Lesson Learned from Liquefaction Potential Assessment of Silty Sand Deposits in a Case Study in Italy

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Summary

The paper shows a case study concerning the liquefaction potential assessment of deposits which mainly consist of non-plastic silts and sands. The site under study has been characterized by means of in situ tests (CPTU, SPT and DPSH), boreholes and laboratory tests on undisturbed samples. Liquefaction susceptibility has been evaluated by means of several standard procedures prescribed by codes or available in technical literature. The evaluation of liquefaction potential has been carried out by means of different standard procedures based on in situ and laboratory tests. A qualitative estimation of possible damages to shallow structures has been obtained on the basis of indications contained in TC4-ISSMGE [1993]. In addition, quantitative effects of minor liquefaction phenomena on shallow structures have been studied by means of simplified soil-foundation interaction. The main scope of the paper is to show that application of standard procedures for liquefaction susceptibility or liquefaction potential in areas of medium – low seismicity can lead to overemphasize the liquefaction problem. In addition, any liquefaction study should consider, at least in a simplified way, the effects of possible liquefaction on the shallow structures. In addition useful information about the liquefaction strength of silty sands is provided.

Introduction

Since 2003, the Tuscany Seismic Survey (Italy) 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 (Fig. 1). Investigations for the existing buildings concerned the structure, the structural materials, the geology of the site and the geotechnical characterization of the soil deposits.

Different levels of investigation have been undertaken. After a preliminary stage, a second level of investigation has been undertaken. This usually consisted of (at least) a borehole with SPT and down-hole measurements for each existing public building. The borehole extended down to the seismic bedrock or at least down to 30 m depth. If possible, undisturbed samples have been retrieved for laboratory testing such as Resonant Column tests - RCT, Cyclic Triaxial tests CTX, Torsional Shear tests TST and conventional classification, odometer and direct shear tests. Special tests (RCT, CTX, TST) were aimed at obtaining shear modulus (G) and damping ratio (D).

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measurements and a limited number of SPT measurements. Therefore, additional investigations have been carried out in order to have a better evaluation of liquefaction susceptibility and liquefaction hazards. More specifically, two boreholes with SPT measurements, a continuous dynamic probing (DPSH), three CPTU tests and a number of classification tests and undrained compression loading cyclic triaxial tests (UCTX) in the laboratory were performed.

On the other hand, seismic risk analysis based on a probabilistic approach indicated, for the site under consideration, that the earthquake which mainly contributes to the seismic risk in terms of horizontal peak ground acceleration (HPGA) can be characterized by a Magnitude of 5.8 with a distance of 20 km [LAI et al., 2005]. Historically, the site has experienced a number of earthquakes with Magnitude and distance similar to those above indicated even in recent years (Tab.I), [FIALDINI, 2008]. Nonetheless, relevant 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 in the past or in the future of minor liquefaction phenomena (e.g. sand boils, water spouts). In fact, in the past the area was not urbanised, therefore in remote times minor liquefaction phenomena could have happened without any record of them.

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. The analysis results are discussed with special attention to the effect of non-plastic silt on liquefaction resistance of cohesionless soils. Moreover the prescriptions of Eurocode 8 and Italian code, concerning the liquefaction susceptibility, are critically discussed, in the light of the case study.

Seismicity of the study area

In recent years big efforts have been done in Italy to improve the seismic macrozonation of the territory. Actually the HPGA for different return periods are available at the apexes of a square net of 0.05° of side [INGV, 2005], which really represents an extremely advanced tool in the a-seismic design. Moreover a MS-Excel based software [SpettriNTC ver.1.0.3, 2008] is available to interpolate the HPGA with respect to period and referring to the effective site co-ordinates.
For the case study the values reported in Table II have been obtained.

In Table II, $T_R$ is return period, $F_0$ and $T^*_c$ are parameters used in NTC2008 to define elastic spectrum. The parameter $F_0$ is also used in NTC2008 to compute the stratigraphic amplification factor $S = k_1 - F_0 k_2 \cdot HPGA(\text{rock})$, in which $k_1$ and $k_2$ are constants depending on type of soil.

Due to amplification effects, the HPGA at the top of the soil deposit has been computed as:

$$HPGA(\text{soil}) = S \cdot HPGA(\text{rock})$$

where the parameter $S$ takes into account both topographic and stratigraphic amplification.

For the case under consideration the topographic amplification factor suggested by NTC2008 is equal to 1.0 and $S$ is equal to 1.5 for the first two serviceability limit states (SLO and SLD) and to 1.412 and 1.328 for the two ultimate limit states respectively (SLV and SLC).

Even though there is no specification in the NTC [2008] about the limit state to be considered, it seems reasonable (for liquefaction analysis) to refer to the SLV limit state with a return period of 475 years, which corresponds to a probability of exceeding equal to 10% in 50 years.

The seismicity of the study area has been completely defined by LAI et al. [2005] that performed a de-aggregation of the seismic hazard. The study has been carried out using standard de-aggregation procedures [KRAMER, 1996] obtaining the following couple of Magnitudes and epicentral distances which mainly contribute to the hazard in terms of HPGA with a return period of 475 years:

$M = 5.8$, $d = 20$ km.

After that, they selected a group of seven freefield natural accelerograms compatible with the obtained Magnitude-epicentral distance couples, establishing a window for both $M$ and $d$. They also verified the capability of the selected accelerograms of reproducing on average the prescribed spectrum on rock [NTC 2008; Eurocode 8, 2003]. The characteristics of the selected accelerograms are listed in Table III.

Such accelerograms have been used to compute the seismic response at the top of the soil deposit performing 1D total stress analysis. This approach was used as an alternative to the application of NTC [2008] in assessing the HPGA.

More recently, SPALLAROSSA and BARANI [2007] published the results of de-aggregation of the seismic hazard with respect to the HPGA for a return period of 475 years for the whole Italian territory. Their study indicate for the site under consideration a Magnitude of 5.0 – 5.5 with a distance range between 5 and 25 km.

**Ground investigation**

Figure 2 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 samples have been retrieved from boreholes by means of thin wall inox tube sampler (Shelby sampler). About the possibility of obtaining “undisturbed” samples with any type of sampler, there are several doubts in particular with this kind of soil. On the other hand, there are not reliable procedures to assess sample disturbance. Therefore, the fact that we are dealing with “undisturbed” samples is a consequence of visual inspection of sample after its extraction (e.g. Fig. 7).

Several laboratory tests have been carried out, including RCTs, TSTs, and UCTX. Table IV lists the laboratory tests performed on undisturbed samples. In addition to listed tests, several determination of grain size distribution and plastic index have been carried out on remoulded specimens.

Figure 3 shows the grain-size distribution curves of both undisturbed and remoulded samples. Figure 4 shows the plastic index and granulometric fractions with depth. Simplified geologic - stratigraphic profile, as inferred from boreholes is also shown in Figure 4. The upper layer with a thickness ranging from 9 to 15 m is a recent alluvial deposit (CT-ALL) mainly consisting of silty sands to sandy silts. The underlying layer is a Pliocene fluvial – lacustrine deposit (ARG) mainly consisting of gravels in a clayey-sandy matrix, clayey sands and sandy clays which extend down to over 50 m depth from the ground level. It is supposed that the ARG formation overlies the Macigno Toscano sandstone. Water table oscillates from 1.0 to 3.0 m below the ground level. Figures 3 and 4 clearly indicated that the deposit under consideration mainly consists of saturated sands and non plastic silt.

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This is also confirmed by the results of CPTU summarized in Figure 5a, 5b and 5c, which respectively show the friction ratio ($R_f$, %) the pore water pressure ($U_2$, kPa) and the corrected cone resistance.
Fig. 4 – Main information about subsoil.
Fig. 4 – Principali caratteristiche del sottosuolo.

Table V – Characteristics of the samples.
Tabella V – Caratteristiche dei campioni.

<table>
<thead>
<tr>
<th>Borehole</th>
<th>Sample</th>
<th>$e_0$</th>
<th>$w_p$ [%]</th>
<th>Borehole</th>
<th>Sample</th>
<th>$e_0$</th>
<th>$w_p$ [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>S15</td>
<td>C1</td>
<td>0.653 – 0.711</td>
<td>19.5 – 19.7</td>
<td>S16</td>
<td>C1</td>
<td>0.624 – 0.740</td>
<td>19.5 – 19.7</td>
</tr>
<tr>
<td>S15</td>
<td>C1</td>
<td>0.746</td>
<td>26.3</td>
<td>S16</td>
<td>C2</td>
<td>0.765</td>
<td>22.0</td>
</tr>
<tr>
<td>S15</td>
<td>C1</td>
<td>0.630</td>
<td>19.9</td>
<td>S16</td>
<td>C2</td>
<td>0.802</td>
<td>22.7</td>
</tr>
<tr>
<td>S15</td>
<td>C2</td>
<td>0.722</td>
<td>26.6</td>
<td>S16</td>
<td>C3</td>
<td>0.565</td>
<td>23.1</td>
</tr>
<tr>
<td>S15</td>
<td>C2</td>
<td>0.799</td>
<td>32.2</td>
<td>S16</td>
<td>C3</td>
<td>0.764</td>
<td>22.8</td>
</tr>
<tr>
<td>S15</td>
<td>C2</td>
<td>0.760</td>
<td>26.6</td>
<td>S16</td>
<td>C3</td>
<td>0.877</td>
<td>25.3</td>
</tr>
<tr>
<td>S16</td>
<td>C1</td>
<td>0.632</td>
<td>14.3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fig. 5 – Profiles of Rf (a), U2 (b) and qT (c) for three different CPT.
Fig. 5 – Andamento con la profondità di Rf (a), U2 (b) and qT (c) per le tre verticali.
LESSON LEARNED FROM LIQUEFACTION POTENTIAL ASSESSMENT OF SILTY SAND DEPOSITS IN A CASE STUDY IN ITALY

The identification of stratigraphic profile from CPTU has been discussed, for the site under consideration, in another paper [Lo Presti et al., 2009] and therefore this aspect is not treated in detail in the present paper.

The results (blowcounts) of dynamic penetration tests (SPT and DPSH) are shown in Figure 6 vs. depth. In Figure 6 the normalised blow counts (N1)60 values have been plotted. In the case of SPT, the (N1)60 values have been computed according to YOUD et al. [2001]. The penetration resistances from DPSH (N20) have been normalised in a similar way. However, the measured N20 values were firstly multiplied by 1.83 in order to obtain N_SPT values. The factor 1.83 accounts mainly for different penetration length (20 cm instead of 30 cm) and different efficiency ER. In fact DPSH tests have been performed using a penetrometer TG63/200 [PAGANI, 2009] having ER = 74 %, whereas the most of SPT equipments used in Italy have an efficiency ER = 60%. Smaller differences of the falling height and mass also contribute to define the corrective factor. Continuous dynamic penetration tests were performed using an external casing in order to eliminate or minimise the side friction. Moreover, in this type of soil the ratio between NSPT and N20 has been experimentally checked [Lo Presti and Squeglia, 2008].

Undisturbed samples have been retrieved from boreholes S4, S15 and S16 using a thin wall sampler. RCTs and TSTs were performed to obtain G-γ and D-γ curves for both the upper alluvial deposit (Ct-All) and the Pleistocene deposit. As for the Macigno Toscano sandstone available data in literature have been used [Lo Presti et al., 2006].

It is interesting to remark that during a RCT on specimen from the upper layer a sudden height reduction was observed during the application of largest strain. After dismounting the test a picture of the sample was taken (Fig. 7). The Figure clearly shows the evidence of a thin liquefied layer.

Based on boreholes and in situ seismic tests, two different stratigraphic and shear wave velocity profiles have been defined for 1D linear equivalent seis-
Mic response analyses. Figure 8 shows the considered profiles and soil parameters.

Undrained Cyclic Triaxial Tests have been performed to obtain liquefaction strength. Undrained, stress controlled, cyclic loading triaxial compression tests have been performed on undisturbed samples, isotropically reconsolidated to an effective pressure ranging from $0.5 - 1.0 \sigma_v'$. Axial strains, after reconsolidation ranged from 1 to 3 %.

Specimens were considered liquefied when they experienced a double amplitude axial strain $DA = 5 \%$. When the above condition was met, the ratio between excess pore pressure $u$ and $\sigma_v'$ ($\mu$) was approximately equal to 100%. A loading frequency of 0.02 Hz has been used because of limits of apparatus. Usually greater frequencies are used. However it is possible to say that the use of low frequencies leads to a possible underestimate of the liquefaction strength in the case of soils containing fines. GHAHREMANI et al. [2007] clearly demonstrate this effect.

A typical example of test result (axial strain and pore pressure vs. time) is shown in Figure 9. The liquefaction curve (i.e. normalized stress ratio vs. number of cycles producing a $DA = 5 \%$) is shown in Figure 10. The curve is similar to that obtained for many reconstituted clean sands and corresponds to the upper limit of published data. Red lines shown in figure 10 indicate the range of CSR for a well graded silty sand ($U_c > 10$) similar to the soil of present study [KOKUSHO, 2007].

Table VI – Main data on UCTX.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Depth [m]</th>
<th>$\sigma_c$ [kPa]</th>
<th>CSR</th>
<th>$\Delta u$ [kPa] (DA=5%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>296 - S15 C2</td>
<td>4.30</td>
<td>42</td>
<td>0.286</td>
<td>34</td>
</tr>
<tr>
<td>296 - S15 C2</td>
<td>4.30</td>
<td>41</td>
<td>0.201</td>
<td>(*)</td>
</tr>
<tr>
<td>296 - S15 C2</td>
<td>4.30</td>
<td>40</td>
<td>0.224</td>
<td>39</td>
</tr>
<tr>
<td>298 - S16 C2</td>
<td>4.30</td>
<td>73</td>
<td>0.279</td>
<td>48</td>
</tr>
<tr>
<td>298 - S16 C2</td>
<td>4.30</td>
<td>76</td>
<td>0.184</td>
<td>(*)</td>
</tr>
<tr>
<td>298 - S16 C2</td>
<td>4.30</td>
<td>45</td>
<td>0.25</td>
<td>40</td>
</tr>
<tr>
<td>299 - S16 C3</td>
<td>8.30</td>
<td>108</td>
<td>0.286</td>
<td>71</td>
</tr>
<tr>
<td>299 - S16 C3</td>
<td>8.30</td>
<td>105</td>
<td>0.185</td>
<td>(*)</td>
</tr>
</tbody>
</table>

(*) 5% not reached. No. of cycles extrapolated by data.

Fig. 9 – Typical results of UCTX.

Fig. 9 – Risultato tipico di una prova triassiale ciclica.
Anyway, the tested specimens contain large percentage of non plastic silt. It is therefore worthwhile to discuss the main published evidences concerning the effect of fines on liquefaction resistance. The increase of liquefaction resistance with an increase of plastic fines is generally accepted [ISHIHARA, 1993; WIJEWICKREME and SANIN, 2007; GHAHREMANI et al., 2007].

On the other hand, there are not too many data on the effect of non plastic fine. BOUCKOVALAS et al. [2003] and PAPADOUPOULOS and TIKI [2007] have shown that, at constant void ratio, liquefaction resistance decreases with increasing fines content up to a threshold value. Beyond such value liquefaction resistance increases when the fine content becomes very large. Different thresholds have been experimentally observed depending on the mean effective stress level and type of soil. The mentioned authors indicate thresholds of 35% and 30% at mean effective stress 100 and 300 kPa respectively.

The tested specimens exhibit a very large percentage of silt ranging from 15 to 55%. It is supposed that the soil under consideration exhibits a quite large cyclic undrained resistance because of the high silt percentage. It is also supposed that a great percentage of silt strongly reduces permeability (i.e. possibility of particle transportation) so that the behaviour becomes like that of clay soils.

Evaluation of liquefaction susceptibility

Liquefaction susceptibility depends on the following factors:

- soil type (i.e. composition, geology);
- soil state (i.e. soil density, water table depth)
- design earthquake (i.e. HPGA)

Several criteria to evaluate liquefaction susceptibility are available in technical literature and standards. For the case study liquefaction susceptibility has been evaluated by means of the following criteria:

- prescriptions of Eurocode 8 [2003], which are very similar to Italian Code [OPCM 3274, 2003; NTC, 2005];
- prescriptions of Chinese criteria [WANG, 1979; SEED and IDRIS, 1982; ANDREWS and MARTIN, 2000]
- criterion suggested by SEED et al. [2003];
- prescriptions of NTC [2008].

For the purpose of comparing the various criteria, only those referring to soil type are discussed in the following.

Eurocode 8 [2003] and OPCM 3274, 2003, NTC 2005 prescribe conservative criteria to exclude the occurrence of liquefaction. More specifically as far as the soil type is concerned, liquefaction analysis can be excluded if:

- clay fraction greater than 20% and Ip > 10%;
- fine content greater than 35% and (N1)/60 > 20;

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

- fine content (d < 0.005 mm) < 15%
- liquid limit (LL) < 35%
- water content (wn) > 0.9 LL.

An equivalent way to account for fine content and liquid limit is [WANG, 1979]:

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

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

SEED et al. [2003] suggest a criterion based on plastic properties of soils, as reported in Figure 11 with reference to case studied.

NTC [2008] states that liquefaction potential analysis can be omitted if at least one of the following conditions is satisfied:

1. Expected magnitude $M < 5$;
2. Expected HPGA < 0,1g
3. Groundwater table depth > 15 m
4. Clean sand with $(N_1)/60 > 30$ or $q_{c1N} > 180$ (see Eq. 7)
5. Granulometric curves falling outside fixed zones (see Fig. 3 for well graded sands)

As for NTC [2008], the whole set of criteria has been reported and not only those referring to soil type, because NTC [2008] is the present technical standard in Italy. Therefore it merits a more de-
tailed discussion. The authors would comment in the following way:

- the statement “expected Magnitude” is ambiguous for several reasons (impossible to define a Magnitude without defining a distance, the return period to be considered is not indicated, therefore it is not clear in which way the “expected Magnitude” will be assessed for a given site);
- the statement “expected HPGA” is ambiguous because the return period to be considered is not indicated;
- the criterion based on soil type (granulometric curves) is ambiguous. It seems that a few percentage of clay makes the liquefaction analysis unnecessary.

The authors believe that susceptibility criteria in technical standards should be simple in use and non-ambiguous. More specifically, for the problem under consideration the susceptibility criteria should avoid to overemphasize the liquefaction problem.

For the case study (referring only to different criteria for soil types), the evaluation of liquefaction potential is necessary for:

- 85 % of tested specimens [Eurocode 8 and OPCM 3274];
- 92 % of tested specimens (SEED et al. 2003, see Fig. 11);
- 31 % of tested specimens (Chinese criteria);
- 0-100% of tested specimens (NTC, 2008; see Fig. 3)

Analysis of liquefaction potential

The values of HPGA have been obtained following two different approaches, the prescription of NTC 2008 and a specific study for the site under consideration. Application of NTC 2008 led to HPGA = 0.280g.

On the other hand, the specific study consisted of:

1) definition of M-d couples by de-aggregation of the seismic hazard [LAI et al., 2005];
2) selection of 7 natural free-field accelerograms (consistent with the M-d couple and with the Eurocode 8 spectrum); [LAI et al., 2005];
3) 1D – total stress- linear equivalent seismic response analyses using EERA [BARDET et al., 2000]. The seven accelerograms of step 2 were scaled to the HPGA on rock.

As for the seismic response analyses the following input data have been used:

- seven input accelerograms on rock from step 2 (Tab. III);
- the profiles and the G and D curves [BENELLI, 2008];
- The specific study led, on average, to HPGA = 0.257.

Shear stress induced by earthquake can be estimated with the following relationship:

\[ CSR = \frac{\tau_{av}}{\sigma_{v0}} = 0.65 \frac{a_{max}}{g} \frac{\sigma_{v0}}{\sigma_{vr}} r_d \]  

in which \( \tau_{av} \) 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 [YOUD et al., 2001]

\[ CRR_{M=7.5} = \frac{1}{34-N_s^{0.6}} \frac{(N_1)_{SPT}}{135} + \frac{50}{100} \left[ 0.15 \right] \]  

where \( (N_1)_{SPT} \) is the value of number of blows per foot corrected in order to take into account the energy ratio, 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 [YOUD et al., 2001; ROBERTSON and WRIDE, 1998]:

\[ CRR_{M=7.5} = 0.833 \left( \frac{q_1}{1000} \right) + 0.05 \text{ if } (q_{k1})_{cs} < 50 \]  

\[ CRR_{M=7.5} = 9.9 \left( \frac{q_1}{1000} \right)^{0.8} + 0.08 \text{ if } (q_{k1})_{cs} < 160 \]
where \((q_{clN})_{cs}\) is the cone penetration resistance corrected in order to take into account both the confining stress and the fine content.

To obtain \((q_{clN})_{cs}\) the following procedure was used [Youd et al., 2001]:

\[
(q_{clN}) = \frac{q_c}{100} \left(\frac{100}{\sigma_{vo}}\right)^n
\]  
(7)

\[
(q_{clN})_{cs} = K_c(q_{clN})
\]  
(8)

\[
K_c = 1.0 \quad I_c \leq 1.64
\]  
(9)

\[
K_c = 0.403 \cdot L_4^4 + 5.581 \cdot L_3^3 - 21.63 \cdot L_2^2 + 3375 \cdot L_1^1 - 17.88 \quad I_c > 1.64
\]  
(10)

\[
I_c = \left(3.47 - \log Q^2 + (1.22 + \log F)^2\right)^{0.5}
\]  
(11)

\[
Q = \frac{q_c - \sigma_{vo}}{100} \left(\frac{100}{\sigma_{vo}}\right)^n
\]  
(12)

\[
F = \frac{f}{q_c - \sigma_{vo}} - 100(\%)
\]  
(13)

The above expressions use [kPa] for both stresses and penetration resistance. The following steps are necessary:

- assume \(n = 1.0\) and compute \(I_c\);
- if \(I_c > 2.6\) the soil is classified as clay and the computation can terminate;
- otherwise assume \(n = 0.5\). If assuming \(n = 0.5\), \(I_c < 2.6\), the soil is classified as cohesionless and it is necessary to evaluate the liquefaction potential;
- if assuming \(n = 0.5\) \(I_c > 2.6\), the soil contains nonplastic silt and computation has to be done with \(n = 0.7\).

It is worthwhile to stress that, according to the suggested procedure [Youd et al., 2001], the soil could be classified as clay (\(I_c > 2.6, n = 1.0\)) which contrasts the laboratory soil classification. Following the laboratory classification, data have been processed assuming \(n = 0.7\).

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. 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 (Fig. 10).

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

\[
FS_L = \frac{CRR_{M=7.5}}{MSF}
\]  
(14)

where \(MSF = 10^{2.24 \cdot M^{-2.56}}\) [Youd et al., 2001] which takes into account the differences in Magnitude. This factor has not been applied at the case of CRR deduced by laboratory tests. An alternative way is to determine \(CRR_{M=5.8}\) in correspondence of 15 cycles (Fig. 10), then to apply MSF to take into account the difference in Magnitude. This alternative way leads to a value of \(CRR_{M=5.8}\) greater than that corresponding to 4 cycles, as a consequence it is less conservative.

Figures 12(a, b, c) show 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 P1, 4.5 ÷ 10 m P2, 8.5 ÷ 14 m P4);
- the use of laboratory tests to determine the liquefaction resistance together with the definition of seismic action through a ground response analysis leads to values of \(FS_L\) always greater than 1.25.
- thickness of liquefiable soil is always lower than thickness of above non-liquefiable soil. It is worthwhile to remark that the first three meters of soil deposit exhibit high penetration resistance, very low compressibility and very high stiffness (from odometer and RCT). This is probably due to overconsolidation because of dessication and water table fluctuation.

**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.

The paper also discusses different criteria for liquefaction susceptibility. Such a criteria generally overemphasize the problem of liquefaction or are ambiguous and not clear in their application, with the only exception of the Chinese criteria.

A qualitative estimation of possible damages to shallow structures can be carried out on the basis of indication contained in TC4-ISSMGE [1993]. 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 order to assess the possible damages to the structure, settlement of the first three meters (often in dry conditions) has been evaluated according to TOKIMATSU and SEED [1987] obtaining value of settlement which has been considered compatible with the characteristics of structure.

Furthermore the increase of bending moments and shear stresses in the foundation beam due to minor effects of liquefaction has been evaluated by considering the possible formation of zones, say 0.5 m large, with zero stiffness [BENELLI, 2008]. The analyses have been carried out by means of winkler interaction method in which some springs have been simply removed to simulate the occurrence of minor effects of liquefaction. The above increase of bending moments and shear stresses can be resisted by an appropriate sizing of the foundation structure.

References


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Lessons Learned from Liquefaction Potential Assessment of Silty Sand Deposits in a Case Study in Italy


Un’esperienza nella definizione del potenziale di liquefazione di un deposito di sabbia limosa in Italia

Sommario

L’articolo riporta un caso riguardante la definizione del potenziale di liquefazione di un deposito di terreno che consiste per la maggior parte di limi non plastici e sabbie. Il sito in questione è stato caratterizzato attraverso prove in sito, come CPTU, SPT e DPSH, e prove di laboratorio su campioni indisturbati. In particolare, oltre alle classificazioni, sono state effettuate prove triassiali cicliche non drenate a controllo di carico e prove di colonna risonante. La susceptibilità alla liquefazione è stata determinata attraverso le diverse procedure rinvenibili nelle normative tecniche o nella letteratura tecnica, sia utilizzando i risultati delle prove in sito, sia i risultati delle prove di laboratorio. Sulla base delle indicazioni contenute in TC4-ISSMGE [1993] è stata ottenuta una stima dei possibili danni alla struttura e delle possibili contromisure. Infatti è stato appurato come elemento critico, la presenza di uno strato non liquefacibile nei primi tre metri. Tale presenza riduce la vulnerabilità della struttura e risulta importante per la gestione dei problemi di liquefazione.