Thessaloniki-GREECE 2007

4th International Conference on
Earthquake Geotechnical Engineering
June 25-28, 2007
Paper No. 1557

ASSESSMENT OF EC8 AND ITALIAN CODE PRESCRIPTIONS FOR SEISMIC ACTIONS

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ABSTRACT

This paper compares the elastic response spectra, prescribed by EC8 (2003) part 1, to those inferred by means of 1D equivalent - linear analyses and non – linear analyses in a number of well characterized sites in Tuscany (Italy). Soil characterization has been done in the framework of a program supported by the Regional Government of Tuscany and devoted to the assessment of local site effects and to the reduction of seismic risk. Reference spectra have been selected according to the type of soil and expected Magnitude. Influence of the variability of soil properties and input motion on the response spectra is considered. Identification of the type of soil has been accomplished by means of both shear wave velocity profile (V_{s30}) and penetration test results (N_{spt}). Limitations and capabilities of SPT and other dynamic penetration tests are discussed. The relevant prescriptions of recently established Italian codes (OPCM - 3274 2003, Norme Tecniche per le Costruzioni 2005), which represent a partial implementation of EC8 in Italy, are also discussed. Basic steps of conventional hazard studies, aimed at defining seismic input motion on rock, are critically discussed.

Keywords: microzonation, seismic response analysis, seismic hazard

INTRODUCTION

EC8 (2003) part 1, prescribes two sets (Type 1 & 2) of elastic response spectra, depending on the expected Magnitude of the design earthquake. Each set consists of five different spectra which depend on the type of soil. the horizontal elastic response spectrum is defined by eqs. (1) to (4).

$0 \le T \le T_B$ $$S_e(T) = a_g S[1 + \frac{T}{T_B}(\eta \cdot 2.5 - 1)] \quad (1)$$

$T_B \le T \le T_C$ $$S_e(T) = a_g S \cdot \eta \cdot 2.5 \quad (2)$$

$T_C \le T \le T_D$ $$S_e(T) = a_g S \cdot \eta \cdot 2.5 \frac{T_C}{T} \quad (3)$$

$T_D \le T \le 4s$ $$S_e(T) = a_g S \cdot \eta \cdot 2.5 \frac{T_C T_D}{T^2} \quad (4)$$

where: $S_e(T)$ = elastic spectrum ordinate; T = period of a SDOF system; $a_g = \gamma_I k \cdot a_{gR}$ is the design ground acceleration for type A soil, established by National Authorities, for a given reference return period T_{NCR} (suggested value for ULS 475 years) or reference probability of exceedance P_{NCR} in 50 years (suggested value for ULS 10 %) or a given life time of the construction T_L; γ_I = importance factor to account for different return period; k = modification factor to account for special regional

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situations; η = damping correction factor (η = 1 when the structural damping ξ = 5 %); T_B, T_C = limits for constant acceleration; T_D limit beyond which the elastic response displacement is constant; S = soil parameter.

National annexes should define appropriate values of k (it is suggested to assume k = 1) and, trough zonation maps, a_{gR}.

Parameter S, and periods T_B, T_C, T_D depend on the type of soil. Table 1 and 2 summarize respectively for Type 1 & 2 elastic spectra the values prescribed for the above parameters. Type 2 elastic spectra should be used in the case that the earthquakes which mostly contribute to the seismic hazard (in the framework of a probabilistic approach) have Magnitude of surface wave $M_S \leq 5.5$.

Table 1. Parameters for type 1 Horizontal Elastic Spectrum

Soil	S	T_B (s)	T_C (s)	T_D (s)
A	1.0	0.15	0.40	2.0
B	1.2	0.15	0.50	2.0
C	1.15	0.20	0.60	2.0
D	1.35	0.20	0.80	2.0
E	1.4	0.15	0.50	2.0

Table 2. Parameters for type 2 Horizontal Elastic Spectrum

Soil	S	T_B (s)	T_C (s)	T_D (s)
A	1.0	0.05	0.25	1.2
B	1.35	0.05	0.25	1.2
C	1.5	0.10	0.25	1.2
D	1.8	0.10	0.30	1.2
E	1.6	0.05	0.25	1.2

Italian codes (OPCM – 3274 2003, Norme Tecniche per le Costruzioni 2005), in an attempt to introduce Eurocodes in Italy, define elastic spectra using equations (1) to (4), but, surprisingly, different spectra are prescribed for ULS and DLS (Norme Tecniche per le Costruzioni 2005). On the other hand, spectra, prescribed by the Italian codes, do not depend on the expected Magnitude. The parameters that should be used for the ULS & DLS are reported in Table 3a and 3b.

Table 3a. Parameters for ULS Horizontal Elastic Spectrum (OPCM 3274, 2003; Norme Tecniche per le costruzioni, 2005)

Soil	S	T_B (s)	T_C (s)	T_D (s)
A	1.0	0.15	0.40	2.0
B, C, E	1.25	0.15	0.50	2.0
D	1.35	0.20	0.80	2.0

Table 3b. Parameters for DLS Horizontal Elastic Spectrum (Norme Tecniche per le costruzioni, 2005)

Soil	S	T_B (s)	T_C (s)	T_D (s)
A	1.0	0.05	0.25	1.2
B, C, E	1.5	0.05	0.25	1.2
D	1.8	0.10	0.30	1.2

Soil types are defined in the same way by EC8, OPCM – 3274 and Norme Tecniche per le Costruzioni. Such a definition is based on stratigraphic profile, average shear wave velocity of the first 30 m (V_{s30}), number of blow/(30 cm) from SPT (N_{spt}), and undrained shear strength (C_u). Table 4 summarizes the criteria, prescribed to identify the soil type, in terms of V_{s30} and N_{spt}. The above mentioned codes also consider very loose or soft soils (S1 and S2) which are not discussed in this paper.

Table 4. Ground conditions

Soil type	V_{s30} (m/s)	N_{spt} (blow/30cm)
A	> 800	-
B	360 - 800	> 50
C	180 - 360	15 -50
D	< 180	< 15
E	(*)	-

(*) Soil type E consists of 5 to 20 m of soil like C or D underlain by soil type A. The above reported definition is the authors' interpretation of what reported on the cited codes. In fact EC8 and OPCM 3274 define type E soil in an ambiguous way, referring to V_{s30} values for layers not exceeding 20 m thickness.

As for the seismic zonation, the Italian codes prescribe the design acceleration as defined inTable 5. The final decision for zonation is up to the Regional Governments which can also introduce sub – zones with intermediate values of a_{gR}. The PGA values reported in Table 5 have been obtained by means of a probabilistic approach, considering a return period of 475 years (i.e. a 90 % of probability of non –exceedance in 50 years).

Table 5. Design ground acceleration for ULS (Italian codes)

Zone	PGA	a_{gR}
4	< 0.05	0.05
3	0.05 – 0.15	0.15
2	0.15 – 0.25	0.25
1	> 0.25	0.35

PGA = Peak ground acceleration from probabilistic hazard studies. a_{gR}= prescribed design acceleration on type A soil for the ULS.

It is quite obvious, looking at the seismic zonation criteria reported in Table 5, that the macro-zonation recently adopted in Italy introduces large safety factors. On the other hand, Regional Governments have the possibility of introducing sub-zones, which leads to a more detailed macro – zonation map with smooth variation of the prescribed design acceleration. Such a possibility require a more detailed hazard study (the basic study has been run considering a net with a 0.05° step) and an evaluation of local site effects for ground motion.

Another fundamental issue, dealing with Eurocode 8 (2003) or the recent Italian codes is the identification of soil type, which, in principle, could be done referring to the penetration resistance of SPT (N_{SPT}) or the undrained shear strength C_u of fine grained soils from laboratory or in-situ tests or the average shear wave velocity of first 30 m ($V_{s,30}$) from in situ seismic tests.

Taking advantage of a large database, made available by the Regional Government of Tuscany – Italy, soil identification by means of penetration tests, seismic down-hole tests and seismic (SH) refraction tests has been compared in a number of well characterized sites. Moreover, for a number of well characterized sites, the PGA and response spectra, obtained from linear-equivalent and non-linear seismic response analyses, have been compared to those prescribed by the Italian codes in order to assess the effectiveness of such a regulations.

VALIDATION OF DIFFERENT CRITERIA TO IDENTIFY SOIL TYPE

The available database consists of investigations performed in 71 sites located in 34 different towns of Tuscany. For each site, data consists of the following:

- stratigraphic profile from boreholes, with SPT measurements;
- shear wave velocity (V_S) profile from Down – Hole (DH) tests performed in the boreholes;
- shear wave velocity (V_S) profile from seismic refraction tests performed generating both compression (P) and horizontally polarized shear waves (SH);

- resonant column tests (RCT), cyclic loading torsional shear tests (CLTST), cyclic loading triaxial tests (CLTX) performed on undisturbed samples (not available for all boreholes and any type of soil);
- dynamic penetration tests (DP), performed by using a cone 51 mm in diameter and with an apex angle of 60° (hammer weight and falling height are the same as for the SPT, blowcounts every 20 cm penetration). This type of tests has been performed in few sites.

The above mentioned investigations have been performed to verify the seismic requirements of existing public constructions (schools, etc.) in the light of the new Italian codes. Nonetheless the huge amount of data, actually only a part of the available results has been analyzed, because the data are undergoing a validation process.

Soil type has been identified by means of N_{SPT} and V_{s30} at 25 different sites. Table 6 compares the soil type identification as obtained trough different test results. It is possible to draw the following conclusions:

- SH seismic refraction tests, which usually do not provide a very accurate shear wave velocity profile, lead to an estimate of V_{s30} not very different from that obtained from DH tests, giving the opportunity to save costs;
- in those cases where the SH tests are not applicable, because of the intrinsic limits of the test, the use of surface waves (i.e. SASW) can be suggested;
- penetration test results gave different indications only in three cases. More specifically, in such cases, SPT results classified the soils in the category immediately above or below that selected on the basis of V_{s30};
- nonetheless the penetration test results lead to an identification of soil type similar to that inferred from shear wave velocity measurements, the use of N_{spt} is not simple and requires an expert evaluation.

Table 6. soil type identification

Case	$V_{s30\ D\text{-}H}$	Soil type $_{D\text{-}H}$	$V_{s30\ s\text{-}H}$	Soil type $_{S\text{-}H}$	Soil type N_{SPT}
S1	536.17	B	671.85	B	B
S2	489.84	B	572.41	B	-
S3	736.36	B	944.13	A	-
S4	372.44	B	428.41	B	-
S5	364.83	B	452.38	B	A
S6	592.16	B	568.50	B	-
S7	739.31	B	632.27	B	B
S8	489.40	B	390.40	B	-
S9	426.17	B	432.53	B	C
S10	571.10	B	390.02	B	C
S11	517.57	B	426.74	B	-
S12	626.70	B	499.80	B	B
S13	892.76	A	825.91	A	-
S14	810.43	A	746.95	B	B
S15	666.01	B	535.14	B	B
S16	706.52	B	1117.77	A	C
S17	538.28	B	562.12	B	B
S18	490.62	B	382.76	B	B
S19	598.20	B	532.84	B	B
S20	557.86	B	780.85	B	B
S21	460.39	B	544.22	B	C
S22	503.95	B	404.02	B	B
S23	373.73	B	405.01	B	C
S24	679.82	B	718.87	B	-
S25	848.96	A	759.02	B	-

COMPUTED VS. PRESCRIBED RESPONSE SPECTRA

Elastic response spectra (ξ = 5%) have been computed at 19 different sites by means of 1D equivalent – linear analyses run by means of EERA (Bardet et al. 2000). Non-linear analyses (1D) have also been performed at three of the above considered sites, by means of ONDA (Lo Presti et al. 2006a). Analyses have been repeated to account for parameter variability. More specifically, the following parameter variability has been considered:

- variability at a given site of the shear wave velocity (V_s) profile as inferred from DH and SH tests, which lead to maximum and minimum values of V_s for each macro-stratum (macro-strata have been defined on the basis of geology. Local variability of strata thickness is negligible);
- laboratory tests (Resonant Column Test – RCT, Cyclic Loading Torsional Shear Test – CLTST, Cyclic Loading Triaxial Test – CLTX) were used to determine the variation of stiffness (G) and damping ratio (D) with shear strain (γ). For a given geologic formation the variability of stiffness and damping ratio has been considered by repeating the computations with the upper and lower envelopes of the G/Go-γ D-γ curves. Because of the limited number of laboratory tests, in comparison to the extension of the investigated area, the whole variability of a given geologic formation has been considered. The complete set of curves has been published elsewhere (Lo Presti et al. 2006b);
- variability of input motion has been considered by means of the procedure outlined below (Mensi et al. 2004, Lai et al. 2005);

    - definition of PGA at each site by means of standard probabilistic hazard analysis;
    - de-aggregation of the hazard analysis (T_R = 475 years) to obtain the Magnitude-distance couple which mostly contribute to determine the hazard. The following couples have been obtained: M=5.3 d=10.2 km (Mensi et al. 2004), M=5.4 d= 13 km (Lai et al. 2005);
    - selection of free-field natural accelerograms seismically compatible with the M-d couples, given a certain tolerance. The largest tolerance was adopted by Lai et al. 2005 in order to guarantee the compatibility with the elastic response spectra prescribed by the Italian code. More specifically a tolerance of ΔM=0.5 and distance intervals of d=9-17 km and 5-20 km were adopted by Lai et al. (2005).

On the basis of the above described procedure, three accelerograms selected by Mensi et al. 2004 (A1, A2, A3) and another three selected by Lai et al. 2005 (A6.1, A6.2, A6.3) have been used.
As an alternative, an artificial accelerogram, obtained from the probabilistic elastic spectrum on rock (Ferrini et al. 2000) according to the procedure suggested by Sabetta and Pugliese (1996) was used.

For each of the seven selected accelerograms the analyses summarized in Table 7 have been done.

Table 7. analyses performed

Case Number	Vs (bedrock)	Vs (surface layers)	G-γ D-γ envelopes
1	Max	Min	Max
2	Max	Min	Min
3	Min	Min	Max
4	Min	Min	Min
5	Max	Max	Max
6	Max	Max	Min

To evaluate analysis results, PGA, elastic response spectra and a synthetic amplification parameter have been considered.
The amplification parameter is defined in the following way:

$$F_a = \frac{\int_{0.1}^{0.5} PSA(deposit)}{\int_{0.1}^{0.5} PSA(outcrop)} \quad (5)$$

where: PSA = spectral acceleration computed at the top of the deposit or at outcrop, integrated between 0.1 and 0.5 s (corresponding to the most recurrent periods of the constructions existing in the study area).

The above definition of a synthetic amplification parameter is similar, but not coincident, to that proposed by Pergalani et al. (1999).

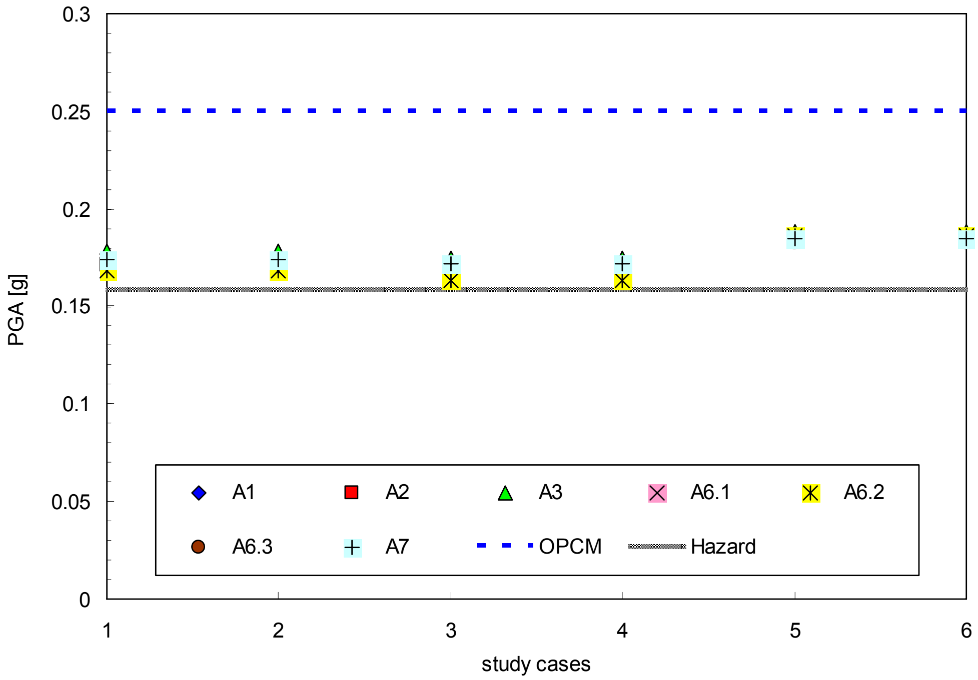

Figure 1a. Computed vs. prescribed PGA (type A soil)

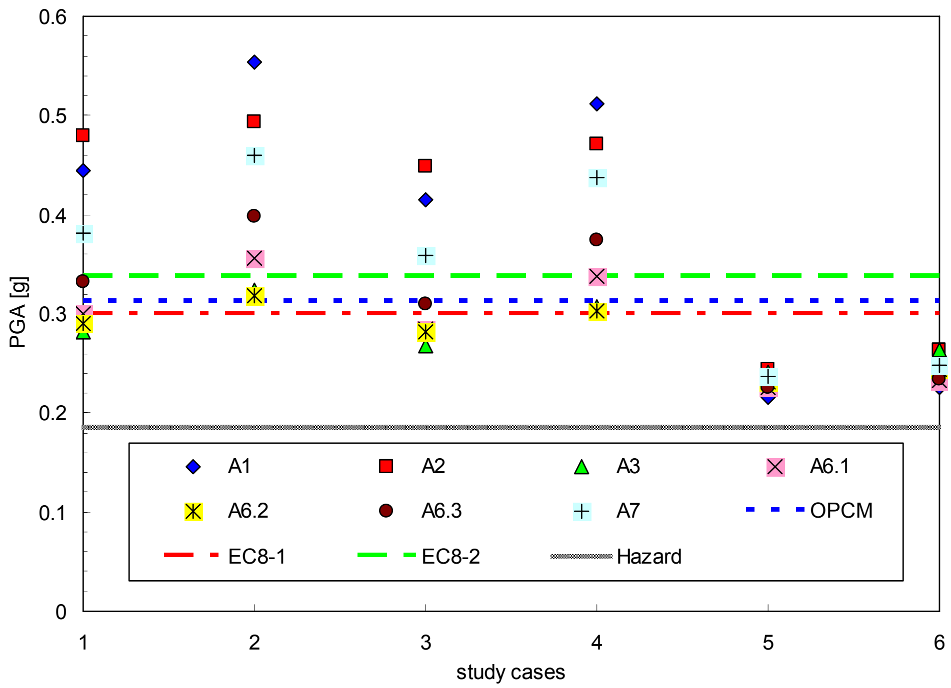

Figure 1b. Computed vs. prescribed PGA (type B soil)

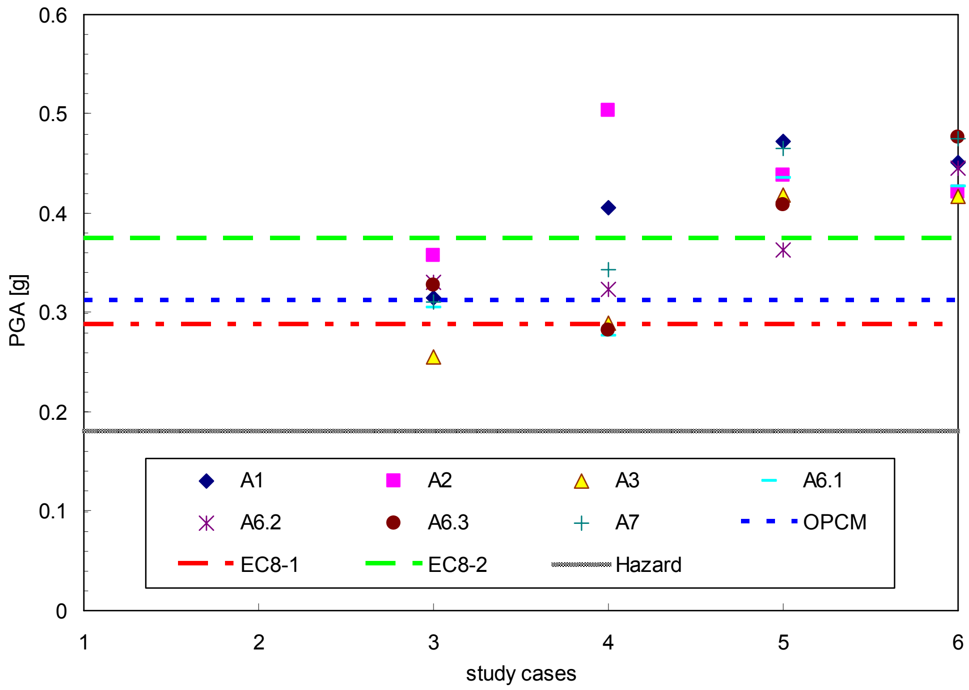

Figure 1c. Computed vs. prescribed PGA (type C soil)

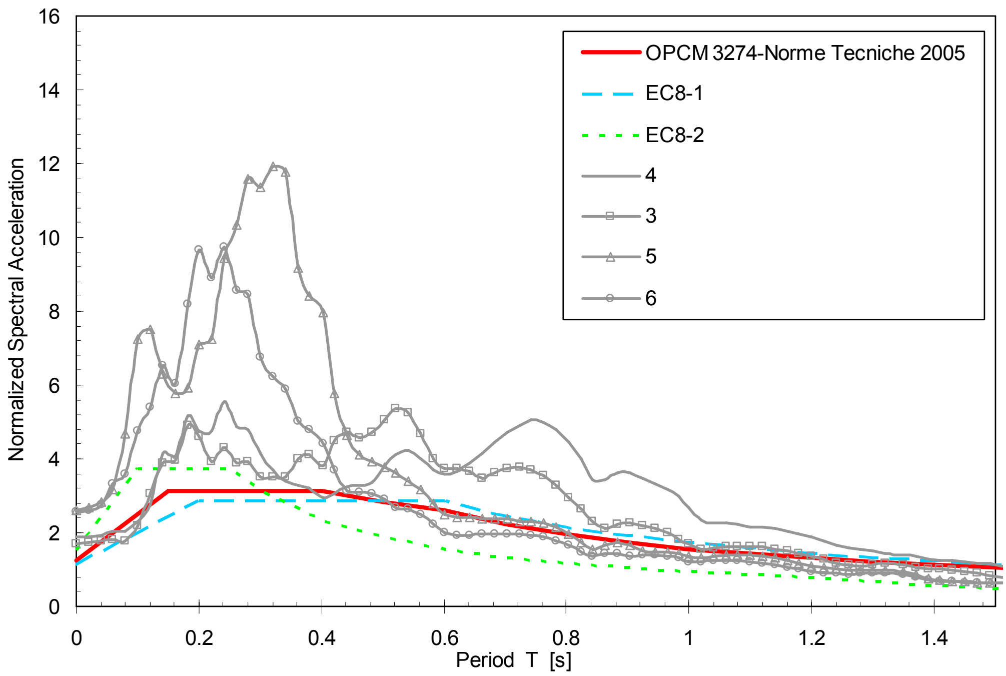

Figure 2. Elastic response spectra – Type C soil – Artificial accelerogram

Figures 1a to 1c show the PGA obtained at the top of soil deposits, by means of linear equivalent analyses at three sites, classified respectively as Type A, B and C soils.
Figure 1a refers to type A soil. For this case stratigraphic amplification seems negligible if not zero. Anyway, because of the conservative criteria of Table 5, in type A soils the computed PGA never exceed the prescribed design acceleration, as shown in Figure 1a.

Figures 1b and 1c refer to type B and C soil respectively. In these Figures the effect of both input accelerograms and ground conditions on PGA is seen. It appears that the variability of PGA because of differences in ground conditions is comparable to that due to differences in input motion. More specifically, the ground conditions that mostly affect the response are: a) shear wave velocity profile and b) soil damping ratio (as for the soil parameters). These effects are shown in Figure 2 where the response spectra for type C soil have been plotted considering the artificial input accelerogram (the computed spectra have been normalized with respect to the PGA on rock, while the prescribed ones have been divided by a_{gR} of Table 5).

Design accelerations have been plotted in Figures 1b and 1c. The design accelerations have been obtained multiplying a_{gR}, prescribed by the Italian Code, by the soil factor S prescribed for type B and C soils by OPCM 3274 (2003) and those prescribed by EC8 for the same type of soils in the case of type 1 (EC8-1) and 2 (EC8-2) spectra. It is evident that in some cases the prescribed design accelerations are lower than those obtained from linear – equivalent seismic response analyses, nonetheless the conservativisme introduced by Table 5 (OPCM).

Table 8 summarizes the average PGA, obtained from calculations for the three sites, and the design accelerations prescribed by OPCM and EC8.

Table 8. Computed PGA and prescribed design accelerations

Soil type	PGA(*)	OPCM	EC8-1	EC8-2
A	0.176	0.250	0.250 (-)	0.250 (-)
B	0.330	0.312	0.300	0.338
C	0.385	0.312	0.288	0.375

(*) Average; (-) value from Table 5

It is clear that for type B soil the case EC8-2 gives values of the design acceleration greater than that computed, while for type C soil the prescribed value is slightly lower than that computed. Not considering soil type A, where the above considerations are meaningless, the other cases (OPCM, EC8-1) prescribe design accelerations lower than those computed.

A general comparison between computed and prescribed design accelerations is reported in Table 9. More specifically, Table 9 indicate, for the 19 investigated sites the percentage of cases where the following condition is observed $F_a * PGA > Sa_{gR}$ + a given tolerance in (g) ranging from 0.025 to 0.075 (where: F_a is the synthetic amplification parameter, computed according to eq. 9; PGA = Peak Ground Acceleration on rock obtained from hazard analysis; S is the soil factor prescribed by OPCM and EC8; a_{gR} = design acceleration on rock as prescribed by OPCM, see Table 5).
It is worthwhile to remark that even for type A soils a certain amplification can occur. Generally, the computed accelerations remain smaller than the prescribed ones because of the conservativisme introduced by the rules of Table 5.

Table 9. Prescribed vs. computed ground accelerations

Condition	% of Cases		
	OPCM	EC8-1	EC8-2
$F_a * PGA > Sa_{gR} + 0.025g$	25.9	29.6	18.5
$F_a * PGA > Sa_{gR} + 0.050g$	7.4	11.1	7.4
$F_a * PGA > Sa_{gR} + 0.075g$	7.4	7.4	7.4

Data in Table 9 confirm that the values of the soil parameter S (as prescribed by EC8-2, EC8-1 and OPCM), together with the prescriptions of the Italian code for the design acceleration on rock (Table 5) give design acceleration values greater than those prescribed, with very few exceptions. Prescriptions of EC8-2 seem the most conservative for the cases under consideration.

The influence on soil response of input accelerogram clearly appears by comparing Figures 2, 3 and 4 which show the prescribed and computed elastic response spectra.. The computed spectra have been divided by the PGA as obtained by hazard analysis on rock. The prescribed spectra have been divided by a_{gR} of Table 5.

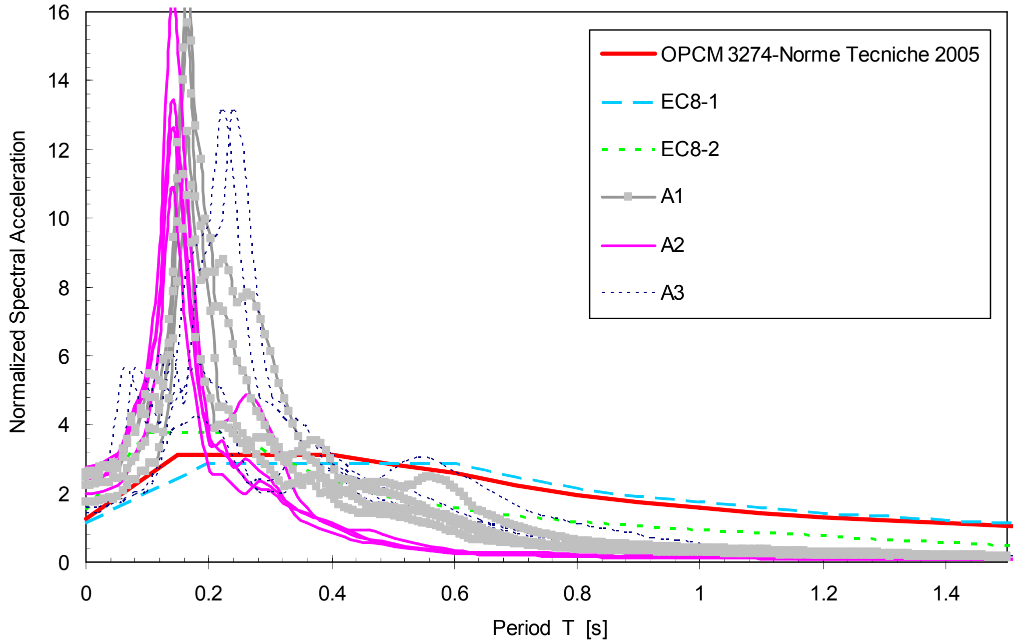

Figure 3. Elastic response spectra – Type C soil – Accelerograms A1-A2-A3: M = 5.3; d = 10.2 km

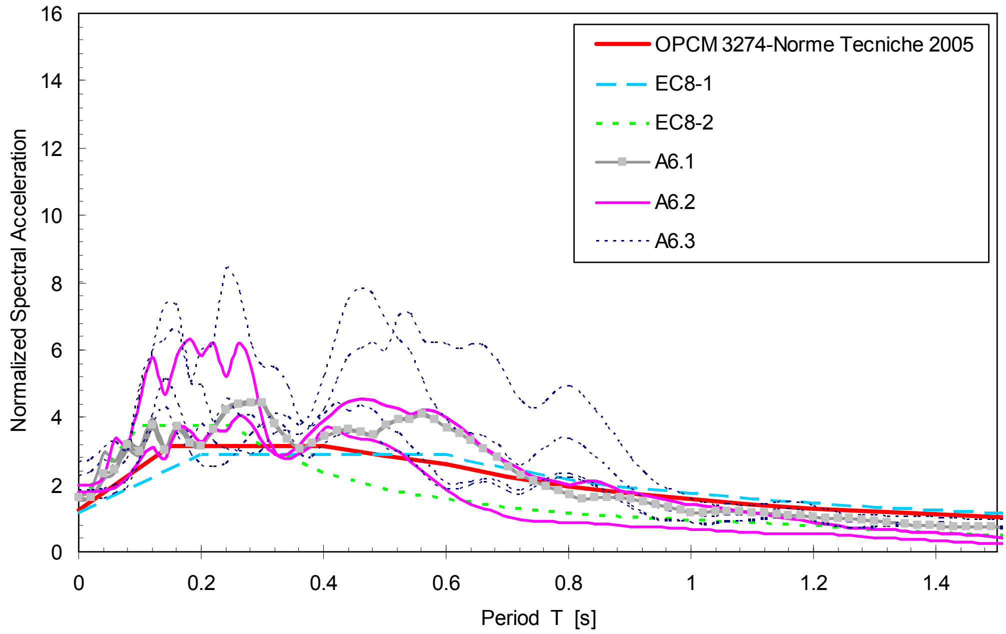

Figure 4. Elastic response spectra – Type C soil – Accelerograms A6.1-A6.2-A6.3: ΔM = 0.5; d = 9-17 km; 5-20 km

More specifically:

- natural accelerograms, (A1, A2, A3) selected to represent earthquakes with M=5.4 and d=13 km, lead to high spectral accelerations (PSA) in the high frequency range (T=0.1-0.2s) while PSA are much smaller than that prescribed by EC8 and OPCM 3274 at T>0.2 s;
- the second set of natural accelerograms (A6.1, A6.2, A6.3), selected considering a range of Magnitude and distances (ΔM=0.5; d=9-17 km; 5-20 km), give the same picture, but the PSA spikes in the high frequency range are less pronounced whilst, at larger periods (T>0.2s) the underestimation observed for accelerograms A1 A2 and A3 disappear;
- the artificial accelerogram leads, on the whole, to a better agreement between computed and prescribed spectra especially at periods greater than 1s
- computed PSA may be greater than prescribed PSA for the most unfavorable ground conditions, anyway the average values are generally smaller. Figures 5a, 5b and 5c show the average response spectra obtained at three different locations classified as type A (Figure 5a), type B (Figure 5b) and type C (Figure 5c) soils. Shear wave velocity profiles and V_{s30} have also been shown. For each type of soil a case where $F_a * PGA < Sa_{gR}$ has been considered.

  It is possible to see that the shape of the prescribed spectra (EC8-2) is more appropriate for type A and B soils, whereas for type C soils the shape of the spectra prescribed by OPCM 3274 (2003) or EC8-1 seems more appropriate. It is worthwhile to remark that the prescribed PSA values have been obtained by using the pertinent values of the S parameter.

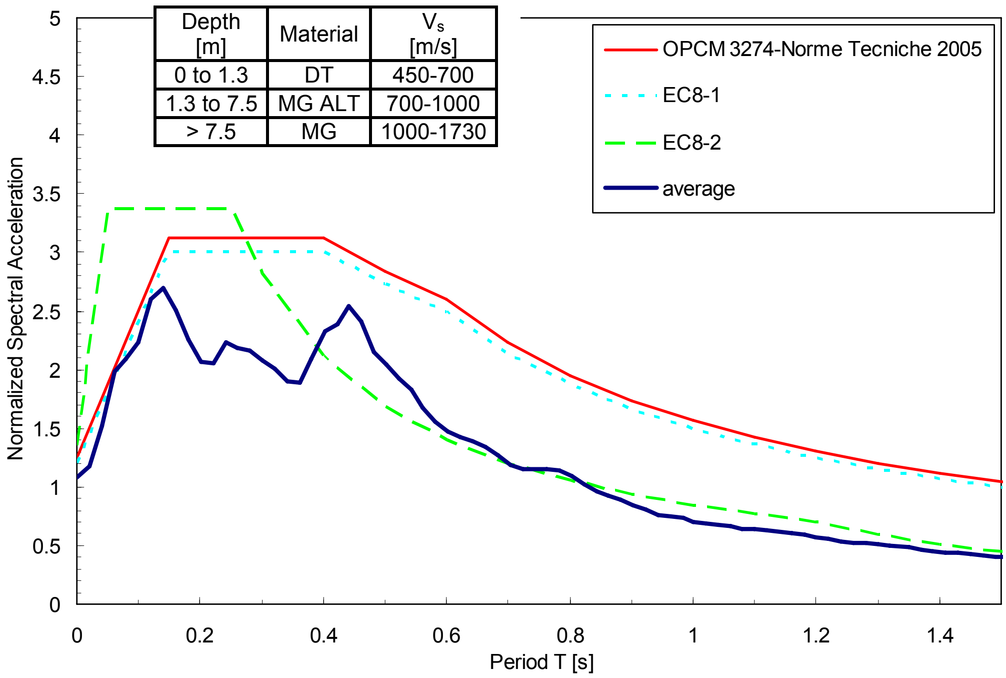

Figure 5a. Computed average elastic response spectra (Type A soil)

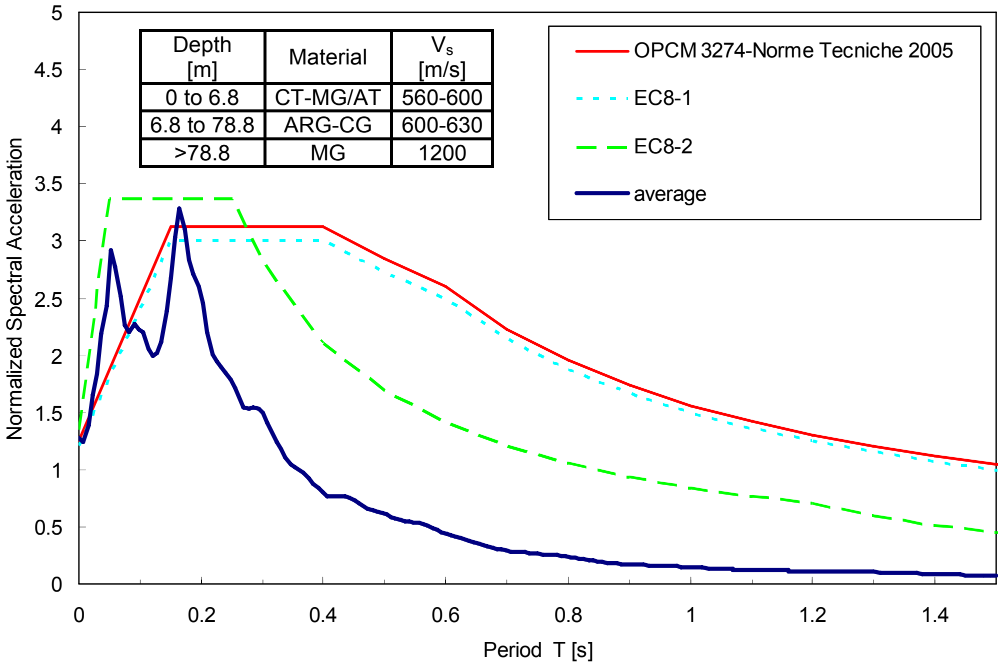

Figure 5b. Computed average elastic response spectra (Type B soil)

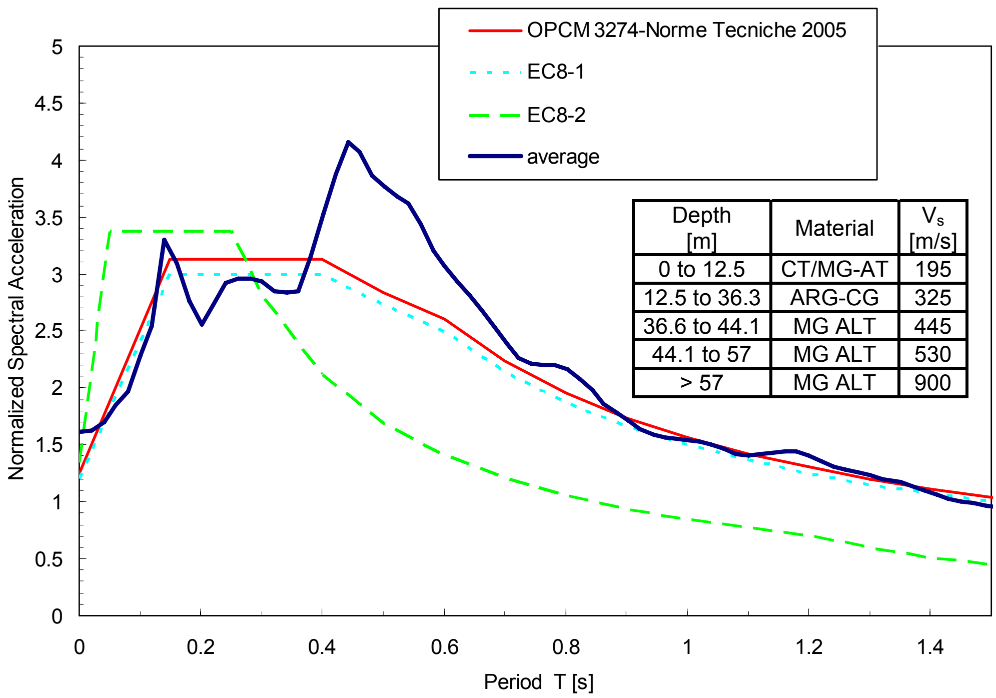

Figure 5c. Computed average elastic response spectra (Type C soil)

Figures 6a, 6b and 6c compare response spectra, accelerations and shear strains, computed at a given site by means of EERA to those obtained with ONDA. The comparison clearly shows that the use of non-linear codes lead to lower values of response spectra, accelerations and shear strains as expected. Input parameters for ONDA have been selected in order to match the G-γ and D-γ curves used for EERA.

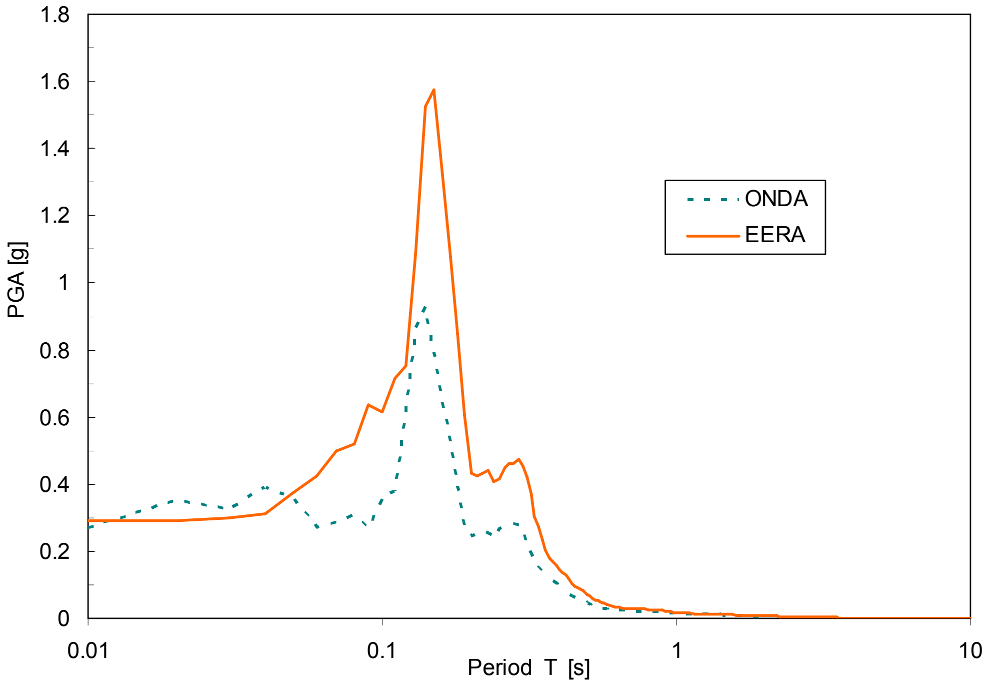

Figure 6a. Response spectra obtained by means of EERA and ONDA

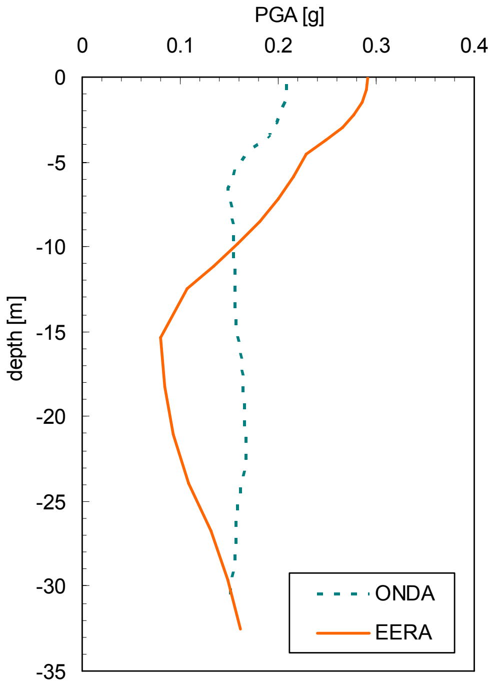

Figure 6b. Peak Ground Acceleration vs. depth

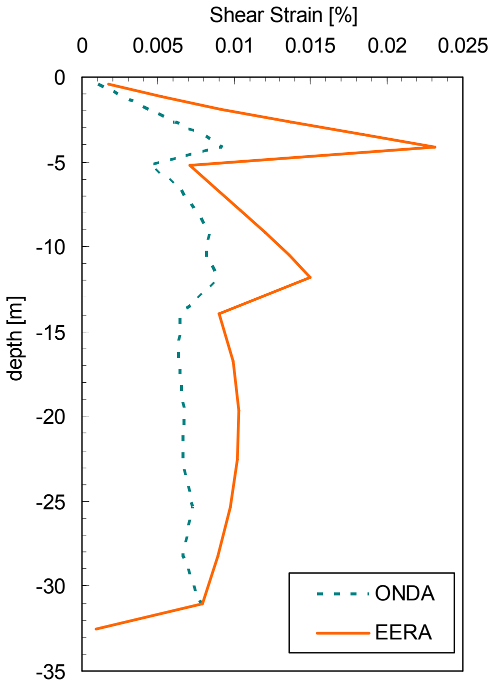

Figure 6c. Shear strain vs. depth

CONCLUSIONS

The following conclusion can be drawn:

- soil type identification can be easily accomplished by means of SH refraction tests or other surface seismic tests (i.e. SASW), at low costs. Use of SPT test results is not straightforward but, through an expert use of experimental data, can lead to the same identification obtained from seismic tests;

- Macrozonation criteria of Table 5 are generally very conservative. Anyway, it is possible to compute (1D analyses) soil amplification even for type A profiles. Therefore it is possible to obtain PGA values (hazard + 1D analysis) greater than those prescribed (see Table 9). This makes the conservativisme introduced by Table 5 casual, that is penalizing in some cases and unconservative in some others.
- Similar comments applies for type B and C soils. In such cases, the prescribed acceleration values have been multiplied by the pertinent soil factor S. The computed PGA's exceed the prescribed values more frequently in B & C soils than in A soils. Anyway, prescriptions of EC8-2 appear more conservative and appropriate for the examined cases.
- The shape of the EC8-2 spectra appears more appropriate for A & B soils, whereas the shape of EC8-1 and OPCM 3274 spectra seems more appropriate for type C soils. It should be remembered that the Magnitude which mainly contribute to the hazard of the considered sites is just below 5.5;
- In conclusion it seems reasonable to adopt the PGA as obtained from hazard studies. In this case the evaluation of possible local amplification of seismic motion becomes mandatory.
- Finally, two additional comments appear necessary: a) in general, linear equivalent analyses overestimate stratigraphic amplification and b) rarely acceleration database, which is used for hazard analysis, refers to type A soils.

AKNOWLEDGEMENTS

The authors would like to thank dr. Ferrini, head of the Seismic Survey of Regione Toscana, for making available the data used in this paper

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