Liquefaction Hazards for Seismic Risk Analysis

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ABSTRACT: Seismic risk analysis is a helpful tool that supports decision making for planning and prioritizing seismic risk management strategies. This paper focuses on the importance of an appropriate geotechnical hazard representation within a seismic risk analysis process, focusing on the earthquake-induced geotechnical hazards due to liquefaction. After an overview of alternative methods for the representation of the liquefaction hazards available in the literature, more refined approaches are proposed for implementation in the seismic risk analysis. As a matter of fact, conventional seismic risk and damage scenario analyses use generalized parameters for estimating the consequences of liquefaction that provide a qualitative description of the effects of liquefaction instead of quantitative engineering parameters for the ground deformation. This paper suggests a simple remedy to this practice through the use of some existing methods for estimating the ground distortion caused by liquefaction.

1 INTRODUCTION

Earthquake risk is a public safety issue that requires appropriate risk management measures and means to protect citizens, properties, infrastructure and the built cultural heritage. The aim of a seismic risk and damage scenario analysis is the estimation and quantitative description of the consequences of seismic events upon a given geographical area (a city, region, state or a nation) over a certain period of time. The effects to be predicted include the physical damage to buildings and infrastructure, the number of casualties, potential direct and indirect economic losses including business interruption and downtime, loss of function in lifelines and critical facilities as well as the impact at social, organizational and institutional levels.

The results provided by a seismic risk analysis and damage scenarios, along with the use of a GIS-environment for presentation of the results, could thus be regarded as helpful during all phases of risk management, before and after the critical event. Comprehensive frameworks for damage scenarios and seismic risk analysis, including GIS-based evaluation tools for end-users, have been developed and proposed as part of major international programmes, e.g. HAZUS (FEMA 2003); RADIUS (1999), Risk-UE (2004), DBELA (Crowley et al., 2004), in addition to private implementation carried out by major industries and insurance/risk management companies. Regardless of the common framework, based on the traditionally accepted definition of seismic risk, i.e. convolution of hazard, exposure, vulnerability analyses and cost evaluation, improvements for each of these steps are still possible and necessary in order to increase the reliability of the estimation and hence reduce the uncertainties.

The impact of geotechnical hazards directly caused by ground shaking is well recognised and hence is incorporated in the seismic risk analysis as ground shaking is widely accepted as the primary cause of damage to structures. In spite of the fact that ground failures are the second largest contributor to the damage to buildings and the key factor in the damage to transportation systems and utilities (Bird and Bommer 2004), their inclusion in the seismic risk analysis has been relatively inconsistent and often oversimplified. HAZUS (FEMA 2003) is one of the few published methods implemented in the seismic risk analysis that evaluates the building damage due to ground failure by taking into account the expected mode of ground failure and the type of engineering structure. Only recently Bird et al. (2006) highlighted various shortcomings of the HAZUS method and proposed an alternative approach for the assessment of the damage to reinforce concrete frames due to ground failure.

There are many reasons for the absence of ground failure hazards from the seismic risk analysis, one of the main reasons being the lack of necessary data together with financial and time constraints for collecting such data. The reliability of the simplified models in predicting the potential for ground failure including quantification of ground deformation and damage to buildings has also been an issue.

This paper focuses on the representation of soil liquefaction phenomena and associated ground distortion within the seismic risk analysis. An overview of alternative liquefaction zonation methods, differently defined depending on the available level of knowledge/information is first of all provided. Secondly, with reference to a case study and within the framework of a deterministic seismic risk analysis, an assessment of the liquefaction susceptibility and induced ground deformations are implemented for two different levels of approach, referred in this paper as Level 2 and Level 3 approaches. The Level 2 approach, which corresponds to the one proposed within HAZUS manual (FEMA 2003), is implemented here for a territorial scale area, as it only relies on a detailed geological map. In order to provide a remedy to the over-simplifications made by Level 2 for the treatment of ground failure, this paper proposes the use of some existing methods for estimating the ground distortion caused by liquefaction that are able to provide quantitative engineering evaluation of the ground deformation and that can be reasonably implemented within a seismic risk analysis, by making use of in-situ data. The set of these methods is referred here as the Level 3 approach, proposed in the paper for high liquefaction susceptible areas, where SPT data is available.

Finally, the results from implementation of the two levels of analysis for the study case are compared and commented on. The possibility to integrate the results of different level analyses when dealing with a territorial scale assessment is highlighted in the paper. This will help verify and increase the reliability of the results from a Level 2 analysis, using the outcomes from a Level 3 analysis for the areas where in situ data is not available.

2 LIQUEFACTION HAZARDS FOR SEISMIC RISK ANALYSIS

When using simplified methods for liquefaction assessment it would be necessary to consider three stages in the assessment, as summarised in Figure 1. Presuming that the soils have been identified as susceptible to liquefaction, the factor of safety against liquefaction has to be first evaluated by considering the resistance of the soil to liquefaction and the anticipated ground shaking due to the earthquake excitation considered. If the factor of safety is less than unity and hence liquefaction is expected to occur, then in the second step the consequence of liquefaction in terms of resulting ground deformation has to be estimated. Here, both cyclic lateral ground displacements in the course of the ground shaking and development of liquefaction, and permanent ground distortion such as settlement or lateral spreads need to be predicted. In the third and final step, effects of liquefaction and consequent ground deformation on foundations, waterfront structures and superstructures (buildings, bridges, industrial facilities) have to be evaluated. The assessment of liquefaction potential and associated ground deformation in the first two steps would require availability of results from field tests such as SPT (Standard Penetration Tests), CPT (Static Cone Penetration Tests) and shear-wave velocity measurements (Vs), while for the last step a detailed knowledge of the engineering structure should be gained.

When dealing with seismic risk analysis performed at a territorial scale, the collection of the above data for the whole area under analysis is hardly feasible. For this reason, alternative approaches to the one in Fig. 1 have been proposed in the literature, differently defined depending on the available level of knowledge/information. A reference text for them is the Manual for Zonation on Seismic Geotechnical Hazards TC4 (ISSMGE 199) where various methods available in the literature for the assessment of liquefaction susceptibility are presented and classified into three different levels, referred to as Level 1, Level 2 and Level 3 methods. The three-grade methods have been classified in this paper, depending both on the amount of knowledge/data required for their implementation and on the type of outputs they are able to provide, as summarised in Table 1.

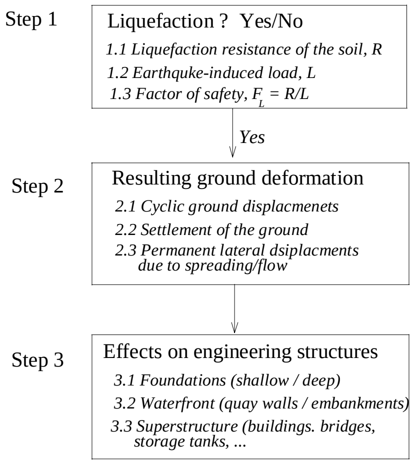


Figure 1. Flow chart for evaluation of liquefaction and its effects on engineering structures in Level 3 geotechnical hazard analysis

A Level 1 approach can be implemented by the compilation and interpretation of existing information available from historic documents (i.e. compiled data on the distribution of damage induced during past destructive earthquakes) or by direct reference to a very simplified representation of the site surface geology. A Level 2 approach would require the availability of detailed geological and geomorphologic maps, while field tests are only necessary when implementing a Level 3 approach. It is worth noticing that all three levels of analysis are primarily focused on the assessment of liquefaction susceptibility, namely Step 1 in the flow-chart shown in Fig. 1. This Step is addressed with increasing reliability: from qualitative description of the likelihood of liquefaction in Level 1 and Level 2 methods, to the assessment of the factor of safety against liquefaction, F_L using Level 3 approaches.

Table 1. TC4 (ISSMGE 1993): three levels of zonation for liquefaction - required data and outputs

	Required Data	**Outputs**
Level 1	Magnitude-Distance or Intensity	Max. extent of a liquefaction susceptible area
	Geological and Geomorphologic maps	Liquefaction susceptibility
	Observed horizontal displacement of lateral spreading after earthquakes	Liquefaction Severity Index
Level 2	Detailed Geological and Geomorphologic maps, past liquefaction sites maps, geotechnical and geological sites investigations	Liquefaction susceptibility
Level 3	Standard Penetration Test	Liquefaction susceptibility
	Thickness liquefiable layer and surface crust	Effects of liquefaction (damage to the ground)

The TC4 manual does not propose any method for the assessment of Step 2 and Step 3 of the flow-chart in Fig. 1 except for some generalized assessment methods of whether liquefaction will or will not cause damage on the ground surface such as that suggested by Ishihara (1985).

On the other hand, within the HAZUS handbook (FEMA 2003), a three step procedure is presented for the assessment of damage to engineering structures due to liquefaction-induced ground failure, namely: Step 1, evaluation of the liquefaction susceptibility and potential; Step 2, assessment of two types of induced permanent ground displacements, namely settlements and lateral spreading; Step 3, expected consequences on engineering structures.

Table 2 summarises the three-step procedure according to HAZUS manual and shows both the method to be implemented and the output gained from each step.

Regarding the level of knowledge/information, a detailed geological and geomorphologic map is the only information required for the implementation of both Step 1 and Step 2, according to HAZUS approach, thus it can be regarded as a Level 2 approach.

Table 2. HAZUS (FEMA 2003): steps for the liquefaction assessment - implemented methods and outputs.

		Output	Implemented Method
Step 1	Liquefaction Susceptibility	Qualitative rating	Youd and Perkins (1978)
	Liquefaction Probability	P[L/PGA]	Liao et al. (1988)
Step 2	Ground settlements	Length [inches]	Assumptions after Tokimatsu and Seed (1978)
	Lateral Spreading	Length [inches]	Seed and Idriss (1982)
Step 3	Damage on structures	P[D_k/PGD]	Assumptions

3 THE STUDY CASE AREA: WESTERN LIGURIA REGION AND ARGENTINA VALLEY

The reference study case in this paper is the Western part of the Liguria Region, Italy (Fig. 2a). The area has been the object of a GNDT-INGT funded collaborative Research Project between different Italian Universities (http://adic.diseg.unige.it/gndt-liguria/index.html). The interest in the western Liguria Region was raised by the seismic sequence that struck the area between 1818 and 1887, and by the will to preserve the high cultural and monumental value historical centres in that area. Induced geotechnical hazards were not investigated during the project and were not included within any of the simulated scenarios. It is worth noting that an exhaustive analysis of these effects is completely outside the scope of this paper, the purpose of which is to only explore the implementation of a number of existing approaches for the assessment of liquefaction potential and the consequences.

In this paper, a territorial scale analysis has been implemented for the Argentina Valley (identified with a red circle in Fig. 2a), an area of about 30 km^2, along the Argentina River, characterized by geological and topographic heterogeneity, where recent alluvium is suspected to be characterised by a high liquefaction susceptibility. An assessment of the liquefaction susceptibility of the soil and of the consequent ground-displacement is presented using existing information, in particular: 1) a geological map available for the whole area providing the age and material characteristics of the geologic units, 2) geotechnical boreholes for the recent alluvium identified in the study area.

Two historical events have been assumed as the scenario earthquakes: 1) the maximum historical event in the region corresponding to the Western Liguria Feb 23, 1887 earthquake (M=6.3, I_0 = X, Long=8°,1430, Lat = 43°,7480); 2) the historical event with the epicentre closest to the study area, similar to the May 26, 1831 earthquake (M=5.5, I_0 = VIII, Long=7°,8594, Lat = 43°,8627). A deterministic PGA hazard scenario for each one of the reference earthquakes has been drawn in accordance with the predictive equation from Ambraseys et al. (1996). It is worth noting that all the procedures described in the following can be as well implemented within a probabilistic framework, making reference to a disaggregated representation of a probabilistic seismic hazard analysis (PSHA).

The water-table has been detected for the Argentina Valley after Peloso (1999) via geophysical-electrical soundings, ranging between d_w=1 m to 9 m from the lower to the upper part of the valley.

4 IDENTIFICATION OF THE MAXIMUM EXTENT OF LIQUEFACTION SUSCEPTIBLE AREA

A preliminary analysis using methods based on observed damage data and on information collected from geological maps, has been considered in this paper as a reliable starting point in order to identify the area for which more detailed analyses of liquefaction susceptibility and induced ground deformation were needed.

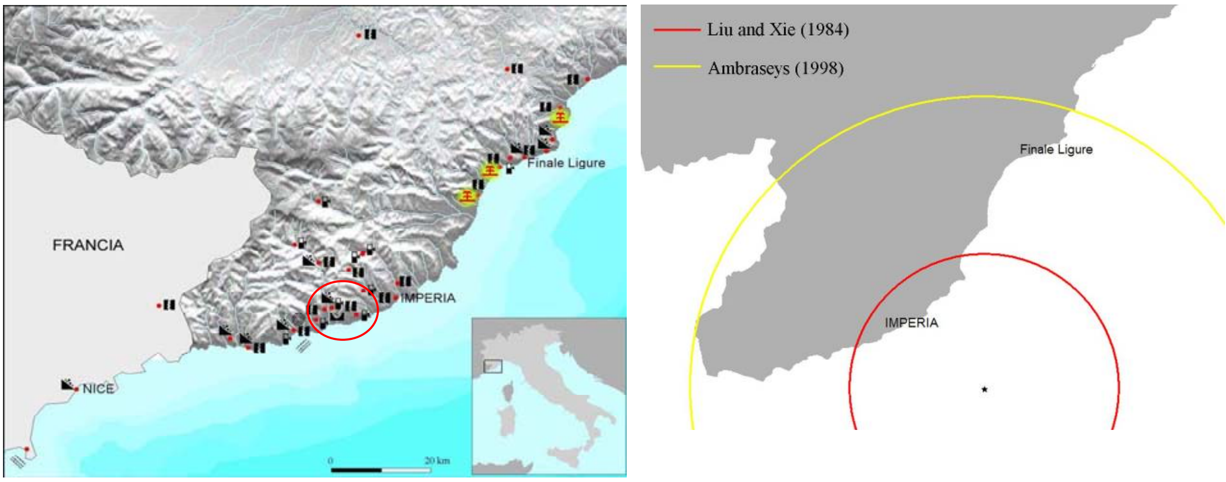


Figure 2. Earthquake-induced geotechnical hazard in Western Liguria: a) effects observed after 1887 event (courtesy Guidoboni – SGA Bologna); b) predicted maximum liquefaction susceptible areas

In order to identify the maximum extent of the liquefaction susceptible area within a region and for a given scenario earthquake, reference has been made to magnitude-maximum distance criteria reviewed and proposed by TC4-ISSMGE manual, among the Level 1 approaches. The proposed criteria include significant scatter since they have been derived by processing data for earthquakes from different parts of the world. The lower and the upper bound magnitude-maximum distance criteria have been implemented for the case study, respectively corresponding to those proposed by Liu and Xie (1984), and Ambraseys (1998). The maximum radius of the liquefaction susceptible area for the 1887 earthquake has resulted respectively in R=24 km and R=52 km (Fig. 2b). The identified areas have been compared with the extension of the secondary geological hazard effects for Western Liguria (Fig 2a) according to historical document investigation performed by Guidoboni (personal communication). The comparison between Fig. 2a and 2b, shows how the Liu and Xie (1984) criteria (red line) effectively identifies the most affected area of recorded induced geotechnical-hazards, while the Ambraseys (1998) criteria (yellow line) comprises all the observed effects.

The Level 2 approach described in the following section has been implemented within the liquefaction susceptible area identified by Liu and Xie (1984) magnitude-distance criterion (red line).

5 IMPLEMENTATION OF HAZUS (LEVEL 2) APPROACH FOR THE ASSESSMENT OF LIQUEFACTION HAZARDS

5.1 *Liquefaction susceptibility and potential: Step 1 for Level 2 approach*

The method proposed by Youd and Perkins (1978) is the one adopted in HAZUS for a qualitative mapping of liquefaction susceptibility. This is the only method identified by the TC4-ISSMGE (1993) manual for the assessment of liquefaction susceptibility, within the Level 2 methods, the other one being from Wakamatsu (1992) only feasible for a $I_{M.M.S}$=VIII (M.M.S. Modified Mercalli Scale) earthquake event. The reliability of this approach has been validated following 1989 Loma Prieta earthquake, when nearly all of the sites where liquefaction occurred lie within zones previously mapped as having a high or very high liquefaction hazard according to Youd and Perkins (1978). According to the Youd and Perkins (1978) approach, the susceptibility to ground failure of sedimentary deposits is evaluated based on geological descriptions of the near-surface soils from a geological map. For each mapped geological unit, the identification of the age, the depositional environment and the material type required by the method involves a certain degree of expert-judgement. Based on the aforementioned three characteristics, a relative liquefaction susceptibility rating is distinguished on a 5-grade scale as: very low, low, moderate, high and very high susceptibility.

Fig. 3a shows the implementation of the Youd and Perkins (1978) approach to the study area.

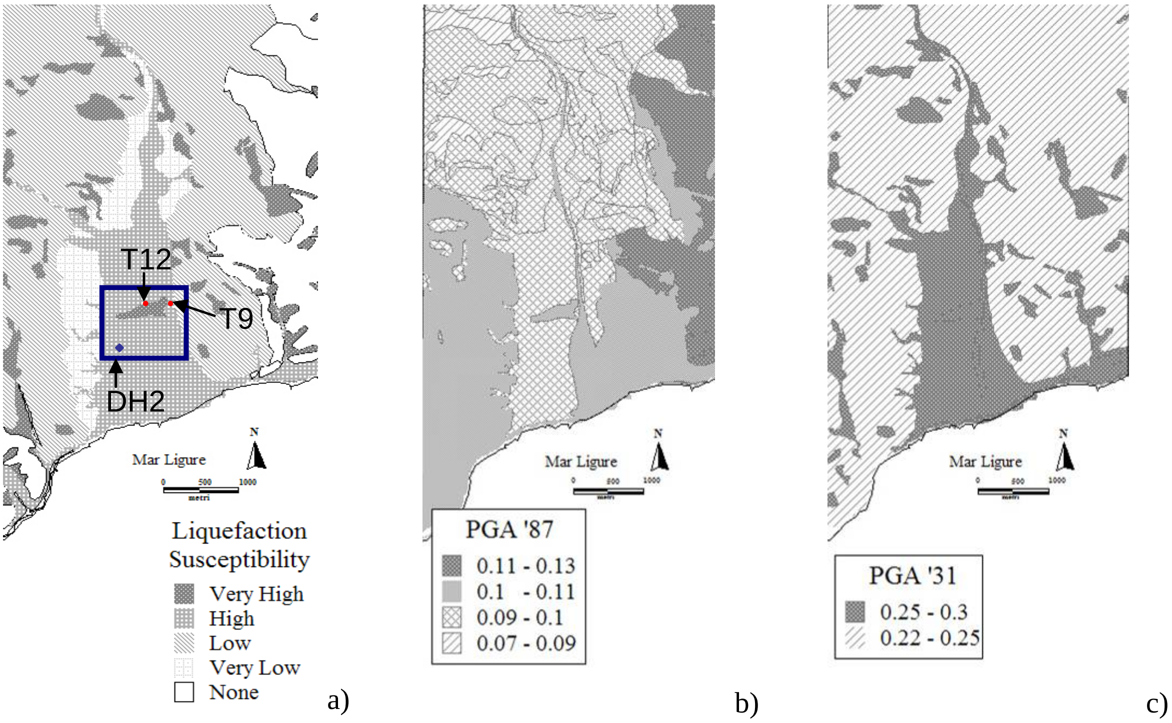


Figure 3. Argentina valley: a) Liquefaction susceptibility according to Youd and Perkins (1978), b) 1887 - PGA [g] hazard scenario, c) 1831- PGA [g] hazard scenario

Once the different susceptibility categories have been identified for the analysed area, the HAZUS approach allows the assessment of the probability of liquefaction as a function of the expected peak horizontal ground acceleration, PGA via simplified linear-relationships, proposed after the empirical approach by Liao et al. (1988).

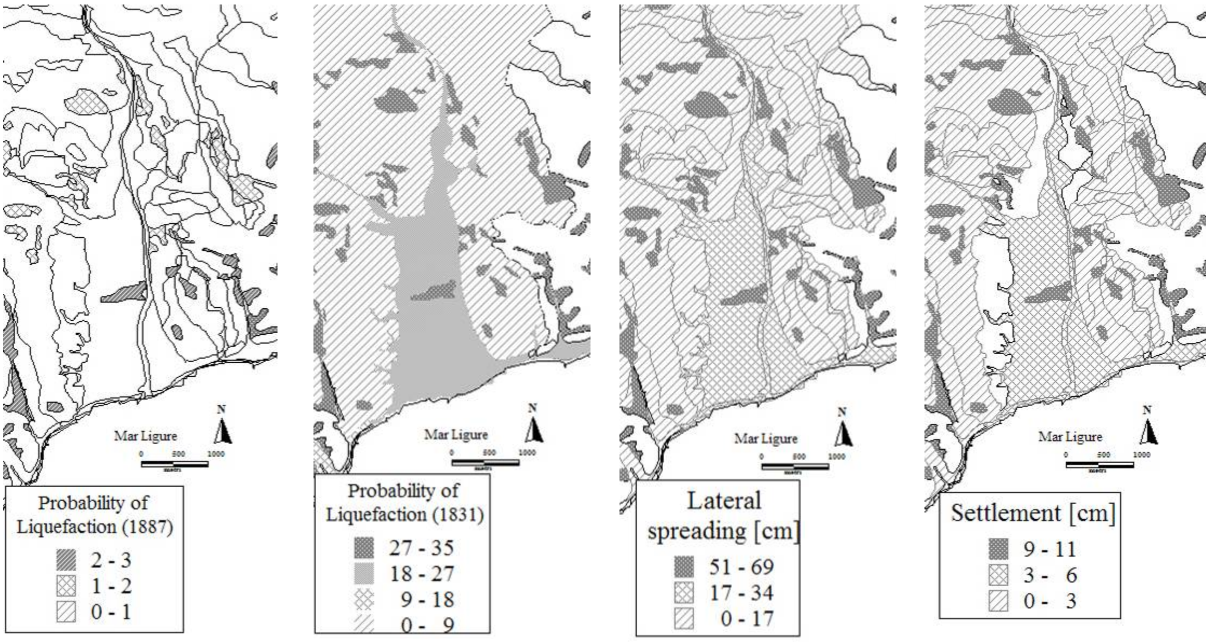
Figures 4a and 4b show the assessment of the liquefaction probability in the Argentina valley implementing the aforementioned approach, making reference both to: 1) the susceptibility categories identified as in the map in Fig. 2a; 2) the deterministic hazard assessment respectively for the 1887 (Fig. 3b) and the 1831 (Fig. 3c) scenario earthquakes. According to the implemented procedure, correction factors to account for the magnitude K_M and for the ground water level K_w have to be applied (resulting in $K_{M=6.7}=1.24$, $K_{M=5.5}=1.43$ and $K_w=1.12$, when assuming on average, a $d_w=3$m water-table).

Concentrating the attention on the two geological units within the blue square on Figure 3a where a high (H) and very-high (VH) liquefaction susceptibility has been identified the procedure assesses a nearly null liquefaction probability for the 1887 scenario ($P[L/PGA_{1887}] \cong 0\%$ for H, $\cong 2\%$ for VH), and a probability of $P[L/PGA_{1831}] \cong 20\%$ and $\cong 30\%$ for the 1831 earthquake respectively for H and VH areas.

5.2 *Permanent ground displacements due to lateral spreading: Step 2 for Level 2 approach*

The expected maximum permanent ground displacements due to lateral spreading are determined, according to the HAZUS approach, as a function of the expected level of ground shaking, PGA normalized to the threshold ground acceleration necessary to induce liquefaction for each susceptibility category, PGA(t). The implemented function is obtained by combining the Liquefaction Severity Index (LSI) relationships derived by Youd and Perkins (1987) with the attenuation relationships of Sadigh et al. (1986). Default values are provided by the HAZUS manual for the threshold ground acceleration, PGA(t). A displacement correction factor K_Δ accounting for the earthquake magnitude has to be applied, resulting in $K_{\Delta\,(M=6.7)} = 0.48$ and $K_{\Delta\,(M=5.5)} = 0.33$ for the 1887 and 1831 earthquakes respectively.

Figure 4c shows the lateral spreading expected according to the Level 2 procedure for the 1831 scenario earthquake. For the H and VH liquefaction susceptible areas, a lateral spreading of LS=22cm and LS=60cm has been evaluated.



a) b) c) d)

Figure 4. HAZUS approach: a) Liquefaction probability for 1887 scenario; b) Liquefaction probability for 1831 scenario; c) lateral spreading for 1831 scenario; d) ground settlement for 1831 scenario

5.3 *Permanent ground displacements due to ground settlement: Step 2 for Level 2 approach*

According to the HAZUS approach, the ground settlement induced by liquefaction, can be evaluated as the product of the probability of liquefaction for a given ground motion level (Fig. 4b for the 1831 earthquake) and a characteristic settlement amplitude (default values are provided by the HAZUS manual) appropriate to the susceptibility category (Fig. 3a).

Figure 4d shows the assessed expected settlement for the 1831 scenario earthquake. For the H and VH liquefaction susceptibility areas identified, a settlement of S=3cm and S=11cm is expected according to the Level 2 procedure.

6 IMPLEMENTATION OF A LEVEL 3 APPROACH FOR THE ASSESSMENT OF LIQUEFACTION HAZARDS

A Level 3 analysis are proposed here and the results are presented for the recent alluvium Holocene sediments area in the blue square in Fig.3a, identified as a high, H and very high, VH susceptibility areas according to Step 1 of the Level 2 approach. For these areas, a number of geotechnical boreholes and related Standard Penetration Test (SPT) results have been collected from previous studies.

6.1 *Assessment of the factor of safety against liquefaction, F_L: Step 1 for Level 3 approach*

The summary report from 1996 and 1998 NCEER/NSF Workshop on the Evaluation of Liquefaction Resistance of Soils (Youd and Idriss, 2001) has been used as the basis for the assessment of the factor of safety against liquefaction according to Level 3 approach. According to the method, the onset of liquefaction occurs when the Cyclic Stress Ratio, CSR exceeds the Cyclic Resistance Ratio, CRR.

The Cyclic Stress Ratio, $CSR_{7.5}$ has been evaluated as a function of the maximum horizontal acceleration, PGA assessed for the deterministic hazard scenario (Section 3), the total σ_{vo} and effective σ'_{vo} vertical overburden stresses, and a stress reduction coefficient r_d as a function of the depth below ground surface, z. The $CSR_{7.5}$ has been modified to account for a different earthquake magnitude by dividing it with a magnitude scaling factor, MSF, evaluated according to Youd and Idriss (2001) and resulting in $MSF_{6.7}$=1.23 and in $MSF_{5.5}$=1.69 for the two scenario earthquakes.

The Cyclic Resistance Ratio for an earthquake magnitude M=7.5, CRR has been evaluated as a function of the corrected SPT N-value $(N_1)_{60}$, where correction factors have been accounted for the energy ratio and for the overburden pressure. The correction of the SPT N-value for variations in fines content has not been applied as the information on the fines content was not available. Table 3 and Table 4 show the results in terms of factor of safety against liquefaction, F_L evaluated for the 1887 and for the 1831 scenario earthquakes for the boreholes T9 and T12, respectively laying in H and VH liquefaction susceptibility areas according to the Level 2 procedure. Table 5 shows the result in terms of F_L evaluated as a function of the shear wave velocity, $V_{s1,}$ according to Andrus and Stokoe (1997) when assuming the values of a=0.022, b=2.8, V_{s1}^*=200m/s for the parameters of the proposed formula.

Nonetheless, a direct correspondence can not be established between the Level 2 liquefaction probability P[L/PGA] and the Level 3 factor, F_L the results from the two procedures seem to be consistent.

Table 3. Borehole T9

from [m]	to [m]	Soil Type	γ [kN/m^3]	$(N_1)_{60}$	F_L('87)	F_L('31)
0	14.1	gravelly soils	20	>30	-	-
14.1	18.5	loose sandy soils	18	15.4	2.8	1.5
18.5	20.5	loose sandy soils	18	7.1	1.6	**0.9**
20.5	21.3	silt	17	2.3	1.0	**0.5**
21.3	22.5	sandy soils	18	>30	-	-
22.5	24	gravelly soils	20	0.8	1.0	**0.5**

Table 4. Borehole T12

from [m]	to [m]	Soil Type	γ [kN/m^3]	$(N_1)_{60}$	F_L('87)	F_L('31)
0	3	gravelly soils	20	>30	-	-
3	4.5	gravelly soils	20	10.4	2.8	1.5
4.5	6	silt	17	6.6	1.8	1.0
6	7.5	silt	17	5.6	1.4	0.8
7.5	9	sandy soils	19	4.6	1.2	**0.6**
9	10.5	sandy soils	19	2.2	**0.9**	**0.5**
10.5	12	sandy soils	19	5.0	1.2	**0.6**
12	13.5	sandy soils	19	13.8	2.4	1.3
13.5	15	sandy soils	19	12.7	2.2	1.2

Table 5. Down Hole DH2

from [m]	to [m]	Soil Type	γ [kN/m^3]	V_s'	F_L('87)	F_L('31)
0	2.5	surface sandy soils	17	362	3.7	1.5
2.5	5.5	gravelly soils	20	319	1.9	**0.8**
5.5	12	sandy soils	18	280	1.1	**0.4**

6.2 *Cyclic ground displacements: Step 2.1 for Level 3 approach*

Based on observations from strong earthquakes including analysis of strong motion records and detailed field surveys of piles, Tokimatsu and Asaka (1998) proposed a simple procedure for calculating cyclic ground displacements of liquefied soils. The procedure is based on an empirical chart correlating the cyclic shear strain with the SPT blow count and cyclic stress ratio induced by the earthquake, as shown in Figure 5a. The chart is essentially equivalent to the conventional SPT-based charts for evaluation of liquefaction potential. For each liquefiable layer the cyclic shear strain is first evaluated from the chart using the respective SPT blow count and then the cyclic ground displacement profile is calculated by integrating the shear strains throughout the depth of the soil profile. Using this method, a peak cyclic ground displacement of 18cm was evaluated for the T12 borehole.

6.3 *Settlement of the ground: Step 2.2 for Level 3 approach*

Similarly, based on laboratory tests and D_r-N correlation, Ishihara and Yoshimine (1992) developed a chart correlating the post-liquefaction volumetric strain with the SPT blow count and factor of safety against liquefaction, as shown in Figure 5b. For each liquefiable layer, the factor of safety F_L is first evaluated by conventional methods, and then the volumetric strain is evaluated from the chart using the representative SPT blow count for the layer. The settlement is eventually calculated by integrating the volumetric strains throughout the depth of the soil profile. Based on the Ishihara and Yoshimine (1992) procedure, the expected settlement is S_{T9}=13cm and S_{T12}=22cm respectively for the two analysed boreholes T9 and T12. The Level 2 procedure seems to underestimate the expected settlement as for the areas corresponding to the T9 and T12 boreholes settlement of S=3cm and S=11cm have been respectively evaluated (Section 5.3).

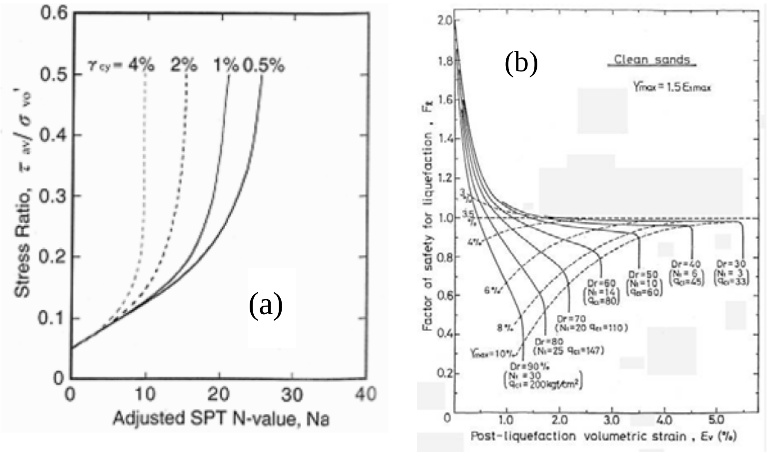


Figure 5: SPT based charts for estimating ground deformation due to liquefaction: a) cyclic shear strains; b) settlement due to volumetric strains

6.4 *Lateral spreading: Step 2.3 for Level 3 approach*

Several empirical methods for estimation of lateral spreading displacements are now available and are applied in geotechnical practice. In most cases it would be very difficult to make a reliable prediction for the spreading displacements, and this uncertainty has to be properly addressed in the interpretation of the estimated lateral spreading displacements. The assessment of the expected lateral spreading is not addressed in this paper.

7 CONCLUSIONS

When dealing with seismic risk analysis performed at a territorial scale, the collection of the data required in order to perform an in-depth geotechnical zonation for liquefaction is hardly feasible. Nonetheless the representation of earthquake-induced geotechnical hazards due to liquefaction cannot be disregarded when performing a seismic risk analysis. As a matter of fact, earthquake-induced geotechnical hazards have shown to be the second largest contributor of damage to buildings and a key factor in the damage to transportation systems after the recent earthquake events.

This paper has shown how, under the guide of a geotechnical expert, the outcomes from different levels of analysis can be integrated in order to reach this aim. A preliminary analysis, namely a Level 1 analysis, using methods based on observed damage data can be implemented in order to identify the extent of the area where implementation of a Level 2 approach is appropriate. A Level 3 approach can be implemented for restricted areas, in order to verify the reliability and in order to tune the results from the preliminary analysis. The outcomes from a Level 2 analysis can be used as a reference for collecting in situ data, from public and private sources, or for planning further investigations, only for the more liquefaction susceptible areas, thus limiting the time and the cost of the study.

The consequences of operating via the integration of different level approaches, on the expected damage predicted for engineering structures and the treatment of the uncertainties is on-going and will be the object of future further studies.

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