**Proceedings of the XVIII ECSMGE 2024**
GEOTECHNICAL ENGINEERING CHALLENGES
TO MEET CURRENT AND EMERGING NEEDS OF SOCIETY

ISBN 978-1-032-54816-6
DOI 10.1201/9781003431749-241


# Performance design of shallow foundations interacting with liquefiable soils

## Performance design des fondations superficielles sur sols liquéfiables

I. Sarao Mannelli*, V. Minardi, G. Canetta
*CEAS S.r.l., Milan, Italy*

B. Becci
*IN.SPE. S.r.l., Bergamo, Italy*

**irene.saraomannelli@ceas.it*

**ABSTRACT**: Dealing with foundation soils that are likely prone to liquefaction, or at least cyclic mobility during Ultimate Limit State earthquakes, some relevant uncertainties may arise in selecting appropriate remedial schemes. While it is a most common practice to consider soil improvement works, in some practical scenarios such ordinary policy may result in unreasonable choices under both engineering and economical points of view. A typical case is the design of spread foundations or buried caissons of limited size, in moderately liquefaction prone soils. A case history is discussed in this contribution, related to a touristic infrastructure installation in a reclaimed area, at the seashore of Northern Adriatic Coast, Italy. Ordinary free-field, one-dimensional liquefaction analyses have been compared with results from numerical models based on Flac3D, implementing P2PSand constitutive model (Cheng and Detournay, 2021), capable of reproducing pore pressure build up. The seismic response has been predicted and maximum displacements as well as design stresses for the structure have been evaluated; consequently, the Performance Based Design Approach has allowed to assess the minimum countermeasures required, ensuring an appropriate safety level even without the need of huge soil improvement works.

**RÉSUMÉ**: Dans le cas de sols susceptibles à la liquéfaction, ou au moins à la mobilité cyclique en cas de séisme à l'état limite ultime, certaines incertitudes pertinentes peuvent survenir lors de la sélection des programmes de remédiation appropriés. Bien qu'il soit courant d'envisager des travaux d'amélioration du sol, dans certains scénarios pratiques, cette politique ordinaire peut aboutir à des choix déraisonnables d'un point de vue technique et économique. Un cas typique est la conception de fondations étalées ou de caissons enterrés de taille limitée, dans des sols modérément sujets à la liquéfaction. Dans cet article est explique un cas type, lié à une installation d'infrastructure touristique dans une zone de remise en état, au bord de la mer au nord de la côte Adriatique, en Italie. Les analyses de liquéfaction unidimensionnelles ordinaires en champ libre ont été comparées aux résultats numériques du logiciel Flac3D, en mettant en œuvre le modèle constitutif P2PSand (Cheng et Detournay, 2021), capable de reproduire l'accumulation de la pression interstitielle. La réponse sismique a été prédite et les déplacements maximaux ainsi que les contraintes de conception pour la structure ont été évalués; par conséquent, l'approche de méthodes de conception fondées sur la performance a permis d'évaluer les mesures minimales requises, afin de garantir un niveau de sécurité approprié même sans la nécessité d'énormes travaux d'amélioration du sol.

**Keywords:** Liquefaction phenomenon; performance design.

## 1 INTRODUCTION

The liquefaction phenomenon is a quite common issue concerning the reduction or the loss of soil stiffness and strength caused by earthquake shaking.

It takes place in loosely packed, water-saturated sediments at or near the ground surface. Liquefaction occurring beneath buildings and other structures can cause major damage during earthquakes.

Therefore, in order to mitigate the liquefaction hazards several techniques have been implemented. The ordinary methods include vibroflotation, compaction grouting, stone columns. The objective is to improve the strength, the density of the soil so that soil skeleton will not collapse under rapid loading and possibly improve the drainage characteristics of soils.

These common practices, in some scenarios, may result in unreasonable choices under both engineering and economical points of view. This is the case for lightweight buildings. In this respect, this proceeding provides a case history in which a different approach has been adopted based on Performance Design Approach. Following this procedure, as done by many researchers such as Murashev, Keepa et Tai, (2015) and Liu, Macedo et Candia (2021), the soil is not improved while the structure is designed to resist the effects of liquefaction, in terms of internal forces due to settlements.

The evaluation of those settlements, in the state of practice still largely involves 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. Two dimensional or three dimensional soil structure interaction nonlinear effective stress dynamic analysis can provide a reliable estimates of building movement. Lopez-Caballero and Farahmand-Razavi (2008), Shakir and Pak (2010), Adrianopoulos et al. (2006), Dashti and Bray (2013), and Karimi and Dashti (2016a,b) perform numerical analysis in this field.

## 2 CASE STUDY

The effect of liquefaction on ancillaries, lightweight buildings, related to a touristic infrastructure installation in a reclaimed area, at the seashore of Northern Adriatic Coast, Italy, will be examined in this study.

At this purpose, several numerical analyses were carried out, starting from stratigraphic column model and comparing it with simplified models. The stratigraphic column was compared with a subsequent two-dimensional model under free-field conditions. And finally, the influence of the light building on the soil layer was analysed.

The aims of the seismic numerical analyses are, therefore, to:

- check the triggering of the phenomenon and compare the results obtained with the results of the empirical methods;
- assess the interaction between the liquefaction phenomenon and the foundation structures, in terms of maximum displacement and stresses acting on the foundation.

### 2.1 FLAC3D analysis: procedure

Analyses were carried out with the Flac3D finite difference code, implementing the 'P2PSand' constitutive model (Cheng and Detournay, 2021). The model is specially formulated and calibrated to reproduce main experimental evidences of the liquefaction phenomenon. It defines the deformability of the material, in terms of shear modulus, as a function of relative density by means of the following expression:

$$G = G_0 \, P_{atm} \left(\frac{p'}{P_{atm}}\right)^{0.5} \qquad (1)$$

$$G_0 = g_0 \left(D_R + C_{D_r}\right) P_{atm} \left(\frac{p'}{P_{atm}}\right)^{0.5} \qquad (2)$$

where, for clean and uniform sands, it is possible to assume: $g_0 = 1240$ e $C_{D_r} = 0.01$ ($P_{atm} = 101.3\ kPa$).

The main calibration parameters of the constitutive model are the relative density $D_R$ and the shear modulus multiplier $g_0$. These parameters were calibrated to reproduce the design shear modulus along depth.

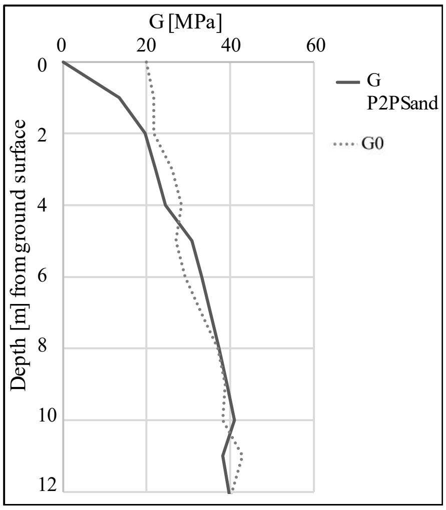


*Figure 1. Calibration of shear modulus with P2PSand model.*

The stratigraphy at the site can be modelled by four geotechnical units, defined as “R”, “A”, “B” and “C”. The liquefaction prone sandy sediments of geotechnical units R and A are modelled with the P2PSand constitutive model. Whereas Mohr-Coulomb model is used for geotechnical units B and C. The bottommost meter at the base of the model is modelled as elastic material in order not to have input wave distortion or noise in the application of seismic stress.

The main geotechnical input parameters are reported in Table 1 and Table 2. Water table is 2m below the ground surface.

*Table 1. Geotechnical input parameters: P2PSand.*

| Unit | z [m] | γ [kN/m³] | φ [°] | Dr [%] | Vs [m/s] |
|---|---|---|---|---|---|
| R | 0 ÷ 5 | 19 | 30 | 30 | 125 |
| A | 5 ÷ 10 | 19 | 30 | 35 | 161 |

*Table 2. Geotechnical input parameters: Mohr-Coulomb.*

| Unit | z [m] | γ [kN/m³] | φ [°] | c' [kPa] | E [MPa] |
|---|---|---|---|---|---|
| B | 10 ÷ 29 | 19 | 25 | 5 | 25 |
| C | 29 ÷ 39 | 20 | 33 | 30 | 70 |

REXEL, the database provided by ITACA, was used to define the seismic input: seven seismocompatible and spectrum-compatible accelerograms.

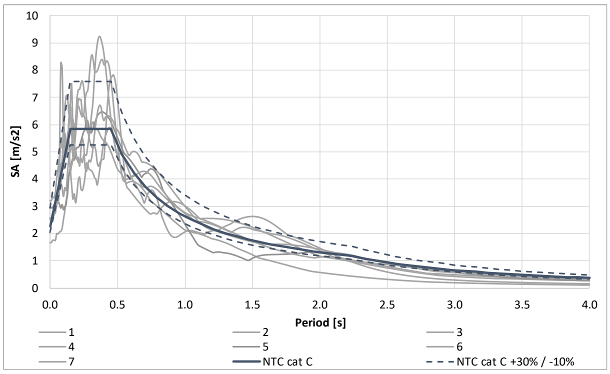


*Figure 2. Response spectra of accelerograms extracted from REXEL. Comparison with NTC2018 spectrum.*

The boundary conditions imposed on lateral sides of the numerical model are of the free-field kind. They consist of inserting a small column of soil at the extremes of the model to prevent both the distortion and the reflection of the seismic waves.

At the base of the model the following conditions were applied:

- The compliant base boundary conditions, which are implemented with independent dashpots in the normal and shear directions. They dampen the seismic wave, preventing its reflection within the model;
- the design velocity history, applied as time-dependent tangential stress:

$$\tau(t) = -\rho \, V_s \, v(t) \qquad (3)$$

where $\rho$ and $V_s$ are the density and shear wave velocity of the soil at the base of the model. v(t) is the seismic velocity diagram, obtained by integrating the deconvoluted design accelerograms.

The potential for liquefaction can be assessed by analysing the $R_u$ coefficient, defined as the ratio of pore pressure increment $\Delta u$ to geostatic vertical effective stress:

$$R_u = \frac{\Delta u}{\sigma'_{v0}} \qquad (4)$$

Soil is considered not subject to liquefaction if $R_u<0.95$.

The calculation phases implemented in the model are as listed below:

1. Initialisation of the geostatic stress state with the Mohr-Coulomb constitutive model;
2. Application of the P2PSand constitutive model on the geotechnical units R and A;
3. Dynamic analysis by applying the history of $\tau(t)$ of the 7 design accelerograms at the base of the model.

## 2.2 1D model: stratigraphic column

The numerical model is defined by a column with planimetric dimensions 2.0 m x 2.0 m (X and Y axes), and a height of 40.0 m (Z axis).

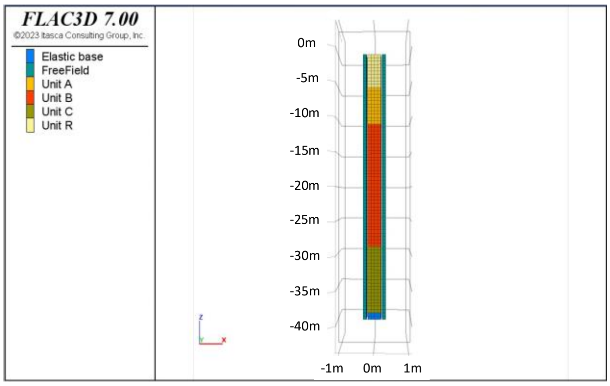


*Figure 3. 1D model.*

Figure 4, on the left, illustrates the 7 profiles of the $R_u$ envelopes with depth, related to the 7 design accelerations; the profiles are sampled at the centre of the column. On the right, shows results from a common empirical approach (Robertson, 2009) for CPTus conducted in the same site. The factor of safety ($F_s$) against liquefaction is assessed by CRR (Cyclic Resistance Ratio) and CSR (Cyclic Stress Ratio) ratio.

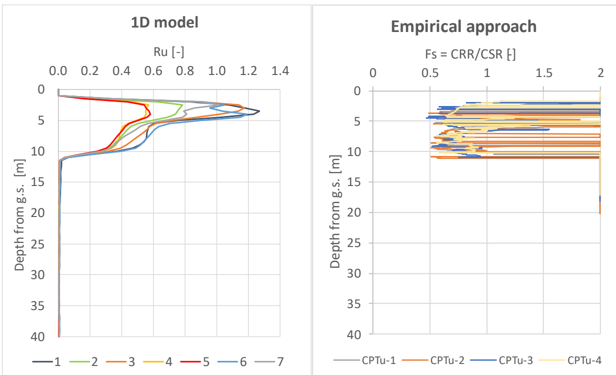


*Figure 4. Results from numerical model (left) and empirical approach (Robertson, 2009).*

It is noticeable that, by the empirical approach, geotechnical Unit R and A are prone to liquefaction ($F_s$ <1), while the 1D model indicates probability of liquefaction just for Unit R, associated with a $R_u$>0.95 factor.

## 2.3 2D model: free field

The numerical model is defined by a two-dimensional block with planimetric dimensions 40.0 m x 0.5 m (X and Y axes), and a height of 40.0 m (Z axis).

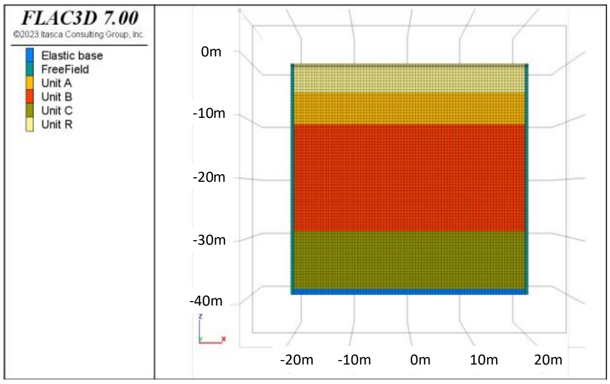


*Figure 5. 2D model in free field conditions.*

The 2D free field model confirm results observed in 1D model: soil belonging to unit R, as shown in Figure 6, when subjected to the seismic design stresses, induced by accelerograms 1, 3, 6 and 7, is liquefiable.

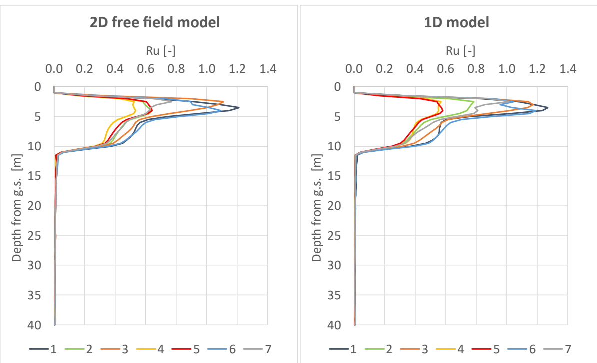


*Figure 6. Comparison of free field 2D model (left) and stratigraphic column (right).*

## 2.4 2D model: interaction with the shallow foundation

Based on free-field analyses, which reveal the occurrence of the liquefaction phenomenon, it is deemed necessary to further investigate the problem by modelling the on-site condition associated with the presence of some light buildings.

The building slab dimensions are B=9.5m and H=0.6m. It is modelled with elastic solid elements, having a specific weight equal to that of concrete ($\gamma$=25 kN/m$^3$), and a stiffness 20 times that of the surrounding soil (E=300 MPa).

The presence of the superstructure is simulated with elastic solid elements of equivalent weight and stiffness equal to that of the surrounding soil. The load due to the superstructure is 7.1 kPa, corresponding to an equivalent specific weight of 23.6 kN/m$^3$.

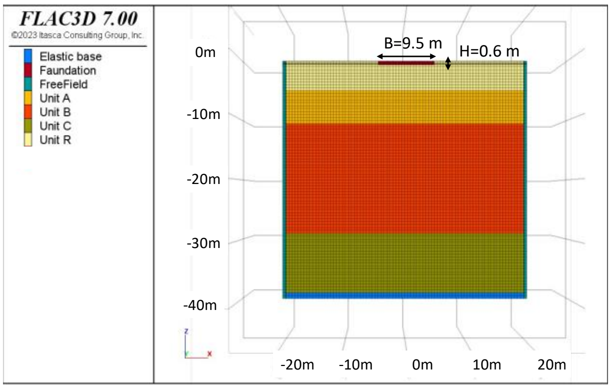


*Figure 7. 2D model with shallow foundation.*

In addition to the stages previously mentioned, three further stages run before phase 3: i) modelling the foundation slab; ii) applying the loads coming from the superstructure; iii) resetting the displacements before carrying out the dynamic analysis.

Obtained $R_u$ profiles with ancillary building are compared with free-field profiles.

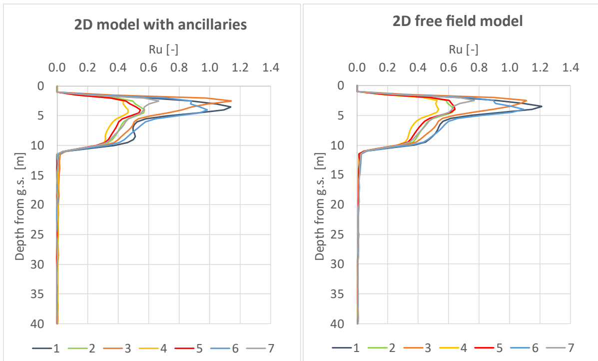


*Figure 8. Comparison of model with ancillaries (left) and free-field model (right).*

The presence of the building reduces the maximum value of $R_u$: this result is consistent with the fact that the building load increases the effective stresses near the foundation, reducing $R_u$ ratio.

The results of the analysis in terms of displacement histories, associated with accelerogram 3, are shown in Figure 9. The upper Figure shows the vertical displacements of: the left end side of the foundation, in green; the point at the centre, in brown; and the right end side, in red. While the bottom one shows the horizontal displacements, which are uniform within the slab, which behaves as a rigid body.

The maximum horizontal displacement, experienced during the earthquake, is of the order of ± 5cm. While the residual one is close to 1 cm. To the contrary, vertical settlements tends to increase with time, gathering some millimiters at mid axes as residual values.

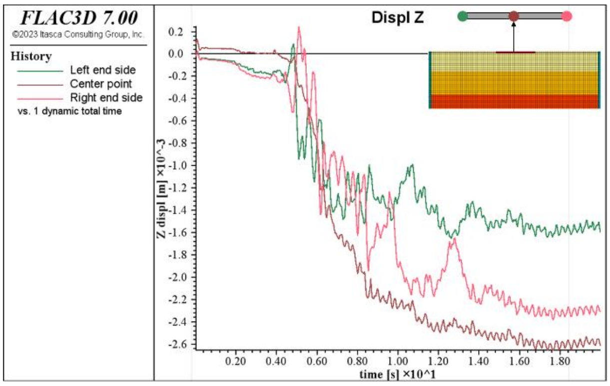


*Figure 9. Time history of vertical displacements - accelerogram 3.*

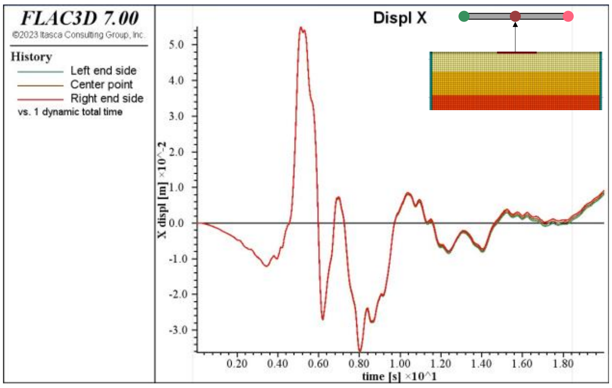


*Figure 10. Time history of horizontal displacement - accelerogram 3.*

The profile of the seven vertical displacements along the slab, associated to the design accelerations, are shown in Figure 11:

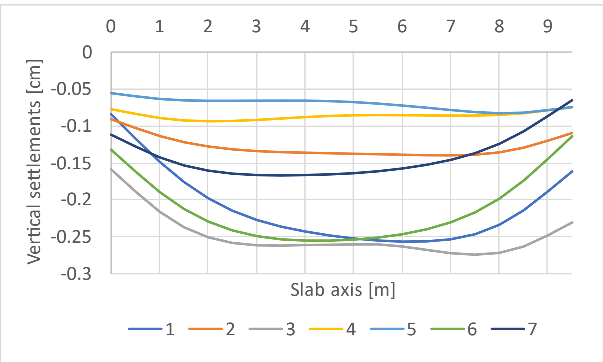


*Figure 11. Vertical settlements along the slab.*

The maximum vertical displacement is of the order of 0.27cm, associated with accelerogram 3. While the maximum differential settlement is of the order of 0.2cm associated with accelerogram 5.

Considering that the liquefaction phenomenon occurs without leading to excessive horizontal displacements and settlements, it has been decided to design foundations in such a way as to resist the additional stresses induced by liquefaction with a Performance Based Design approach.

## 2.5 Performance based design approach

The structure should be designed to resist the effects of liquefaction.

As a result of the analyses, the profiles of the horizontal minimum and maximum $\sigma_{xx}$ stresses and the shear $\sigma_{xz}$ stress acting in the slab elements were obtained.

Hence, the maximum bending moment and shear force were computed. These actions have to be added to the static design actions of the foundation as an additional loading condition due to soil liquefaction. In this way, the designed foundation will be able to resist to the envelope of the maximum expected deformations coming from the liquefaction phenomenon.

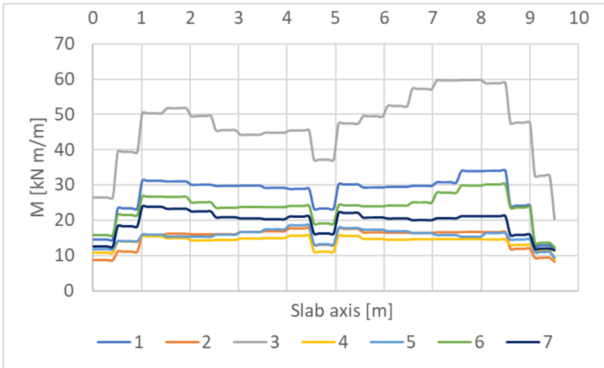


*Figure 12. Bending moment along the slab.*

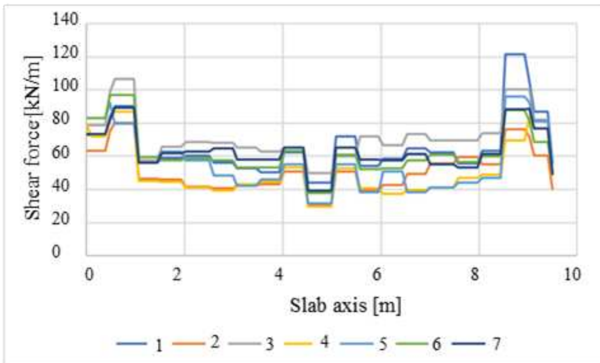


*Figure 13. Shear force along the slab.*

These supplementary actions, in the current project, are moderate. This is due to shallow setting of the foundation and to an $R_u$ factor not far above unity.

Horizontal and vertical displacements of the foundation, due to the liquefaction phenomenon, are therefore compatible with design requirements.

## 3 CONCLUSIONS

FLAC 3D analyses confirmed the risk of soil liquefaction in same soil region where conventional calculations, based on empirical correlation, reported these issues.

However, such numerical approach is providing a relevant added value represented by a more accurate prediction of soil and foundation deformations occurring during seismic event. Such additional result is very important in the light of a Performance Based Design approach. In fact, according to this approach, the design of the foundation has the purpose to resist the effects of liquefaction in terms of deformations and supplementary actions, that are the results of the FLAC 3D analyses.

In the authors' opinion, such improvement in analysis results well counterbalances the increased efforts in terms of computational time and analysis complexities.

A further output of the numerical analysis, not dealt with in this contribution, is the evaluation of the increase of water pressures at the interface between foundation and soil. In case such overpressures may cause instability problems, such as uplift, appropriate countermeasures could be reliably assessed.

## REFERENCES


Andrianopoulos, K., Bouckovalas, G., Karamitros, D., Papadimitriou, A. (2006) Effective stress analysis for the seismic response of shallow foundations on liquefiable sand, In: 6th *European Conference on Numerical Methods in Geotechnical Engineering*, Graz, Austria. Available at: https://www.researchgate.net/publication/253795767_Effective_Stress_Analysis_for_the_Seismic_Response_of_Shallow_Foundations_on_Liquefiable_Sand.

Cheng, Z., Detournay, C. (2021) Formulation, validation and application of a practice-oriented two-surface plasticity sand model, *Computers and Geotechnics,* 132, https://doi.org/10.1016/j.compgeo.2020.103984.

Dashti, S., Bray, J.D. (2013) Numerical Simulation of Building Response on Liquefiable Sand, *Journal of Geotechnical and Geoenvironmental Engineering*, 139(8), https://doi.org/10.1061/(ASCE)GT.1943-5606.0000853.

Flora, A., Bilotta, E., Lirer, S., Mele, L., Modoni, G. (2023) Liquefaction mechanisms and mitigation techniques, *Rivista Italiana di Geotecnica,* 2, pp. 33-88 https://doi.org/10.19199/2023.2.0557-1405.033.

Karimi, Z., and Dashti, S. (2016) Numerical and Centrifuge Modeling of Seismic Soil-Foundation-Structure Interaction on Liquefiable Ground, *Journal of Geotechnical and Geoenvironmental Engineering*, 142(1) https://doi.org/10.1061/(ASCE)GT.1943-5606.0001346.

Itaca - ITalian ACcelerometric Archive - INGV. REXELweb. Available at: https://itaca.mi.ingv.it/ItacaNet_32/#/rexel.

Itasca Consulting Group, Inc. (2023) FLAC3D — Fast Lagrangian Analysis of Continua in Three-Dimensions, Ver. 7.0. Minneapolis: Itasca.

Liu, C., Macedo, J., Cambria, G. (2021) Performance-based probabilistic assessment of liquefaction-induced building settlements, *Soil Dynamics and Earthquake Engineering,* 151, https://doi.org/10.1016/j.soildyn.2021.106955.

Lopez-Caballero, F., and Farahmand-Razavi, A. M. (2008) Numerical Simulation of Liquefaction Effects on Seismic SSI, *Soil Dynamics and Earthquake Engineering*, 49, pp. 2-38. https://doi.org/10.1016/j.soildyn.2007.05.006.

Murashev, A. K., Keepa, C., Tai, A. (2015) Performance Based Design of a Structural Foundation on Liquefiable Ground with a Natural Soil Crust, In: 6th *International Conference on Earthquake Geotechnical Engineering*, Christchurch, New Zealand. Available at: https://www.issmge.org/uploads/publications/59/60/39.00_Murashev.pdf.

Robertson, P. K. (2009) Interpretation of cone penetration tests — a unified approach, *Canadian Geotechnical Journalx,* 46(11), https://doi.org/10.1139/T09-065.

Shakir, H. and Pak, A. (2010). Estimating Liquefaction-Induced Settlement of Shallow Foundations by Numerical Approach, *Computer and Geotechnics*, 37(3), pp. 267-279. https://doi.org/10.1016/j.compgeo.2009.10.001.