Liquefaction Potential Assessment Of Silty And Silty-Sand Deposits: A Case Study

Diego C. F. Lo Presti and Nunziante Squeglia

Department of Civil Engineering, University of Pisa, via Diotisalvi 2 – 56126 Pisa

Abstract. The paper shows a case study concerning the liquefaction potential assessment of deposits which mainly consist of non plastic silts and sands (FC > 35 %, Ip < 10 %, CF negligible). The site under study has been characterized by means of in situ tests (CPTU, SPT and DPSH), boreholes and laboratory tests on undisturbed and remolded samples. More specifically, classification tests, cyclic undrained stress-controlled triaxial tests and resonant column tests have been performed. Liquefaction susceptibility has been evaluated by means of several procedures prescribed by codes or available in technical literature. The evaluation of liquefaction potential has been carried out by means of three different procedure based on in situ and laboratory tests.

Keywords: Liquefaction, cyclic triaxial test.

INTRODUCTION

Since 2003, the Tuscany Seismic Survey has started an investigation plan for retrofitting and repair of existing Public Buildings (Schools, Hospitals etc.) and for the design of the new ones, in the most seismic areas of Tuscany. Investigations for the existing buildings concerned the structure, the structural materials, the geology of the site and the geotechnical characterization of the soil deposits. Obviously for the design of new buildings only geological and geotechnical investigations have been done. Different levels of investigation have been undertaken. The first level consisted in geological surveys and seismic refraction tests in P and SH waves, in order to have geological maps (1:2000) and geological sections. The second level usually consisted in a borehole (at least) with SPT and down-hole measurements. The borehole extended down to the seismic bedrock or at least down to 30 m depth. In some cases, undisturbed samples have been retrieved. In the framework of these activities, the geotechnical investigations undertaken for the construction of a new Primary School located in Fornaci di Barga, in the northern part of Tuscany, indicated that the subsoil was susceptible to liquefaction. Therefore additional investigations have been carried out in order to have a better evaluation of liquefaction susceptibility and liquefaction hazards. The designed building is a one-storey construction with a reinforced concrete cast-in-situ structure. The paper shows the results of the investigations and analyses. Moreover the paper comments on the prescriptions of [1] and [2] in the light of the case study.

GROUND INVESTIGATION

Figure 1 shows the location in plan of preliminary and integrative investigations. The ground investigations consist in: 3 boreholes up to 52 m (S4) or 15 m (S15, S16); 9 Standard Penetration Tests; a down-hole test in borehole S4; a seismic refraction test (ST4); a super heavy dynamic probing (DPSH4) up to 19 m; 3 cone penetration tests, CPTU1, CPTU2 and CPTU3, carried out by means of a piezocone up to 10, 11.3 and 16.4 m, respectively. Nine undisturbed samples have been retrieved from boreholes. Several laboratory tests have been carried out, including resonant column test (CR), torsional shear test (TTC) and triaxial cyclic test (TXC). Table 1 lists the laboratory tests performed on undisturbed samples. In addition to listed tests, several determination of grain size distribution (Fig. 2a) and plastic index (Fig. 2b) have been carried out on remolded specimens.

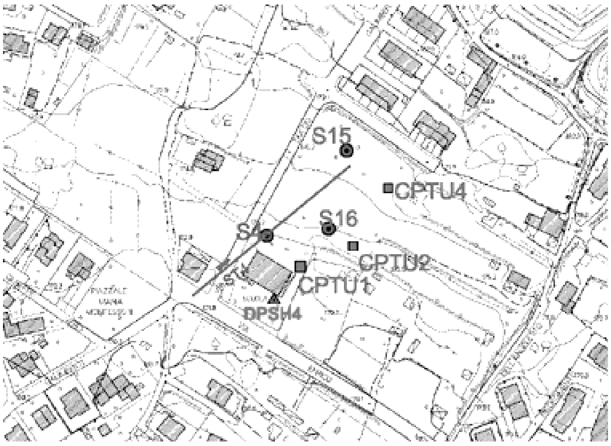

FIGURE 1. Location in plan of ground investigation.

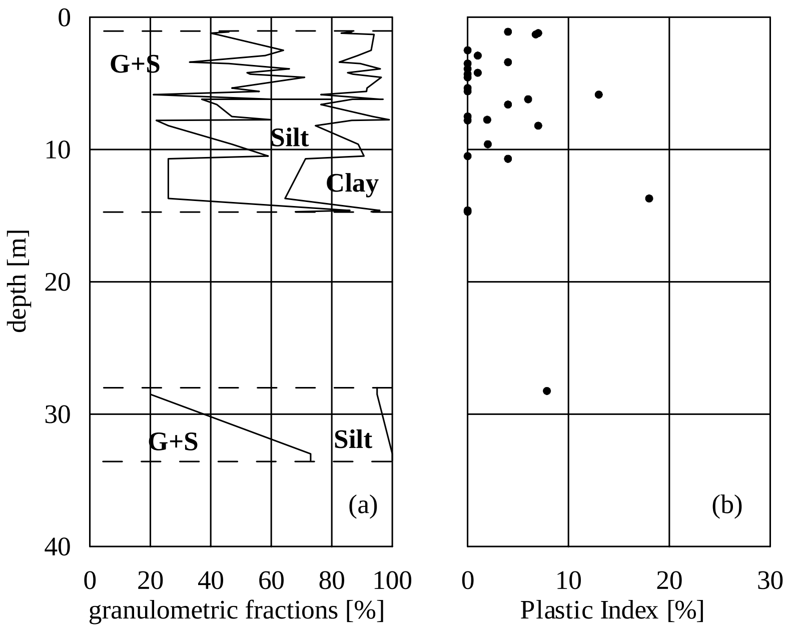

FIGURE 2. Granulometric fractions (a) and Plastic Index (b) profiles.

TABLE 1. List of laboratory tests performed on undisturbed samples.

Sample	Depth [m]	Tests
S4 C1	1.30	ED-IL, direct shear (TD), CR, TTC
S4 C2	7.80	CR, TTC
S4 C4	33.30	TD
S15 C1	1.20	ED-IL, TD, CR
S15 C2	4.30	TXC
S16 C1	1.15	ED-IL, TD
S16 C2	4.30	TXC
S16 C3	8.30	CR, TXC

EVALUATION OF LIQUEFACTION SUSCEPTIBILTY

Liquefaction susceptibility has been evaluated by means of procedures prescribed by [1], which is very similar to Italian Code [2], Chinese Code [3] [4] [5] and the criterion suggested by [6].

[1] and [2] consider a simplified and very conservative approach to exclude the occurrence of liquefaction. This approach is based on the expected peak ground acceleration (PGA), on soil composition and on soil state. As for the expected PGA, [1] and [2] assume that liquefaction hazard analysis can be omitted if PGA < 0.15g. In addition the considered sandy soils should met, at least one of the following conditions:

- clay fraction greater than 20% and Ip greater than 10%;
- fine content greater than 35% and $(N_1)_{60}$ greater than 20;
- $(N_1)_{60}$ greater than 25.

The compositional criterion reported in the [1] and [2] leads to the following considerations:

- [6] and [7] stated that cyclic strength of fine-grained soils with Ip > 10% is greater than that of non-plastic fine-grained soils. As a consequence it seems that the plasticity of fine content is more important of its quantity in defining cyclic strength. In fact the greater is the plastic index, the lower is liquefaction susceptibility;
- The limit of 20% for clay fraction seems too much conservative;
- The parameter $(N_1)_{60}$ is strongly affected by grain size distribution [8]. As a consequence the same value for $(N_1)_{60}$ refers to a high relative density for a fine sand, whereas refers to a very low relative density for a medium coarse sand. Also in this case the criterion reported above seems too much conservative.

In the Chinese Code, soils are susceptible of liquefaction if all the condition listed in the following simultaneously occur:

- fine content (d < 0.005 mm) minor than 15%
- liquid limit (LL) minor than 35%
- water content (w_n) greater than 0.9 LL.

An equivalent way to stress the condition about fine content and liquid limit are [3]:

- clay fraction (d < 0.002 mm) minor than 10%
- liquid limit (LL) minor than 32 %.

In this criterion there is no reference to plastic index, although liquid limit is strongly correlated to it.

Lastly, in [6] a criterion based on plastic properties of soils has been suggested, as reported in Fig. 3.

The evaluation of liquefaction susceptibility carried out by means of Eurocode criterion is positive in 85 % of considered cases (the expected PGA has not been considered). A similar result has been obtained with the criterion reported in Fig. 3, in which almost all (92 %) the analyzed specimens are susceptible to liquefaction.

A different result has been obtained by means of Chinese code, in which only 8 of 26 analyzed specimens resulted susceptible to liquefaction. Since the susceptibility of soil to liquefaction does not means that liquefaction will occur, these results impose to evaluate the liquefaction potential.

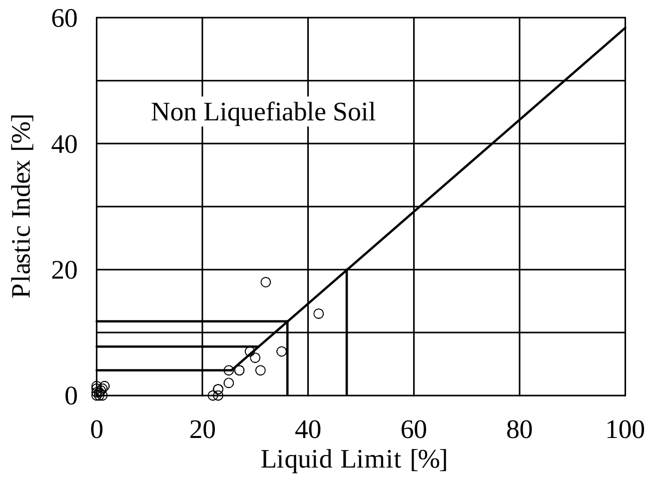

FIGURE 3. Compositional criterion proposed by [6].

ANALYSIS OF LIQUEFACTION POTENTIAL

In a stress approach the first step consists in determination of seismic action. Following the simplified procedure proposed by Italian Code, the seismic action in terms of PGA is expressed as:

$$a_{max} = \frac{\gamma_I S a_g}{g} = 0.375 \quad (1)$$

in which a_{max} is the PGA, γ_I is a factor which takes into account the “importance” of the building (γ_I = 1.2), S is the soil factor (S = 1.25 for type C soil) and a_g/g is the PGA at the rock outcrop prescribed by Code according to macrozonation rules. In addition, determination of number of cycles due to earthquake is essential and subordinate to definition of earthquake Magnitude.

An alternative procedure consists in:

– definition of PGA at rock outcrop by means of a Probabilistic Seismic Hazard Approach (PSHA) for a return period of 975 years [9];

– definition of site effect by means of 1-D seismic response analysis using EERA [10], in which ground profile characterization has been based on in situ and laboratory tests and a group of seven natural free-field accelerograms on rock has been selected after deaggregation of PSHA [11]. The selected accelerograms match, on average, the prescribed spectrum on rock;
– deaggregation of PSHA in order to obtain the most likely earthquake in terms of Magnitude and the most likely distance [11].

Following this procedure a value of a_{max} equal to 0.257 has been estimated, with a reduction greater than 30 % with respect to value calculated by means of Eq. 1. The deaggregation leads to couples Magnitude/Distance [km] equal to 5.4/13 or 5.8/20. Shear stress induced by earthquake can be estimated with the following relationship:

$$CSR = \frac{\tau_{av}}{\sigma_{v0}'} = 0.65 a_{max} \frac{\sigma_{v0}}{\sigma_{v0}'} r_d \qquad (2)$$

in which τ_{av} is the average shear stress and r_d is a stress reduction factor which takes into account the reduction of shear stress with depth.

The normalized cyclic shear stress that causes liquefaction (CRR) has been estimated by means of three different procedures based on in situ tests or laboratory tests.

With reference to a Magnitude equal to 7.5 the CRR can be evaluated using dynamic probing (SPT and DPSH) by [12]:

$$CRR_{M=7.5} = \frac{1}{34-(N_1)_{60cs}} + \frac{(N_1)_{60cs}}{135} + \frac{50}{[10(N_1)_{60cs}+45]^2} - \frac{1}{200} \qquad (3)$$

where $(N_1)_{60cs}$ is the value of number of blows per foot corrected in order to take into account both the confining stress and the fine content. DPSH tests have been processed by means of the same expression after converting N_{20} into N_{SPT} by a factor equal to 1.83, which takes into account the difference in penetration length and efficiency of equipments.

Cone penetration tests can be processed in order to obtain CRR by the expressions [12] [13]:

$$CRR_{M=7.5} = 0.833 \frac{(q_{c1N})_{cs}}{1000} + 0.05 \quad \text{if} \quad (q_{c1N})_{cs} < 50 \qquad (4)$$

$$CRR_{M=7.5} = 93 \left[\frac{(q_{c1N})_{cs}}{1000}\right]^3 + 0.08 \quad \text{if} \quad (q_{c1N})_{cs} < 160 \qquad (5)$$

where $(q_{c1N})_{cs}$ is the cone penetration resistance corrected in order to take into account both the confining stress and the fine content. It is worthwhile to stress that, according to the suggested procedure [12], the soil could be classified as clay (Ic > 2.6) which contrasts the laboratory soil classification.

Lastly, CRR can be obtained by means of laboratory tests. In particular eight undrained triaxial cyclic tests have been carried out on samples retrieved at depth showing the lowest penetration resistance. Figure 4 shows the results of such tests. The number of cycles reported in abscissa has been determined for each test applying the condition $\varepsilon_{a,DA}$ = 5%. Since a Magnitude equal to 5.8 corresponds to a number of equivalent uniform stress cycles equal to 4, the value of CRR deduced by laboratory tests is 0.280.

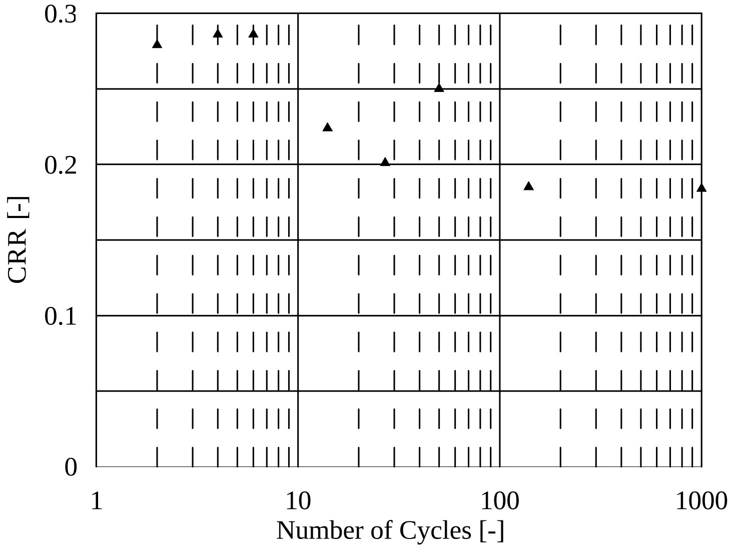

FIGURE 4. CRR as a function of number of cycles in triaxial tests.

Starting from the considerations reported above, an estimation of factor of safety against liquefaction has been computed by means of the following relation

$$FS_L = \frac{CRR_{M=7.5}}{CSR} MSF \tag{6}$$

where MSF = $10^{2.24} \cdot M^{-2.56}$ [12] which takes into account the differences in Magnitude. This factor have not been applied at the case of CRR deduced by laboratory tests.

Figure 5 shows the profiles of FS_L deduced by in situ and laboratory tests. The results reported in the above figures lead to the following considerations:

– the simplified definition of seismic actions, in terms of CSR, lead to conservative estimation of FS_L; this is often due to the hidden introduction of margin of safety both in definition of PGA at outcrop and site effect;
– dynamic penetration tests lead to locate a liquefiable stratum between 3 and 4 m depth;
– cone penetration tests lead to locate liquefiable strata at different depth (4 ÷ 9 m CPTU1, 4.5 ÷ 10 m CPTU2, 8.5 ÷ 14 m CPTU3);

- the use of laboratory tests to determine the liquefaction resistance together with the definition of seismic action through a ground response analysis lead to values of FS_L always greater than 1.
- thickness of liquefiable soil is always lower than thickness of above non-liquefiable soil.

Historically, the site under consideration has experienced a number of earthquakes with Magnitude and distance equal or greater than those obtained from deaggregation of PSHA as shown in Table 2 [14]. Nonetheless, liquefaction phenomena have never been observed in the study area. Therefore it is possible to conclude that a true liquefaction can be excluded. On the other hand it is not possible to exclude the occurrence of localized phenomena (e.g. sand boils, water spouts).

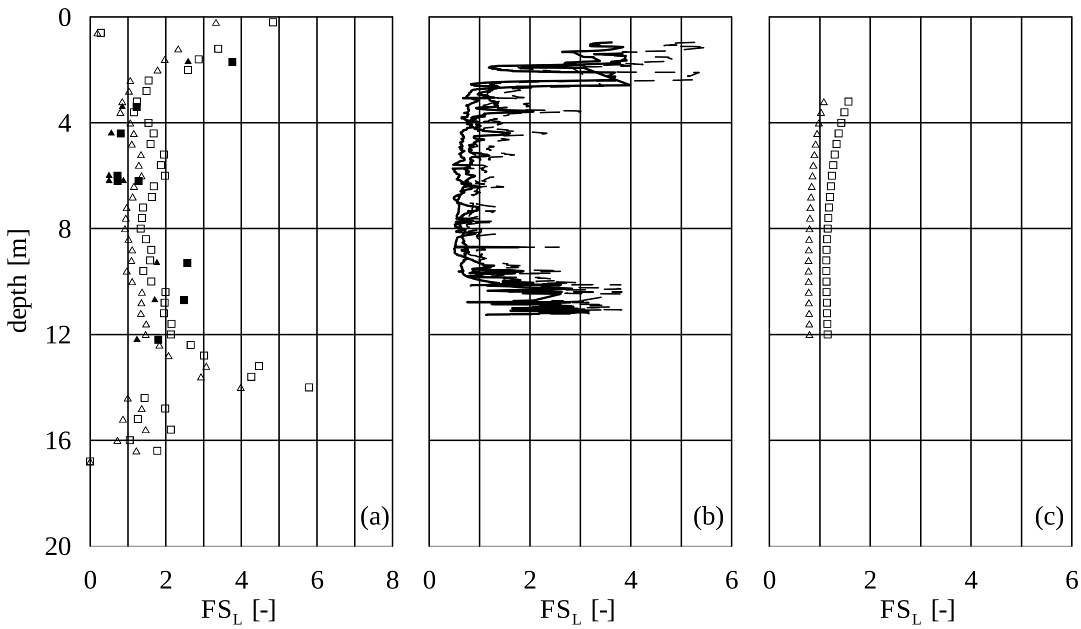

FIGURE 5. Profiles of FS_L deduced by: a) SPT and DPSH tests; b) CPTU tests and c) laboratory tests. Lower series of FS_L are related to simplified evaluation of PGA.

TABLE 2. List of earthquakes with an epicentral distance minor than 20 km.

Date	Location	I_0 (MCS)	Mw	Distance [m]
6 March 1740	Garfagnana	VII	5.18	9975
23 July 1746	Garfagnana	VI	4.83	3758
5 March 1902	Garfagnana	VII	5.17	2432
27 July 1916	Fosciandora	VI	4.83	3131
25 September 1919	Fosciandora	V – VI	4.63	7230
7 September 1920	Garfagnana	IX – X	6.48	19904
15 October 1939	Garfagnana	VI – VII	5.20	18475
12 August 1951	Barga	V – VI	4.74	10245
30 June 1934	Abetone	IV – V	4.38	17641
7 June 1980	Bagni		4.70	9841
23 January 1985	Garfagnana	VI	4.69	10192

CLOSING REMARKS

The paper presents an analysis of liquefaction hazard in a site devoted to construction of a school. The analysis has been carried out by means of different approaches based on in situ and laboratory test, for the aspects concerning the resistance, and on simplified coded procedures and PSHA with 1D-GRA, for the aspects concerning the seismic action. Simpler approaches often introduces hidden margin of safety both in definition of resistances and actions.

A qualitative estimation of possible damages to shallow structures can be carried out on the basis of indication contained in [15]. The presence of a unliquefiable and resistant stratum from ground surface to 3 m depth, in addition to the condition that the thickness of liquefiable soil is always lower than thickness of above non-liquefiable soil, reduces the vulnerability of structures. This last aspect is crucial in managing the problem of liquefaction. In fact, an estimation of earthquake consequences on a construction in that area reported in [16] shows that the possible damages are negligible.

REFERENCES

1. Eurocode 8, *Design Provisions for Earthquake Resistance of Structures - Part 1-1:General Rules for the Representation of Seismic Actions." Part 5: Foundations, Retaining Structures and Geotechnical Aspects*, 1998.
2. OPCM 3274 "Primi elementi in materia di criteri generali per la classificazione sismica del territorio nazionale e di normative tecniche per le costruzioni in zona sismica" in *Gazzetta Ufficiale della Repubblica Italiana,* 8 maggio 2003, n. 108, 2003.
3. W. Wang, "Some findings in soil liquefaction", Research Report of Water Conservancy and Hydroelectric Power Scientific Research Institute, Beijing, 1979.
4. H. B. Seed and I. M. Idriss, *Ground motion and soil liquefaction during earthquakes,* Oakland, EERI, 1982.
5. D. C. Andrews and G. R. Martin, "Criteria for Liquefaction of Silty Soils" in 12th WCEE Proceedings, Auckland, New Zealand, 2000.
6. R. B. Seed, O. Cetin, R. E. S. Moss, A. M. Kammarer, J. Wu, J. M. Pestana, M. F. Riemer, R. B. Sancio, J. D. Bray, R. E. Kayen and A. Faris, "Recent Advances in Soil Liquefaction Engineering: a Unified and Consistent Framework", 26th Annual ASCE Los Angeles Geotechnical Spring Seminar, 2003.
7. K. Ishihara "Liquefaction and flow failure during earthquakes", *Géotechnique* **43**(3), 351-415, 1993.
8. M. Cubrinovski and K. Ishihara, "Empirical Correlation between SPT N-value and Relative Density of Sandy Soils", *Soils and Foundations*, **5**, 61-71, 1999.
9. INGV, http://esse1-gis.mi.ingv.it/ , 2006.
10. J. P. Bardet, K. Ichii and C. H. Lin, "EERA – A Computer Program for Equivalent-Linear Earthquake Site Response Analyses of Layered Soil Deposits", Department of Civil Engineering, University of Southern California, http://geoinfo.usc.edu/gees, (2000).
11. C. Lai, C. Strobbia and Dall'Ara, "Convenzione tra Regione Toscana e Eucentre. Parte 1. Definizione dell'Input Sismico per i Territori della Lunigiana e della Garfagnana", Departement of Structural and Geotechnical Engineering, Technical University of Turin, 2005.
12. T. L. Youd and I. M. Idriss, "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction resistance of Soils", *Journal of Geotechnical and Geoenvironmental Engineering*, **127**(4), 297-313, 2001.
13. P. K. Robertson and C. E. Wride, "Evaluating Cyclic Liquefaction Potential Using the Cone Penetration Test", *Canadian Geotechnical Journal*, **35**, 442 – 459, 1998.
14. G. Fialdini, "Indagini per la valutazione del rischio di liquefazione e dei suoi effetti: un caso reale", Thesis, University of Pisa, 2008.
15. Technical Committee for Earthquake Geotechnical Engineering, TC4, ISSMGE, "Manual for Zonation on Seismic Geotechnical Hazards (Revised Version)", Tokyo: Japanese Geotechnical Society, 1999.
16. F. Benelli, "Valutazione del rischio di liquefazione e dei suoi effetti: un caso reale", Thesis, University of Pisa, 2008.