to Wu and Seed (2004) the steps required to perform an empirical liquefaction settlement analysis are as follows:

1. Evaluation of liquefaction susceptibility for each saturated soil layer or sub-layer, that is compute the cyclic resistance ratio
2. Estimate post-liquefaction settlements using an appropriate correlation between cyclic stress and liquefaction resistance of each soil layer. If appropriate, corrections for overburden stress are to be taken into account
3. For dry soil layers, estimate the seismic settlements using an appropriate correlation between cyclic stress and volumetric strain
4. Sum the volumetric changes of all soil layers and sub-layers to get the total estimated ground settlement

5. If lateral displacements are to be expected, the vertical displacements have to be corrected by the vertical settlement component of the primarily lateral displacements (e.g. lateral spreading)

## SIMPLIFIED METHOD FOR LIQUEFACTION SUSCEPTIBILITY ASSESSMENT

Updates and extensions to the seminal publication of Seed and Idriss (1971) have been published by Seed (1979), Youd et al. (2001), Finn (2002), Seed et al. (2003), Cetin et al. (2004), Idriss and Boulanger (2008), and Boulanger and Idriss (2014), to name a few. It should be noted that to none of the aforementioned publications the same level of acceptance has been awarded by the geotechnical earthquake engineering community as to the method after Youd et al. (2001), because it represents a consensus reached during two workshops in 1996 and 1998 attended by experts of highest reputation in geotechnical earthquake engineering. Furthermore, it is commonly considered as the latest undisputed publication of a simplified method. Later, some scientific dissent emerged, see Youd (2011).

Following basic concepts of structural design, the safety against liquefaction *FSL* is defined as the ratio of the soil's resistance over the seismic demand. In the simplified method, the resistance is termed the "cyclic resistance ratio" (*CRR*), while the seismic demand induced by the design earthquake is termed "cyclic stress ratio" (*CSR*).

$$Factor\ of\ Safety\ against\ liquefaction = \frac{Cyclic\ Resistance\ Ratio\ (CRR)}{Cyclic\ Stress\ Ratio\ (CSR)} \tag{1}$$

The CRR is commonly normalized to an earthquake of magnitude $M_w = 7.5$, level ground, i. e. no horizontal static shear stress, $\sigma'_0 = 100$ kPa effective overburden pressure, and freshly deposited clean sand layers not yet exhibiting any ageing effects. To account for different earthquake magnitudes, a magnitude scaling factor *MSF* has to be applied. Additional corrections are applicable for high overburden stress ($K_\sigma$), static shear stress ($K_\alpha$), and age of deposit ($K_{\mathrm{DR}}$). The final equation for the Factor of Safety against liquefaction then reads

$$FS = \frac{\mathrm{CRR}_{\mathrm{M_W}=7.5,\sigma'=100\ \mathrm{kPa},\alpha=0°,DR\approx 0\ a}}{\mathrm{CSR}} \cdot MSF \cdot K_\sigma \cdot K_\alpha \cdot K_{\mathrm{DR}} \tag{2}$$

To account for fines content, the results of SPT or CPT tests are further normalized in order to obtain an equivalent clean sand blow count and tip resistance, respectively. Then, from a chart providing the relationship between normalized clean sand blow counts and tip resistance, the required CSR to initiate liquefaction and the actual FS can be obtained. Such charts can be found in any of the publications about the simplified method mentioned above.

The simplified methods are capable of taking into account a building in the following ways:

- the increased overburden pressure due to the weight of the building will change the CSR,
- the weight of the building will be detrimental for the CRR the higher the additional overburden stress, which is accounted for by the overburden stress correction factor $K_\sigma$.
- the additional static shear stress in the soil is accounted for by the shear stress correction factor $K_\alpha$.

The simplified methods do not take into account:

- kinematic soil-structure interaction, that is wave reflection and wave refraction at the soil-structure interface,

- inertial effects, that is an additional wave field in the soil excited from the inertial forces resisting the seismic excitation, as well as the period lengthening of the combined soil-structure system

In Table 1 the influence of some factors on the factor of safety against liquefaction is summarized. There, Δσ is the additional vertical stress due to the weight of the structure. It becomes obvious that a building might have as well a beneficial, as well as a detrimental effect on the safety against liquefaction. The influence of the static horizontal shear stress is particularly remarkable, because it depends on the relative density of the sand deposit. The lower the soil's density, the more detrimental is the static shear stress.

For the case at hand – low-rise, nearly rigid structures – no final statement can be made about the impact of a building on liquefaction resistance. While the soils resistance against shear deformations decreases with increasing overburden stress, the liquefaction resistance increases due to the increased inter-particle friction, while the impact of the static shear stress, which is largest at the building's boundaries, depends on the soil's relative density.

Table 1: Influence of various factors on factor of safety against liquefaction for low-rise buildings on shallow foundation, after Rollins and Seed (1990)

| Case | $K_\alpha$ | $K_\sigma$ | $\Delta\sigma$ |
|---|---|---|---|
| On $D_r$ = 55 % sand | Large increase | Small decrease | Moderate increase |
| On $D_r$ = 45 % sand | No effect | Small decrease | Moderate increase |
| On $D_r$ = 35 % sand | Small decrease | Small decrease | Moderate increase |

The seismic demand can be computed using

$$CSR = 0.65 \cdot \frac{a_{max}}{\mathrm{g}} \cdot \frac{\sigma_{v0}}{\sigma'_{\mathrm{v0}}} \cdot r_{\mathrm{d}} \tag{3}$$

with $a_{max}$ the peak ground acceleration at the surface, g the acceleration of gravity, $\sigma_{v0}$ the total static vertical stress, and $\sigma'_{\mathrm{v0}}$ effective static vertical stress. The factor $r_{\mathrm{d}}$ accounts for the flexibility of the soil column, being equal to unity at the surface and decreasing with increasing depth. Results of 1D and 2D site response analyses indicate that for rigid buildings embedded in soil layers of similar soil types it is reasonable to set $r_{\mathrm{d}} = 1$ between ground surface and embedment depth, and to use either generic curves or results of site response analyses below. This is in particularly true if the mass of the building and the mass of the excavated soil are approximately equal.

## LIQUEFACTION INDUCED SETTLEMENTS ESTIMATION

The state-of-the-practice still largely involves estimating building settlement using empirical procedures developed to calculate post-liquefaction consolidation settlement in the free-field. This approach cannot possibly capture shear-induced and localized volumetric-induced deformations in the soil underneath shallow foundations, which primarily occur during earthquake loading. Currently, simplified procedures that directly address this problem are not available. Due to the particular circumstances for the structures at hand shear-induced deformations were of little concern.

Two approaches to obtain the volumetric strain due to excess pore pressure dissipation can be used:

- Simplified methods based on evaluation of case histories
- Cyclic laboratory tests

If existing buildings currently in use are the subject of interest, then it is generally not possible to obtain any soil samples below the building, nor is it possible to perform SPT or CPT soundings below the building. If the building is rather small such that soil samples from outside the building can be considered as representative for the soil below the building, then cyclic laboratory tests with undisturbed samples are the preferred method, because the in-situ static stress state, as well as the site-specific seismic impact can be applied. Potential test types and test devices are presented in Savidis et al. (2013). For one of the cases at hand cyclic simple shear tests were performed.

At large buildings the use of simplified methods can hardly be avoided. For our purposes the method after Zhang et al. (2002) was applied, because results from the CPT-based liquefaction assessment can be re-used straightforwadly. Zhang et al. (2002) provide a relationship between the equivalent clean sand normalized CPT tip resistance on the one hand, and the postliquefaction volumetric strain on the other hand, with the factor of safety against liquefaction FS being a curve parameter. A drawback of these charts, either from Zhang et al. (2002) or from other authors, is the fact that those curves have a strong gradient near FS = 1. Hence, small changes in FS might have a strong influence on the value of volumetric strain.

To apply a simplified method for estimating volumetric strain, some interpolation scheme must be applied to obtain tip resistance and sleeve friction from below the building, because in-situ tests are available only from the close vicinity of the building. For the cases at hand, first available CPT results have been simplified such that for each geologic layer a constant value for tip resistance and sleeve friction was obtained. That procedure was chosen such that at the locations of the CPT soundings the settlements were close if either the raw CPT data or the simplified CPT data was used.

Second, radial basis functions with inverse multiquadric shape functions (RBF-IMQ) have been used to interpolate tip resistance and sleeve friction between CPT locations and outside. RBF-IMQ is suitable here because it is essentially a “local” interpolation, inasmuch as the weight of a value at a CPT location is inversely proportional to the distance between the evaluation point and the CPT location. Indeed, from soil properties being known at a certain location it is not reasonable to make estimates of the soil properties in some distance, and this is the less reasonable, the further away that location, and this is simulated by RBF-IMQ.

Third, a simplified liquefaction analysis and a consequent simplified analysis for a dense 3D grid of locations within the soil volume has been performed. This is – fourth – followed by an integration of volumetric strain over depth for a dense grid of locations at the soil surface and the foundation depth, respectively.

## LIQUEFACTION MITIGATION METHODS FOR EXISTING BUILDINGS

Hausler and Sitar (2001) reviewed more than 90 case studies of liquefaction mitigation methods during 14 seismic events of magnitude 6.3 to 8.1. The result is summarized in Table 2.

Table 2: Soil improvement methods for liquefaction mitigation case studies, after Hausler and Sitar (2001)

| Method | Performance (Acceptable/Unacceptable) | Average Increase in $N_{1,60}$ |
|---|---|---|
| *Densification through vibration and compaction* | | |
| Sand compaction piles | 26/5 | 11 |
| Deept dynamic compaction | 15/0 | 5 |
| Vibrorod/vibroflotation | 11/6 | 13 |
| Stone columns | 7/1 | 8 |
| Preloading | 5/0 | 5 |
| Compaction grouting | 1/1 | n/a |
| Time displacement piles | 1/0 | n/a |
| *Dissipation of excess pore water pressure* | | |
| Gravel drains | 5/0 | 7 |
| Sand drains | 5/0 | 9 |
| Wick or paper drains | 2/0 | n/a |
| *Restraining effect through inclusions* | | |
| Deep soil mixing | 4/1 | n/a |
| Diaphragm walls | 0/1 | n/a |
| *Stiffening through chemical or cement addition* | | |
| Jet grouting | 5/0 | n/a |
| Chemical grouting | 10/ | n/a |

Methods suitable for existing buildings generally require that no actions are necessary from the buildings inside. Hence any method which requires drilling, piling, or similar procedures are not applicable. Hence only the last two groups of methods are suitable for existing buildings, and both group show a promising performance.

Two potential liquefaction mitigation methods for existing buildings are discussed in what follows.

***Fracture Grouting***

From a circular shaft sleeve port pipes are fanned out into the subsoil below the building in predrilled holes. Through the ports a fluid grout appropriate for the respective soil type is injected at high pressure into the soil. Initially the ground injection prestresses the soil locally and voids are filled. Once the injection pressure exceeds the soils' stress state, fracture occur and the grout propagates further into the soil. Repeated application of that procedure creates a lattice of grout fissures extending up to 1 m from the injection point, and provides a reinforcement of the soil, see figure 1.

Because the drillings for the sleeve pipes can be performed horizontally from outside the building into the subsoil below the building over a distance up to 100 m, fracture grouting in particularly attractive as a liquefaction mitigation method for existing buildings. Since fracture grouting increases the soil volume and might hence lead to heaving of the building, the amount of grout injected must be carefully controlled by permanent deformation measurements of the building structure.

Figure 1: Lattice of grout filled fractures in soil by fracture grouting

***Rigid Containment Walls***

The performance of rigid vertical walls as liquefaction mitigation measure has recently be investigated by Mitrani and Madabhushi (2012) in centrifuge tests. The concept is sketched in figure 2.

Figure 2: Stiff and impermeable wall enclosing the shallow foundation of a building on liquefiable soil, after Mitrani and Madabhushi (2012)

The confining wall prevents the motion of soil particles inside the soil body enclosed by the wall, as long as the liquefying layer is located above the wall toe. Though the settlements in the liquefying layer will impose some downward drag forces on the outer face of the wall, these forces are resisted by the non-liquefied soil at the wall toes. But because lateral motions of soil grains within the soil body enclosed by the walls are suppressed, soil particles cannot be rearranged, and hence pore water pressure development is prevented or strongly reduced, see figure 3.

Figure 3: Liquefaction mitigation by rigid walls, after Mitrani and Madabhushi (2012)

A potential procedure for the installation of the rigid walls is Deep-Soil Mixing (DSM). Special tools create a mixture of soil and a suitable suspension, and hence no harmful displacements and vibrations will occur during installation of the wall. The applicability of Deep-Soil Mixing as a liquefaction mitigation method has been proven in the past. Porhaba et al. (1999) report about the Oriental Hotel in Kobe, Japan, in which DSM columns installed below the foundation in liquefiable soil completely prevented liquefaction in an otherwise highly liquefaction susceptible soil during the 1995 Kobe earthquake.

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