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Beyond EC8: the new Italian seismic code

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The 2002 Molise earthquake, which was defined by seismologists as a normal event in the geodynamics of the Italian peninsula but had an international resonance due to the collapse of a primary school, triggered a series of research initiatives in earthquake engineering and significant modifications to building codes in Italy. The modifications were completed at the beginning of 2008 with the release of a new comprehensive building code for Italy. This document was mainly inspired by Eurocode, but it contains some changes and improvements.

In this paper, comments are made on three specific parts of the new code: definition of seismic action, analysis of liquefaction and analysis of slope stability. For the first part, seismic action is defined based on a recent careful study of the seismic hazard in Italy. For liquefaction analysis, some developments are given, keeping the same structure used in Eurocode. Finally, for slope stability, improvements are introduced to avoid overestimation of pseudostatic forces in conventional analyses.

Keywords: Geotechnical characterization, liquefaction, local seismic response, slope stability, seismic code

1. Introduction

On October 31st, 2002, a moderate earthquake (moment magnitude M_w = 5.78) hit the town of San Giuliano di Puglia, Molise Region, Italy, approximately 200 km E of Rome. The earthquake caused the collapse of a primary school and the deaths of 27 students and a teacher.

As a direct consequence of this event, which had a great impact on the Italian community, a new seismic code (OPCM 3274, 2003), inspired mainly by Eurocode 8, was introduced in Italy just a few months after the earthquake. In addition, a new seismic classification of the Italian territory was adopted (Gruppo di Lavoro, 2004) based on a rationale analysis of the seismic hazard. This was necessary because at the time of the 2002 Molise earthquake, San Giuliano di Puglia was not considered a seismic area for code purposes. As a matter of fact, an area was considered seismic in Italy if it had been hit by a deadly earthquake within the last 100 years, since the 1908 Messina and Reggio earthquake.

These actions caused great interest in and some concern about earthquake engineering by the national technical community. At that time, it was decided that the level of knowledge of and the quality of research in this subject in Italy be improved, and a formation center and a university network of seismic engineering laboratories were created (Reluis Consortium, www.reluis.it).

Concerning geotechnical earthquake engineering, two main points of discussion arose soon after the issuance of the new seismic code. The first point was that the document contains several innovations to the previous code and thus needs to be read with care. This was particularly true considering that the average level of knowledge of this subject in Italy was relatively limited: before 2003, classes in Soil Dynamics or in Geotechnical Earthquake Engineering were held only in a few universities. The second point was that new designs of geotechnical structures might be too conservative compared with previous designs due to the proposed computational methods and the design accelerations (e.g., Simonelli, 2003) included in the new code.

Aware of these two points, the Italian Geotechnical Society established a working group to write guidelines for the "Geotechnical Aspect of the Design in Seismic Areas" (AGI, 2005) in the Fall of 2003. The guidelines were intended to fill the gap in knowledge of Geotechnical Earthquake Engineering in Italy and constituted a basis for further improvements in the geotechnical seismic code.

The Italian guidelines follow the so called "performance-based-design approach", requiring analysis of geotechnical systems under two different seismic events with different returning periods. That is, for frequent earthquakes, it is required that a geotechnical system exhibit good performance, satisfying the typical requirements of a Damage Limit State. For rare events, it is required that a geotechnical system exhibit different performances (from the Damage Limit State to the Ultimate Limit State) according to the type and purpose of the construction. The performance-based-design method may be developed using three levels of analysis, varying from traditional empirical and pseudostatic approaches to pseudodynamic and fully dynamic studies according to the importance and requirements of the construction.

Next, the Italian Geotechnical Society established another working group specifically devoted to the review of the geotechnical seismic code after an agreement was made with the Department of Civil Protection, which produced a document that was released in the Spring of 2007 and incorporated into the new technical code for construction (NTC, 2008). The new code was officially released in February 2008 and took effect in July 2009 due to the pressure of public opinion after the April 2009 L'Aquila earthquake.

The new Italian building code NTC is a comprehensive document covering several topics, including the design of new civil and industrial constructions, bridges and geotechnical structures and the modification of existing structures.

Discussion of all of the above topics is beyond the scope of this paper. Instead, a few selected topics related to earthquake geotechnical engineering (seismic motion, liquefaction and slope stability) are discussed. It is assumed that readers are familiar with Eurocode 8 parts 1 and 5 (EN 1998-1, 2003; EN 1998-5, 2003) and that the differences between the Eurocodes and the NTC can be easily recognized.

2. Seismic action

According to Eurocode 8 part 1 (EN 1998-1, 2003), each national territory is subdivided into seismic zones, depending on the local hazard. In each seismic zone, the hazard is assumed to be constant and is described in terms of a single parameter, i.e., the value of the reference peak ground acceleration on outcropping bedrock a_{gR}.

The reference peak ground acceleration, chosen by the National Authorities for each seismic zone, corresponds to the reference return period $T_{N,CR}$ of the seismic action for the no-collapse requirement. An importance factor γ_I, which is a coefficient related to the consequences of a structural failure, of 1.0 is assigned to the reference return period. For return periods other than the reference, the design ground acceleration on outcropping bedrock a_g is equal to a_{gR} times γ_I ($a_g = \gamma_I \cdot a_{gR}$).

In Eurocode 8, it is prescribed that structures in seismic regions comply with the following requirements:

1. the no-collapse requirement; and
2. the damage limitation requirement.

The NTC presents several new terms to describe seismic hazards and seismic actions on structures.

First, it introduces a reference period V_R for seismic actions, which is given by the product of the nominal life of a construction V_N and its coefficient of use C_U. V_N is the number of years during which a structure, if subjected to regular maintenance, should be used for the purpose for which it was designed. It is suggested that V_N = 10 years for temporary structures, $V_N \geq 50$ years for ordinary buildings and structures, and $V_N \geq 100$ years for large or strategic constructions.

The coefficient of use is directly linked to the class of use of the construction, from Class I (rare presence of people, construction for agriculture, $C_U = 0.7$) to Class II (normal presence of people, $C_U = 1.0$) up to Class IV (important public and strategic buildings also used for civil protection, $C_U = 2.0$).

Two damage limit states (SLO, SLD) and two ultimate limit states (SLU, SLC) are established in the code:

1. Operability limit state (SLO): after an earthquake, the entire structure, including its structural elements, nonstructural elements, and apparatuses relevant to its functionality, is neither damaged nor subject to significant interruptions in functioning.

2. Limit state of prompt use or Damage (SLD): after an earthquake, the entire structure, including structural elements, nonstructural elements, and apparatuses relevant to its functionality, has damage that does not compromise its stiffness and resistance against vertical and horizontal actions. The structure is ready to be used but the apparatuses might be subject to malfunctioning.

3. Limit state for the safeguard of human life or Ultimate state (SLU): after an earthquake, the construction is affected by failures and collapses of nonstructural components and apparatuses and significant damage to structural components that result in a significant reduction of stiffness and resistance against horizontal actions. The construction retains significant stiffness and resistance against vertical actions and retains, as a whole, a significant safety margin against collapse from horizontal seismic actions.

4. Limit state for collapse prevention (SLC): after an earthquake, the construction has suffered serious failures and collapses of nonstructural components and apparatuses and very serious damage to structural components that result in a substantial loss of stiffness and a contained loss of resistance against horizontal actions. The construction retains a significant stiffness and resistance against vertical actions but has a small safety margin against collapse from horizontal actions.

According to the code, the probability of exceedance of the seismic action during the reference period varies with the limit state, as shown in Table 1.

It follows that the returning period of the design earthquake can be evaluated assuming a statistical distribution of seismic events. If the Poisson model is used to predict the temporal uncertainty of an earthquake, the returning period T_r is given by:

$$T_r = \frac{1}{\lambda_M} = -\frac{t_S}{\ln(1-P)} \tag{1}$$

In Equation (1), λ_M is the average rate of occurrence of the event, t_S is the time period of interest (the reference period V_R in this case) and P is the prob-

Table 1. Variation of the probability of exceedance of the seismic motion for different limit states.

Limit state		Probability P of exceedance in the reference period V_R
Serviceability limit state	SLO	81%
	SLD	63%
Ultimate limit state	SLU	10%
	SLC	5%

ability of a number of occurrences of a particular event during a given time interval. Therefore, the returning period for the Ultimate Limit State for an ordinary building is given by: $T_r = \frac{50}{\ln(1-0.1)}$ = 475 years, with a nominal life of 50 years, a coefficient of use of 1.0 and a probability P of 10%.

This way of defining the earthquake returning period is associated with a system that has recently become available in Italy, which allows visualization and querying of probabilistic seismic hazard maps of the national territory using several shaking parameters on a regular grid with a 0.05° spacing (Meletti and Montaldo, 2007). This system was directly incorporated into the New Building Code. Quoting the website http://esse1-gis.mi.ingv.it/help_s1_en.html, the maps display two shaking parameters, Peak Ground Acceleration (PGA) and spectral acceleration (Sa) on stiff horizontal outcropping bedrock. Maps of PGA have been evaluated for different probabilities of exceedance within 50 years (9 probabilities, from 2% to 81%). For each evaluation, the distribution of the 50^{th} percentile (the median map, which is the reference map for every probability of exceedance) and the distributions of the 16^{th} and 84^{th} percentiles (which give the variability of each estimate) are available.

Maps of Sa have been evaluated for the same probabilities of exceedance within 50 years and for different periods (10 periods, from 0.1 to 2 seconds). For each evaluation, the distribution of the 50^{th} percentile and the distributions of the 16^{th} and 84^{th} percentiles are available.

In summary, there is now a tool in Italy, incorporated into the NTC that allows determination of the PGA and the design spectrum at each location in the territory for earthquakes with different returning periods. An example is in given in Figure 1, for the city of Termoli (CB).

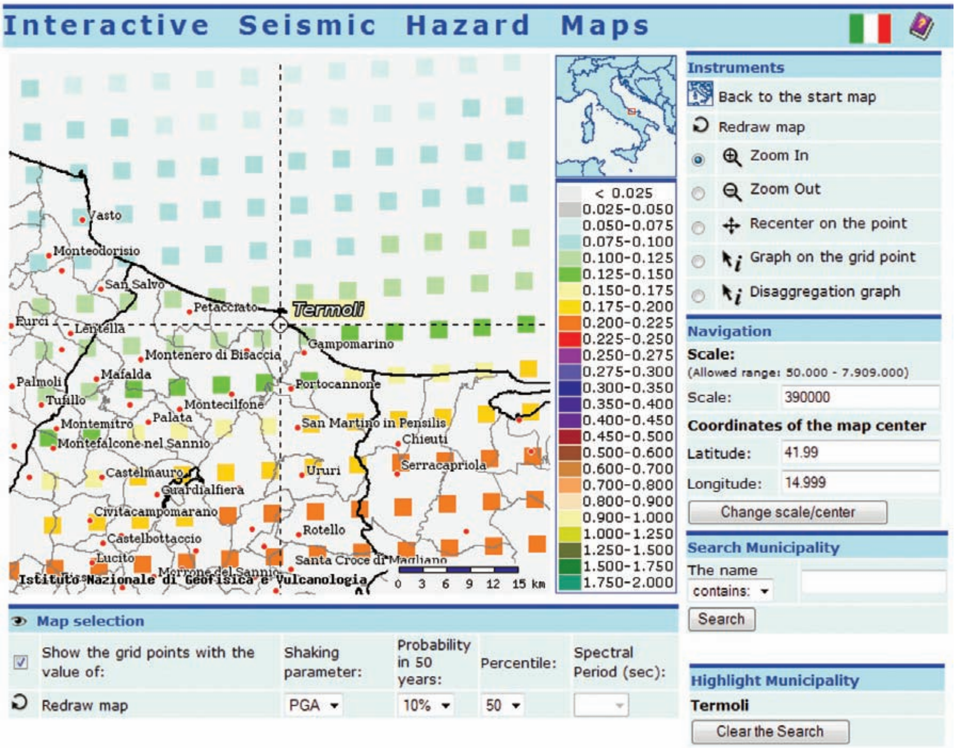
It is worth noting that the PGA for a returning period of 475 years is equal to 0.1248 g. Prior to the introduction of the new code, the city of Termoli was in the 3^{rd} category of the previous seismic code, which used a gross subdivision of the national territory. With the previous code, the acceleration was equal to 0.15 g.

2.1. Subsoil categories

The Geotechnical Earthquake Engineering community is well aware that local soil conditions can greatly modify seismic motion characteristics from those on outcropping bedrock.

In Eurocode 8, site effects are introduced through the determination of ground type, which influences the soil factor and the shape of the design response spectrum.

In the NTC, the same approach is used, and some of the problems encountered in this part of Eurocode 8 are avoided.



Annual frequency of exceedance (AFOE)	Returning period T_r (years)	PGA (Coordinates of the point: Lat.: 41.9746, Lon.: 15.0372, ID: 28106)		
		16th percentile	50th percentile	84th percentile
0.0004	2500	0.1572	0.2175	0.2917
0.0010	1000	0.1167	0.1593	0.2024
0.0021	476	0.0915	0.1248	0.1493
10.0050	200	0.0666	0.0923	0.1005
0.0071	141	0.0574	0.0801	0.0847
0.0099	101	0.0503	0.0713	0.0737
0.0139	72	0.0429	0.0601	0.0644
0.0200	50	0.0361	0.052	0.0557
0.0333	30	0.0275	0.0415	0.0447

Figure 1. Seismic hazard analysis for the city of Termoli (CB) in terms of PGA and uniform hazard spectra on outcropping bedrock (data from http://esse1-gis.mi.ingv.it/s1_en.php?restart=0)).

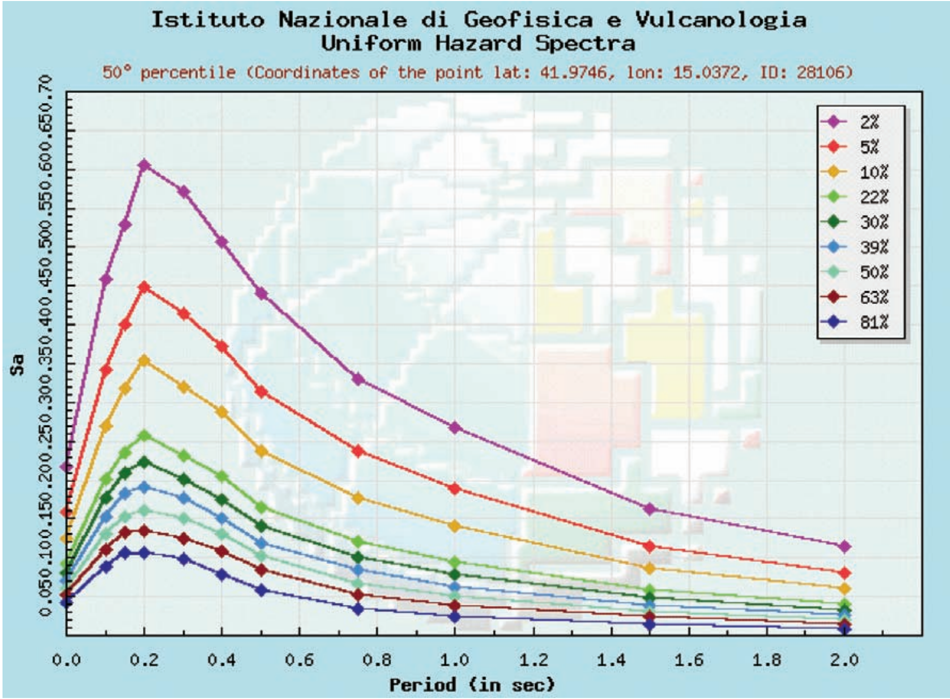


Figure 1. Continued.

In particular, the equivalent shear wave velocity $V_{s,30}$ is introduced, which has been strongly recommended, and an equivalent $N_{SPT,30}$ and an equivalent $C_{u,30}$ are defined.

A clearer definition of the soil depth for which these equivalent parameters may be evaluated is given according to the construction type. The depth should be computed from the embedment depth for shallow foundations; from the pile head for deep foundations; from the wall head for retaining walls for natural soils; and from the depth of the foundation for retaining walls for earthworks.

As for the ground type, it is specified that a deposit can be classified into one of the five conventional categories (from class A to class E) only if a regular increase in its mechanical properties with depth is observed. If not, the site should be classified as S2 and special studies for definition of the seismic action are required.

Further information on this topic can be obtained from documents recently produced by the European Technical Committee ETC-12 that propose improvements to Eurocode 8 (e.g., the proceedings of the Athens workshop, Bouckovalas ed., 2006 or the proceedings of the Madrid workshop, Maugeri ed., 2007).

3. Liquefaction

Changes in the procedures to evaluate liquefaction susceptibility given in Eurocode 8 are introduced in the NTC. In particular, changes in the conditions for exclusion of the liquefaction problem and in the computational analysis are given below.

3.1. Conditions for exclusion of the liquefaction phenomenon

In spite of several case histories of liquefaction (e.g., Galli, 2000), it is a common opinion in the Italian technical community that this phenomenon is of minor concern in Italy. Therefore, the NTC includes a specific paragraph for the conditions under which the liquefaction phenomenon may be excluded.

In the code, it is stated that verification can be avoided when at least one of the following conditions is true:

1. The moment magnitude M_w of the expected earthquake is lower than 5;
2. The maximum expected horizontal acceleration at ground level, in free-field conditions, is lower than 0.1 g;
3. The seasonal average depth of groundwater is greater than 15 m from ground level, for sub-horizontal ground and structures with shallow foundations;
4. The subsoil consists of clean sands having a normalized penetrometer resistance $(N_1)_{60} > 30$ or $q_{c1N} > 180$, where $(N_1)_{60}$ and q_{c1N} are, respectively, the blow count from SPT and the CPT cone resistance, normalized to a vertical effective stress of 100 kPa; and
5. The grading curve distribution lies outside the areas given in Figure 2(a) for soils with a uniformity coefficient $Uc < 3.5$ and in Figure 2(b) for soils with $Uc > 3.5$.

Condition (1) was derived from an analysis of databases of observed liquefaction phenomena. Such data were appropriately combined to derive relationships between the magnitudes of earthquakes and the distances (from the epicenters or from the faults) where liquefaction occurred (Figure 3, modified after ISSEGE TC4, 1999).

Liquefaction phenomena have never been observed for surface wave magnitudes lower than 4.2, even very close to the epicenter. Liquefaction in Italian case histories has a threshold at a magnitude of 5 (Galli, 2000). Therefore, the latter value is given in NTC.

Condition (2) was obtained from evaluation of the peak acceleration at ground level corresponding to the minimum value of the cyclic stress ratio CSR in conventional verification charts, such as those reported in Youd et al. (2001).

Assuming a CSR of 0.050 (the solid circle in Figure 4), the maximum acceleration, for a water table at ground level, is approximately 0.04 g. This figure is directly derived from the classical definition of CSR after Seed and Idriss

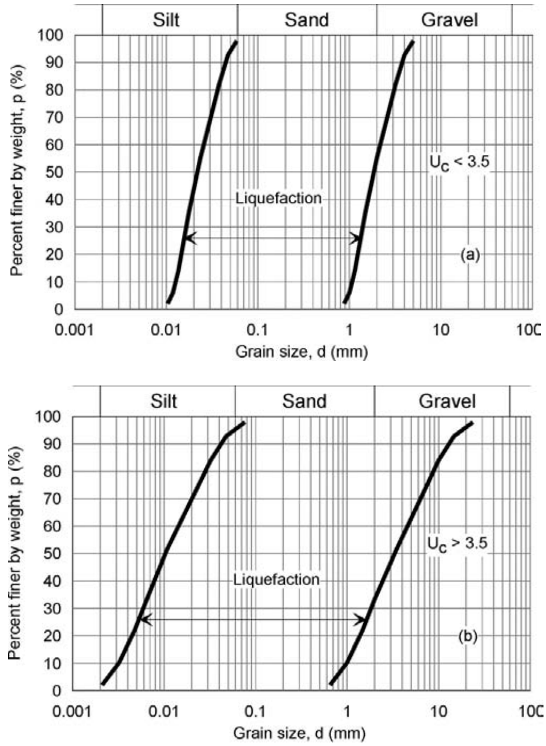


Figure 2. Grading curves for a preliminary evaluation of the liquefaction potential of soils with low and high coefficients of uniformity (modified after Tsuchida, 1970).

(1971): $CSR = \frac{\tau_{av}}{\sigma'_{v0}} = 0.65 \frac{a_{\max s}}{g} \frac{\sigma_v}{\sigma'_v} r_d$, assuming the seismic reduction factor $r_d = 1$ and the total to effective stress ratio $\sigma_v / \sigma'_v = (\gamma_{sat} \cdot z) / (\gamma' \cdot z) = 2$. In this formula, τ_{av} is the induced average cyclic shear stress at the depth of interest and $a_{max\ s}$ is the peak horizontal acceleration at the ground surface generated by the earthquake.

It is worth considering that Yasuda et al. (2004) have shown evidence of liquefaction for the 2003 Tokachi-oki earthquake in Japan ($M_w = 8.0$) in areas where the measured maximum acceleration was equal to 0.05 g. However, the threshold is higher in the NTC because a very low acceleration may cause liquefaction only if generated by an earthquake of very long duration (i.e., recorded far from the epicenter and produced from large earthquakes), which is not expected in Italy.

A recent case of liquefaction was observed in Italy after the 2009 L'Aquila earthquake (Monaco et al., 2011). In this case, the estimated peak ground acceleration on the outcropping bedrock was on the order of 0.065 g. This was

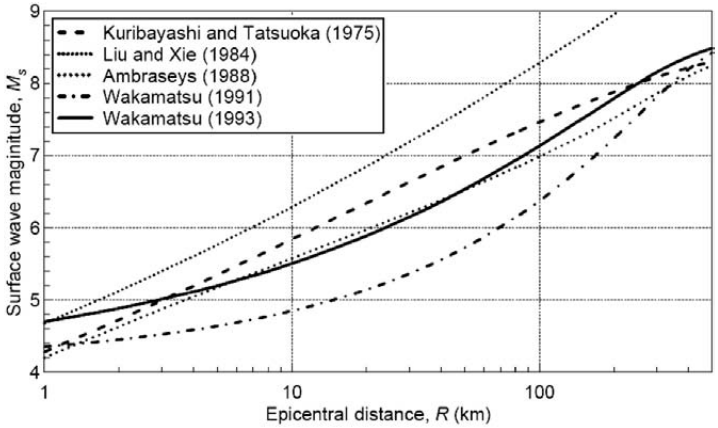


Figure 3. Epicentral distance to farthest liquefied sites R, in km, for surface wave magnitude M_s (modified after ISSMGE- TC4, 1999).

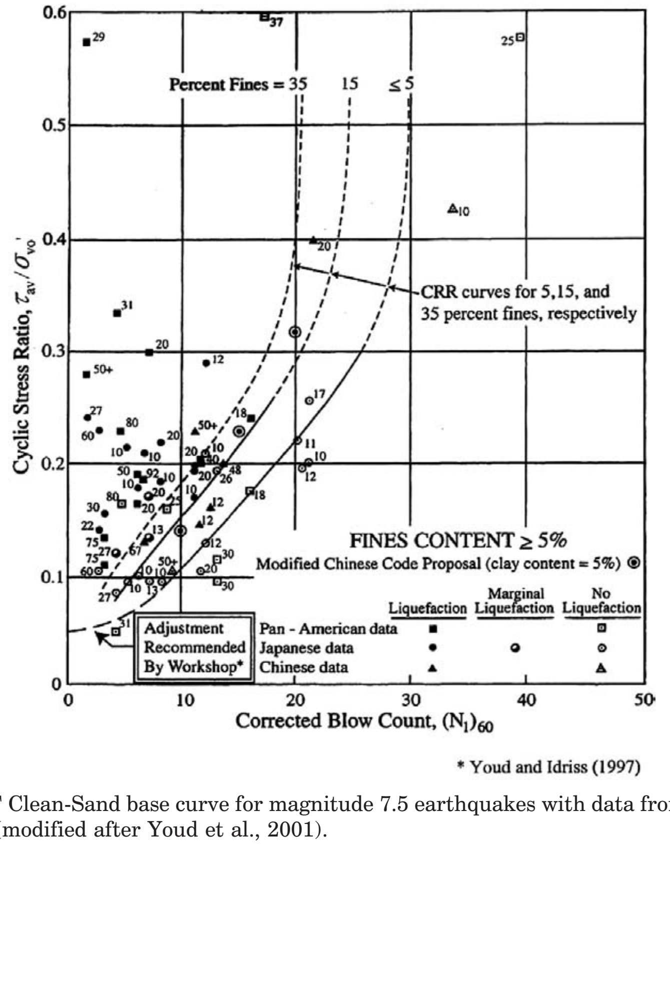


Figure 4. SPT Clean-Sand base curve for magnitude 7.5 earthquakes with data from liquefaction case histories (modified after Youd et al., 2001).

below the limit value in the Italian code, which should be reviewed in the near future.

Condition (3) was directly derived from Eurocode 8, and Condition (4) is an extension of a similar statement in the European Norm. Referring again to Figure 4, a vertical asymptote in the Seed and Idriss-like verification chart seems to exist for the curve separating liquefaction from non-liquefaction case histories (the dotted circle in the figure). It is worth recalling that verification charts allow estimation of the CRR as a function of a normalized parameter that represents the soil resistance to liquefaction; the CRR is the ratio of the shear stress that induces liquefaction to the vertical effective stress.

This asymptote, for sands having a fine fraction equal to or less than 5%, corresponds to $(N_1)_{60} = 30$.

The same asymptote can be seen in charts where the soil properties have been evaluated using the normalized cone penetration resistance. In this case, the threshold value for clean sands corresponds to $q_{c1N} = 180$.

It should be noticed that a similar threshold exists for the normalized shear wave velocity (e.g., the charts given in Andrus and Stokoe, 2000). However, this limit is not included in the new Italian building code.

Finally, Condition (5) is intended to quantitatively express the statement in Eurocode 8: "An evaluation of the liquefaction susceptibility shall be made when the foundation soils include extended layers or thick lenses of loose sand, with or without silt/clay fines, beneath the water table level, and when the water table level is close to the ground surface". Specifically, the grading threshold curves in the NTC were proposed by Tsuchida in 1970 and incorporated into several codes and guidelines (e.g., PHRI, 1997; MoT, 1999; PIANC, 2001).

4. Methods of analysis

In the NTC, it is stated that when the liquefaction phenomenon cannot be excluded a priori, the liquefaction safety factor should be evaluated at depths where potentially liquefiable soils are present. It is also stated that: "Unless advanced analyses are adopted, the verification can be carried out using historical-empirical methodologies in which the safety factor is defined by the relationship between resistance available at liquefaction and the stress induced by the design earthquake. The liquefaction resistance can be evaluated on the basis of the results of in situ tests or based on laboratory cyclic tests. The stress induced by seismic loadings is estimated through the knowledge of the maximum expected acceleration at the depth of interest".

It is also stated that: "the adequacy of the safety factor against liquefaction must be evaluated and motivated by the designer".

It is also stated that: "If the soil is susceptible to liquefaction and the induced effects appear to influence the conditions of stability of slopes or constructions, consolidation interventions and/or transferring of the loads towards layers not subject to liquefaction are needed". Finally, it is stated that: "In the absence of consolidation interventions, the use of deep foundations requires, however, the assessment of the reduction in load-bearing capacity and of the stress increment in piles".

These statements are intended to correct some of the few shortcomings found in Eurocode 8. Eurocode 8 part 5 gives few indications for the use of historical-empirical charts for simplified analysis, and Annex B is devoted to this topic. The seismic shear stress is implicitly neglected in the evaluation, but any stress-reduction coefficient that accounts for the flexibility of the soil column is always reported in the literature for safety. In Eurocode the shear stress is given by:

$$\tau_e = 0.65 \frac{a_g}{g} S \sigma_{v0} \tag{2}$$

where a_g is the design ground acceleration for stiff, type-A ground, g is the acceleration of gravity, S is the soil factor, and σ_{v0} is the total overburden pressure. The use of the soil factor is somewhat unclear because no values are given for the S2 ground type, which consists of deposits of liquefiable soils. Perhaps first it should be assumed that the soil deposit is not subject to liquefaction, and a proper S should be estimated according to the subsoil categories or a conventional site response analysis. Then, if the soil liquefies, specific studies are required for definition of the seismic action on structures (Youd et al., 2001).

In Eurocode 8, is it stated that "soil shall be considered susceptible to liquefaction under level ground conditions whenever the earthquake-induced shear stress exceeds a certain fraction λ of the critical stress known to have caused liquefaction in previous earthquakes. The value ascribed to λ for use in a Country may be found in its National Annex. The recommended value is $\lambda = 0.8$, which implies a safety factor of 1.25".

As mentioned above, this indication was not incorporated into the NTC. In Eurocode, it seems that at a given site, if the "demands" exceed 0.8 times the "capacity" at any depth (or the value indicated by each single country), some "actions" should be taken because liquefaction is expected to be triggered. This indication seems too restrictive because the liquefaction phenomenon is a global occurrence over the soil vertical rather than a punctual event. In this case, only partial help can be found in the European norm from the statement: "If soils are found to be susceptible to liquefaction and the ensuing effects are deemed capable of affecting the load bearing capacity or the stability of the foundations, measures, such as ground improvement and piling (to transfer loads to layers not susceptible to liquefaction), shall be taken to ensure foundation stability".

Further details on liquefaction are given by Santucci de Magistris (2006).

5. Slope stability

One of the greatest shortcomings of the geotechnical part of EC8 is the use of pseudostatic methods to evaluate seismic action for slope stability analyses and for retaining wall computations. This is because the pseudostatic forces evaluated following the Eurocode rules, together with the large expected design accelerations, appear to be particularly elevated in some areas, thus compromising the stability of such geotechnical systems and structures.

The NTC tries to overcome this difficulty while retaining the framework of the Eurocode approach.

First, in the NTC, it is clearly stated that slope stability under seismic action can be evaluated with pseudostatic methods, displacement methods and dynamic analysis methods.

It is also stated that: "In pseudostatic methods the seismic action is represented by a static equivalent force, that is constant in the space and in the time, and that is proportional to the weight W of the soil in the volume potentially unstable. This force depends upon the characteristics of the seismic motion expected in the volume of soil potentially unstable and the capacity of this volume to be subject of movements without significant reductions of resistance."

In the absence of specific studies, for ultimate limit state analyses, the horizontal and vertical components of the pseudostatic forces are given by:

$$F_h = k_h \cdot W; \qquad F_v = k_v \cdot W \tag{3}$$

where k_h and k_v are, respectively, the horizontal and vertical seismic coefficients:

$$k_h = \beta_s \cdot \frac{a_{\max}}{g} \tag{4}$$

$$k_v = \pm 0.5 \cdot k_h \tag{5}$$

where β_s is a reduction coefficient of the maximum expected acceleration at the site a_{max} and g is the gravitational acceleration.

The values of the reduction coefficient β_s are given in Table 2.

The limit state condition must be determined with reference to the characteristic values of the geotechnical parameters and referred to the critical slide surface, characterized by a lower safety margin. The adequacy of the safety margin against slope stability must be evaluated and justified by the designer.

In the NTC, it is stated that analysis of the behavior of slopes under seismic conditions may also be performed with the displacement method, in which a mass of soil that is potentially unstable is treated as a rigid body that can move along a sliding surface. The application of this method requires that the design seismic action be represented by acceleration time histories. The accelerograms used in analysis should not be less than five in number and must

Table 2. Reduction coefficient for the maximum expected horizontal acceleration at a site, as reported in the NTC (2008).

	Subsoil category	
	A	B, C, D, E
	β_s	
$0.2 < a_g\ (g) \leq 0.4$	0.30	0.28
$0.1 < a_g\ (g) \leq 0.2$	0.27	0.24
$a_g\ (g) \leq 0.1$	0.20	0.20

be representative of the seismicity of the site. The choice of accelerograms must be adequately justified. It should be noted that in the NTC, the use of artificial accelerograms is not allowed for evaluation of slope displacements.

The choice of acceptable displacement values, for limit state conditions or for serviceability limit states, must be made and properly justified by the designer.

Information on the approach used to derive the equivalent seismic coefficient can be found in Fargnoli et al., 2007 and Rampello et al., 2008. The criterion is based on the equivalence between the pseudostatic method and the Newmark-type displacement method, for given allowable displacements. In the Fargnoli paper, it is stated that the equivalent seismic coefficient should depend at least on the seismically induced displacement d and the maximum acceleration expected at the site a_{max}. Moreover, d depends on the ratio a_y / a_{max}, where a_y is the Newmark-type critical acceleration; obviously, an increase in the a_y / a_{max} ratio corresponds to a decrease in the seismic induced displacement.

Seismic induced displacements were computed by means of the original Newmark method (Kramer, 1996) for time-independent critical accelerations and space-independent maximum accelerations using 214 acceleration time--histories from Italian earthquakes (Scasserra et al., 2009). Accelerograms were roughly grouped according to the soil characteristics (rock, stiff and soft soil) below each recording station and scaled to include PGAs in the following acceleration intervals: 0.4 g to 0.3 g, 0.3 g to 0.2 g, 0.2 g to 0.1 g, and < 0.1 g. Displacements were computed for an a_y / a_{max} ratio varying over the interval [0.1, 0.8]; each accelerogram was considered according to the two possible methods of application.

The results were plotted on a bilogarithm plot and interpolated using the following expression:

$$\log d = \ln B + A \frac{a_y}{a_{max}} \tag{6}$$

Then, the upper limit of the regression associated with the probability of not exceeding 90% was considered, assuming a normal distribution of the data around their mean value.

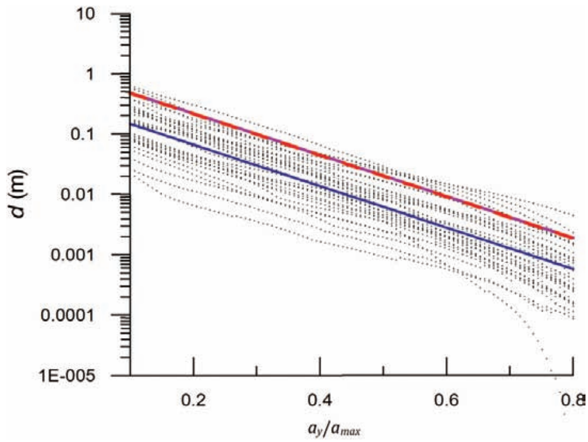


Figure 5. Seismic induced displacement d versus a_y / a_{max} (modified after Fargnoli et al., 2007).

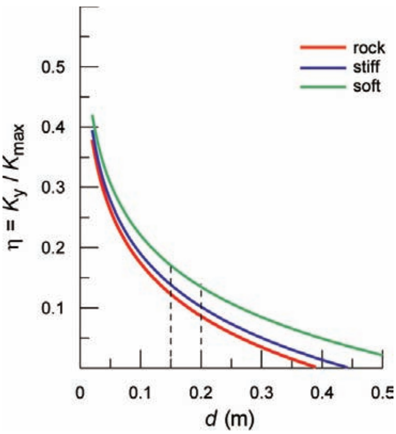


Figure 6. a_y / a_{max} versus seismic induced displacement d for different subsoil conditions (modified after Fargnoli et al., 2007).

Figure 5 shows the calculated seismic induced displacement versus the a_y / a_{max} ratio for rock soil. The maximum acceleration is included in the 0.3 g to 0.4 g interval.

The same data from Figure 5 are rearranged in Figure 6, together with the results obtained for stiff and soft soils.

Once d as a function of the a_y / a_{max} ratio was evaluated, it was possible to compute β_σ as the a_y / a_{max} ratio for a given displacement threshold value d_c. The values of the reduction coefficient in the NTC were obtained assuming an allowable displacement of 20 cm.

6. Conclusion

After the 2002 Molise earthquake, a series of changes in seismic codes was adopted in Italy. This process was completed at the beginning of 2008 when a comprehensive new building code was released. The new code was inspired by Eurocodes, but it included some changes and improvements to the European norm.

In this paper, a few aspects interesting to geotechnical earthquake engineers are discussed, including evaluations of seismic motion, liquefaction and slope stability.

In the Italian code, evaluation of seismic motion is based on detailed seismic hazard study, allowing determination of seismic parameters on outcropping horizontal bedrock (i.e., PGA or design response spectra) over the entire national territory and for multiple returning periods. The influence of local conditions on seismic motion is mainly determined from ground classification, a slight modification to the indication of Eurocode.

For liquefaction analysis, detailed attention is given to the conditions for exclusion of the liquefaction phenomenon, and evaluation of the acceptability of the overall safety is left as the responsibility of the designer.

The same applies for evaluation of slope stability; however, a novel approach to computing pseudostatic forces is proposed in the new Italian building code. Additional papers describing the procedure for slope stability analysis in seismic areas will soon appear in the technical literature.

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SAŽETAK

Nakon EC8: novi talijanski propisi za protupotresnu gradnju

Filippo Santucci de Magistris

Iako su potres u pokrajini Molize, koji se dogodio 2002. godine, seizmolozi kategorizirali kao uobičajenu geodinamičku pojavu na talijanskom poluotoku, on je imao veliki odjek u javnosti jer je prouzročio rušenje jedne osnovne škole. Taj je događaj u Italiji inicirao mnoga istraživanja u području potresnog inženjerstva i značajne izmjene zakona o gradnji. Te su izmjene dovršene početkom 2008. godine, kada je obznanjen novi, detaljno razrađen, talijanski zakon o protupotresnoj gradnji. Taj je zakon izrađen po uzoru na Eurokod, ali donosi i neke novine i unaprjeđenja. Iz tog se zakona u ovom članku komentiraju: definicija seizmičkog opterećenja, te analize potencijala likvefakcije i stabilnosti kosina. Seizmičko je opterećenje određeno na temelju nedavnih detaljnih studija seizmičkog hazarda na području Italije. Što se tiče likvefakcije, prikazane su neke novine u odnosu na Eurokod. Konačno, u vezi stabilnosti kosina unesene su izmjene u odnosu na Eurokod da se izbjegnu prevelike pseudostatičke sile u konvencionalnim analizama stabilnosti.

Ključne riječi: geotehničke kategorizacije, likvefakcija, seizmički odziv lokalnog tla, stabilnost kosina, seizmički propisi

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