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SEISMIC SOIL CLASSIFICATION AND ELASTIC RESPONSE SPECTRA

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Abstract

The Chile-Maule earthquake of February 27, 2010, with a Mw = 8.8, confirmed the well-known fact that the geotechnical characteristics of soil deposits significantly affect the seismic response at the ground surface. Using the available information consisting of boreholes, shear wave velocity profiles, measurements of the fundamental period via spectral ratio method (H/V) and acceleration records, a methodology to estimate the elastic spectrum of pseudo-acceleration of a site is presented. In this paper, the limitations of V_{S30} (shear wave velocity of the upper 30 m of the ground) as a key parameter to estimate the site effect is discussed. Alternatively, the coupled use of two parameters to estimate the site effect is introduced. These parameters correspond to the equivalent shear wave velocity of the upper 30 m of the ground (shear wave velocity that reproduces the fundamental period of the upper 30 m of the soil deposit) and the predominant period of the site, which is evaluated through ambient noise measurements, applying the H/V spectral ratio technique.

Keywords: Site Effect, Seismic Amplification, Spectra

1. Introduction

The empirical evidence left by large earthquakes clearly shows that the intensity of the motion developed at the ground surface is strongly controlled by the type of soil and thickness of the sediments. A remarkable case of amplification is the one observed during the 1985 Mexico City earthquake of Magnitude 8.1, where the shaking was amplified by a factor of 20, or even more, on sites constituted by deep soil deposits of soft fines materials (Celebi et al. 1987; Singh et al. 1993). On the other hand, rock outcrops and stiff soil deposits constituted by dense granular materials, have shown a significant reduction of the seismic disturbance, which can be directly inferred by the limited damage observed on structures placed on these grounds (Montessus de Ballore, 1911; Watanabe et al. 1960; Borcherdt, 1970; Seed et al. 1988). Accordingly, the local ground conditions have to be included in order to properly evaluate the actual seismic demand that can be induced at a site.

The seismic design of structures can be done using the modal spectral analysis, where the seismic demand is characterized by a design spectrum, which is mainly a function of the ground conditions. Most of the seismic design codes available worldwide take into account the local site conditions in order to define different spectral shapes, which represents an earthquake-resistance criteria. In its early work Seed and co-workers (Seed et al. 1976) analyzed 104 records with peak accelerations greater than 0.05g and proposed normalized spectral forms considering the site-dependent ground motion characteristics. In Fig. 1 the mean spectra categories, defined for different site conditions, are shown. The differences of these spectral shapes are evident, being remarkable for periods greater than 0.5 sec, where soil deposits consisting of soft to medium clays and sands present the higher spectral amplification. Conversely, for periods below 0.4 sec, the higher spectral amplification is observed in deposits constituted by stiff soils. These results were also reproduced by other studies (Mohraz, 1976), and then incorporated in the ATC 1978, using idealized spectral shapes considering three site conditions, as shown in Fig. 2. After the 1985 México City Earthquake, a Soil Profile Type S4 was introduced in order to account for deep soft clay deposits (Seed et al. 1988).

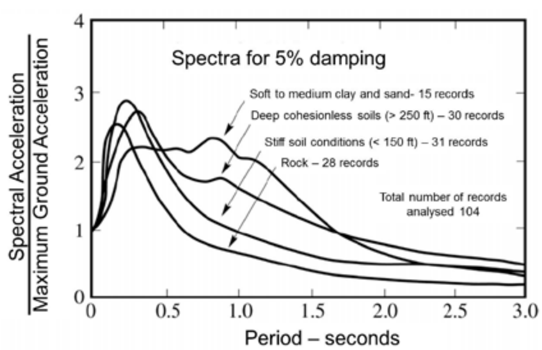

Fig. 1 – Average acceleration spectra for different site conditions (Seed et al. 1976)

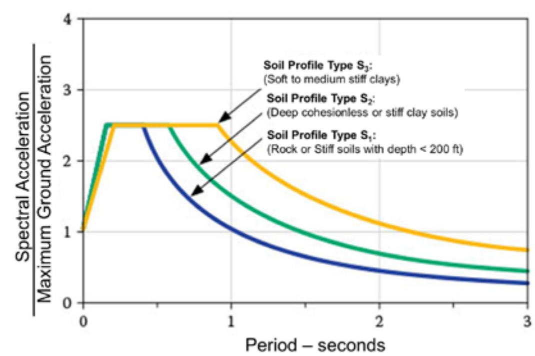

Fig. 2 – Spectral shapes proposed by ATC 3 (1978) for three different soil types codes.

In this paper the spectra established in some modern codes are checked with the spectra obtained from recorded ground motions of recent large earthquakes.

To consider the site effects, the codes have introduced the concept of Site Class or Soil Type for grouping sites with similar geotechnical-geological conditions. Accordingly, to each soil type the same seismic amplification is assigned through a design response spectrum. This means that a different elastic response spectrum is associated to each of these soil types, which are ultimately used for seismic designs. Site Class is determined based on the properties of the soils encountered at the top 30 m of the ground. However, taking into account that deep deposits of soils are definitely impossible of being characterized by any soil property that only considers an upper portion of the ground, in this paper, an alternative methodology is proposed and discussed.

2. Seismic Soil Classifications

Due to the significant seismic damages that have been attributed to local site conditions, most of the countries vulnerable to earthquakes have developed codes that enable an appropriate estimation of the seismic actions in accordance with the soil characteristic. Therefore, the need of a soil classification, from an earthquake engineering point of view, has arisen and the corresponding methodologies for its implementation have been incorporated in the codes. An attempt to classify the geotechnical site conditions unambiguously was introduced by Borcherdt and Glassmoyer (1992) and Borcherdt (1994), by means of the representative shear wave velocity, V_{S30}, of the upper 30 m of the soil profile. The value of V_{S30} is such that reproduces the vertical travel time of the shear wave propagating throughout the top 30 m of the ground. The decision of adopting a depth of 30 m was somehow arbitrary and it is mainly associated with practical reasons, because it corresponds to the typical exploration depth of geotechnical borings. Although in some soil profiles this parameter may lead to incorrect assessments of the site amplification, most of the code provisions for civil structures have adopted it as the main parameter for site classification.

According to the International Building Code (IBC) and ASCE7, a site can be classified from Site Class A to F according to V_{S30} indicated at the top row of Fig. 3. Site class F is used for sites with special soil conditions such as liquefiable soils, highly organic clays, very high plasticity clays and very thick soft clays. Although the main parameter to classify sites is V_{S30}, resistance parameters such as penetration resistance (N-SPT) and undrained shear strength (Su), for the upper 30 m of the ground, are also taken into account.

Analogously, as shown in Fig. 3, the Eurocode 8 (EC8) has adopted five Ground Types, identified as A, B, C, D and E, which are mainly defined by V_{S30}. However, a general description of the stratigraphic profile and resistance parameters such as N-SPT and Su, for the upper 30 m of the ground, are also included for the site identification. In particular, the Ground Type E is introduced, which is defined as a surface alluvium material with V_S < 360 m/s and a thickness less than 20 m, underlain by rock (Vs > 800 m/s). This singular condition is associated with high impedance that amplifies the seismic response. Similar to IBC, the EC8 has defined two additional Ground Types (S1 and S2), which basically consist of soil deposits that require special analyses, for example, fines with high plasticity and high water content, liquefiable soils and sensitive clays.

In the case of Chile, characterized by a long list of large earthquakes throughout its history, the seismic soil classification is established in the code DS-61, which basically defines six Soil Types (identified from A to E) according to the shear wave velocity of the upper 30 m, as described in Fig. 3. Additionally, the Chilean code has grouped as F all those soil deposits considered singulars, for example, liquefiable, organics, fines soils of high plasticity and high sensitive soils, etc.. These soils require a special dynamic analysis.

It can be observed that these three codes use similar values of V_{S30} as the boundaries for each soil type, or site classes, except for the soil type C defined in the Chilean code that was introduced to generate a smoother transition from very dense granular material to medium dense sands and stiff clays.

On the other hand, most of the Japanese provisions for seismic soil classification consider only three site conditions, identified as soil profile types I, II, and III, which correspond to stiff, medium, and soft soil states, respectively. The Soil Type I is described as ground consisting of rock or hard sandy gravel, geologically from the Tertiary Period, or older, whereas the Soil Type III corresponds to alluvium consisting of soft delta deposits, mud, reclaimed land of marsh, muddy sea bottom, etc. The Soil Type II is simply defined as soils that do not classify as Soil Type I or III. Each soil type is numerically characterized by the so-called "critical period of the soil", Tc. The values of Tc are 0.4, 0.6 and 0.8 for soil profiles type I, II, and III, respectively. The critical period of the soil is evaluated using Eq. 1:

$$T_C = 4\sum_{i=1}^{n} \frac{h_i}{V_{Si}} \tag{1}$$

Where, i represents the i-soil layer defined from the ground surface down to the engineering bedrock, and h_i and $V_{s\,i}$ correspond to its respective thickness and shear wave velocity.

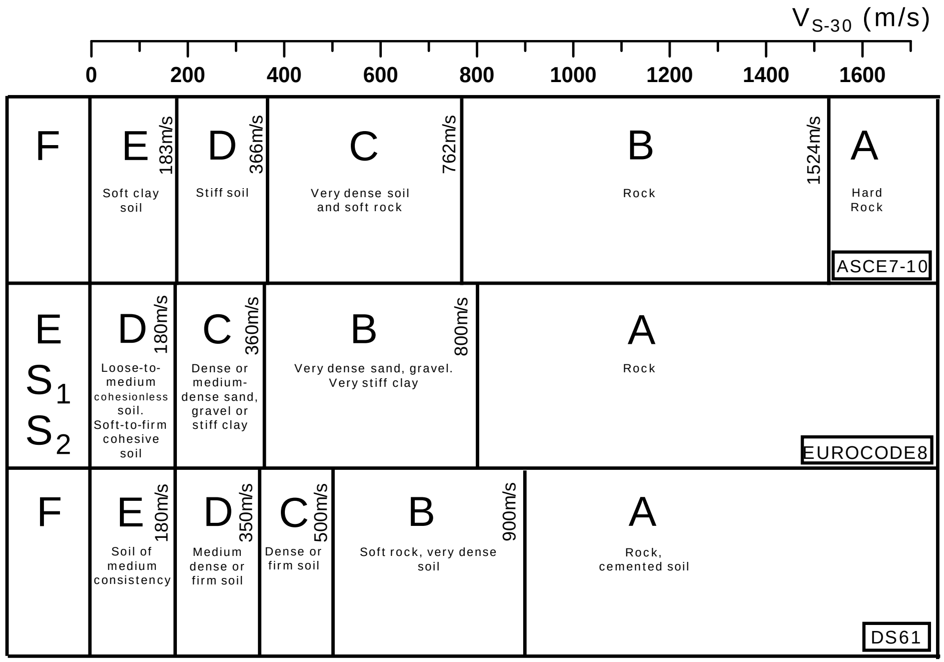

Fig. 3 – Soil classification adopted by ASCE 7, EC-8 and DS61

A soil layer with a shear wave velocity greater than 400 m/s is assumed to be the engineering bedrock. Equivalently, a soil layer with a N-SPT greater than 50 blows/ft is also accepted as engineering bedrock. Taken into account both conditions for the engineering bedrock and the description of soil Type I, it is possible to estimate that sites with values of V_{S30} greater than 400 m/s would correspond to soil Type I. On the other extreme, according to the soil description and critical period (Tc = 0.8 s), soil Type III would correspond to V_{S30} smaller than 150 m/s.

3. Maximum Considered Earthquake to Establish the Response Spectra

In USA, before 1997 the seismic hazard was defined at a uniform 10% probability of exceedance in 50 years. After 1997, NEHRP Provisions defined the maximum considered earthquake (MCE) ground motion with uniform probability of exceedance of 2% in 50 years. This change in the exceedance probability (from 10% to 2%) was applied considering that the use of 10% probability of exceedance in 50 years would not be sufficiently conservative in the central and eastern United States, because the occurrence of earthquakes in these areas has been rather infrequent. However, it has been recognized that what really matters, when dealing with earthquake resistance design, is the probability of structural failure, or structural collapse, because at the end, this is considered the main cause of victims. If it is assumed that there is no uncertainty in the collapse capacity of a structure, the probabilistic uniform-hazard (i.e. 2% of exceedance in 50 years) ground motions would result in a uniform collapse probability. However, it does exist an uncertainty in the collapse capacity of structures, especially due to the lack of actual information related to the as-built material resistance, construction quality and real live loads, amid other issues. Therefore, the probabilistic uniform-hazard ground motions do not provide uniform levels of performance for structures. Consequently, a new risk-targeted probabilistic ground motion was introduced in the ASCE7-10, adjusting the ground motion values in order to obtain an uniform collapse probability of 1% in 50 year. The Risk-Targeted Maximum Considered Earthquake (MCE) ground motion is nominated MCER ground motion.

Notwithstanding, the MCE ground motion that has to be selected in a specific site of USA is the least between the probabilistic and deterministic ground motions. The places that are close to major faults, as the case of those sites located in California, are governed by the deterministic ground motion. It is important to mention that in sites where the deterministic earthquakes control the seismic design, the resulting ground motions are as low as 40% of their probabilistic counterparts (Luco et al. 2009).

On the other hand, in the EC8 is argued that there are many inherent uncertainties associated with size, location, propagation of seismic waves and time of occurrence of future earthquakes. Thus the deterministic approach has been discarded and the seismic hazard of a site has been assessed by employing the probabilistic seismic hazard analysis. The seismic hazard is established in terms of a single parameter: the peak ground acceleration on type A ground (rock). In this scenario, each country has subdivided its territory into different seismic zones, where the seismic hazard is considered constant. In each seismic zone, the PGA on type A ground is associated with a probability of exceedance in 50 years, normally, 10% (equivalent to 475 years of return period).

In the case of Japan, a Level 2-I Earthquake corresponding to events with a periodicity defined as very rare and associated with interplate subduction-type earthquake, of magnitude around 8, has been introduced. The structural seismic design has the main objective of preventing the collapse of buildings and also to avoid harm human lives during the occurrence of this level of seismic action. Additionally, a second type of seismic source is considered; Level 2-II Earthquake, representing inland seismic events of magnitude around 7 that may occur at very short distance by a nearby fault, like the Kobe Earthquake. The recurrence interval of this type of ground motion is estimated to be longer than the Level 2-I, although it is recognized that its evaluation is difficult.

In the case of Chile, the seismic hazard of the provisions was defined at a uniform 10% probability of exceedance in 50 years. However, the recent Maule Earthquake of Mw = 8.8, that occurred in central-south of Chile showed that the Chilean code needed an improvement. Thus a new seismic code, DS61, was introduced considering the main lessons learned from the Maule Earthquake, which may be seen as an appropriate event that can represent the maximum considered earthquake for this region. The previous large earthquakes that have been reported in this area occurred in 1835 (Mw > 8.0), 1751 (Mw > 8.8) and 1657 (Mw ≈ 8.0) (Udias et al, 2012; Lomnitz 2004).

It is interesting to point out that seismic codes around the world are based on probabilistic or deterministic seismic hazard analyses. The EC8 applied a probabilistic analysis approach, whereas the Japanese codes are based on a deterministic seismic hazard analysis. USA represents a singular case, where both approaches have been applied, and the maximum credible earthquake ground motion for a site is selected as the lesser output from these two analyses.

The probabilistic and deterministic methods for the assessment of seismic hazard are usually presented as antagonistic approaches. However, they may certainly complement each other for estimating the ground motions for design, as they have been incorporated in USA. In regions where continental active faults and/or tectonic boundaries generate the largest seismic events expected to occur every 100-200 years, the deterministic seismic hazard analysis is more suitable, providing valuable empirical information about the ground motion effects. It is possible to indicate that the present Chilean code is also based in a combination of both probabilistic and deterministic methods.

4. Elastic Response Spectra

The design response spectra established in the codes are an important part of the earthquake-resistance criteria that represent each code, and amid other considerations (earthquake source, magnitude, epicentral distance, probability of exceedance in a certain life span), these spectra explicitly take into account the geotechnical-geological conditions of the sites.

In the ASCE7 and EC8, a spectrum for a rock site is specified, which is then modified according to the soil types previously defined in Fig. 3. In the ASCE7-10, amplification factors are established for low and long periods, Fa and Fv respectively, which are function of both soil type and intensity of rock motion. The spectral shape adopted in these two codes follow the Newmark type. In Fig. 4 the resulting spectra for the highest considered ground motion and different soil types are shown. It is interesting to observe that for the worst seismic scenario considered and similar soil conditions, these spectra are rather comparable, although the EC8 has spectra with higher amplification for the low period range.

The spectra adopted by the codes of Japan and Chile, associated with the worst seismic subduction scenario, are shown in Fig. 5. The Japanese spectra follow more or less the Newmark´s spectral shape, while the Chilean spectra are different, with a particular shape that was originally proposed by Arias (1989), based on recorded ground motions.

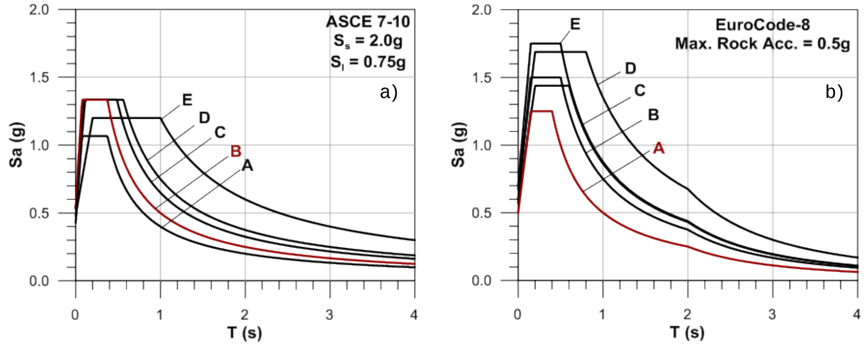

Fig. 4 – Response spectra for the highest respective ground motion. a) ASCE 7 and b) EC-8

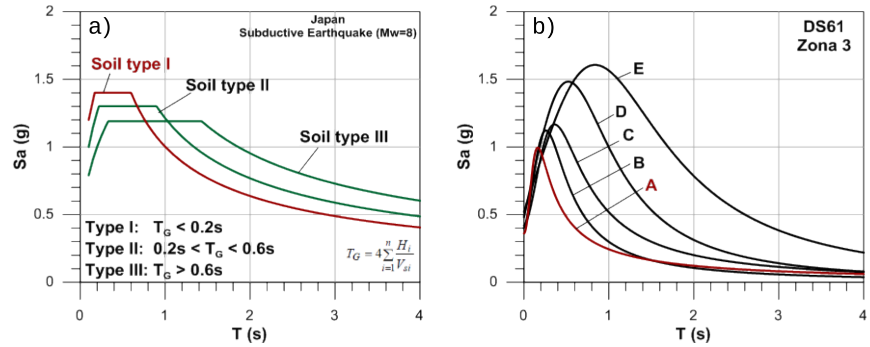

Fig. 5 – Response spectra for the worst subduction ground motion. a) Japanese and b) Chilean codes

A comparison between the response spectra adopted by each of these codes is shown in Fig. 6. For rock and cemented soil sites (Fig. 6a), the Japanese spectrum is significantly higher than the others in all the range of periods considered. On the other hand, the spectrum adopted by the Chilean code is well below the others. However, there is a good agreement between the ASCE7 and EC8. For soil deposits with V_{S30} in the range of 180 – 350 m/s (Fig. 6b), the spectra of ASCE7, EC8 and DS61 are more similar, but the Japanese spectrum shows larger values of Sa for periods above 0.8 s. For soil deposits with V_{S30} smaller than 150 – 180 m/s (Fig. 6d), there is an important difference amid the spectra. This suggests that for soil deposits constituted by soft materials, the available information regarding their seismic response is less conclusive.

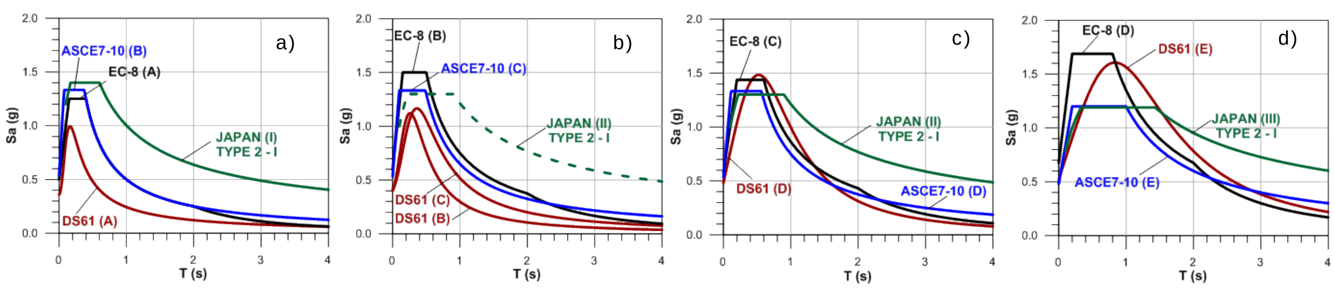

Fig. 6 – Response spectra according to soil conditions in different codes.

5. Response Spectra Obtained from Strong Motion Records

The Maule Earthquake of Magnitude Mw = 8.8, hit the Central-South region of Chile, on February 27, 2010. This earthquake corresponds to a thrust-faulting type event associated with the subduction seismic environment caused by the collision between the Nazca and South American tectonic plates. The rupture zone responsible of this quake covered a rectangular area of approximately 550 km by 170 km, with an average depth of 35 km. It is important to mention that this large rupture zone implies that the hypocenter consists of an approximately planar area from where the seismic energy is emanated according to the evolution of the rupture itself. A total number of 36 seismic stations located in the most affected area recorded the acceleration time histories on rock outcrops and soil deposits of different geotechnical characteristics. The maximum PGA recorded on a rock outcrop was 0.32g in Santa Lucía Hill in Santiago, whereas the maximum PGA recorded on a soil deposit reached a value 0.94g in Angol city, located close to the south end of the rupture zone. The second highest value of PGA recorded on soil was 0.78g, in the city of Melipilla, located close to the north end of the rupture zone.

From the analyses of the elastic response spectra (5% damping) computed from the acceleration records of the Maule Earthquake, it is apparent that several of them resulted significantly higher, within a certain range of periods, than the spectra proposed in the Chilean code. For example, in Fig. 7a are shown three cases (Angol, Constitución and Melipilla) where, regardless the soil type, the associated spectra clearly exceeded all the spectra of the code.

Another large earthquake, of Magnitude Mw =8.3, took place in Central Chile on September 16th of 2015, along the subduction zone interface; the Illapel Earthquake. The event was recorded by more than 30 seismic stations, from which 19 records presented a PGA greater than 0.1g. The maximum recorded PGA was 0.83g (E-W) and 0.71g (N-S). In Fig. 7b are shown the two records with spectra that exceeded the spectra of the code, regardless the soil type.

In Fig. 8 are shown the spectra obtained from the ground motions recorded in the seismic stations of Angol and Concepción, which are resting on Soil Type D, according to DS61. For comparison, the spectra associated with similar soil conditions and worst seismic scenario established by the Japanese code, UC8 and ASCE7 are also plotted. It can be seen that these seismic records generate spectra that definitely are underestimated by the Chilean code and also by the others three considered codes.

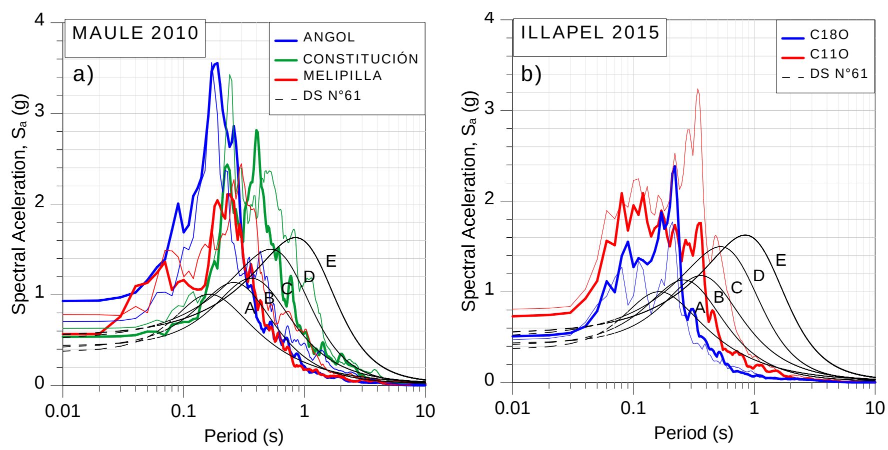

Fig. 7 – Response spectra obtained from Chilean Earthquakes a) Maule and b) Illapel

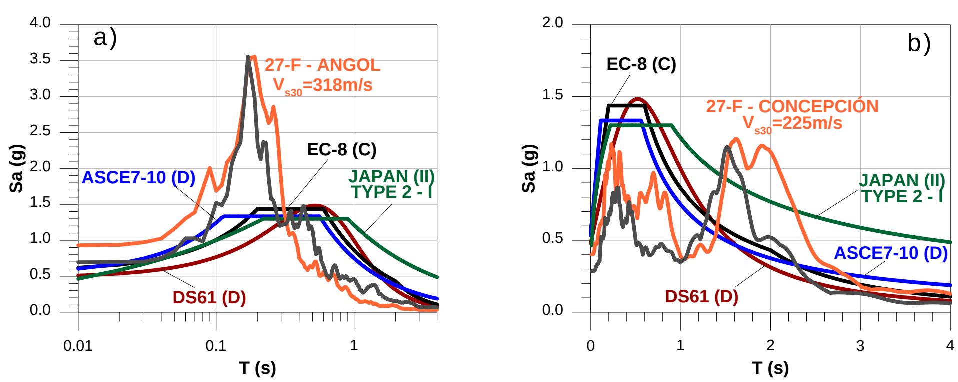

Fig. 8 – Response spectra obtained from records of Maule Earthquake. Soil Type D according to DS61

It can be argued that the Maule Earthquake of Mw = 8.8, represents a seismic event that is larger than the maximum considered earthquakes established by these codes. In terms of probability or statistics it could be correct, however, from a practical point of view this fact has important aftermaths. If the design spectra significantly underestimate the actual seismic forces, an important level of damage should be expected. However, although some structures were severely damaged, the overall seismic behavior of structures during the Maule and Illapel Earthquakes was definitely successful. This fact may suggest that the entire process of: design - load combinations - computation - construction, as a whole engineering package, works properly for any practical purpose. However, from the theoretical point of view of the acting seismic forces, there is no doubt about the disagreement between, the so-called instrumental spectra (obtained from acceleration records) and the official spectra stablished in the Code. This situation has been observed in all the recent large earthquakes. For example, during the Northridge Earthquake of Magnitude Mw = 6.7, that occurred in Los Angeles, USA, on January 17, 1994, the seismic stations JGB and NWH recorded acceleration time histories of the main shock, which response spectra are plotted in Fig. 9. It can be observed how high are the instrumental spectra in comparison with the ones established in the ASCE7 code and the others that have been included as a reference.

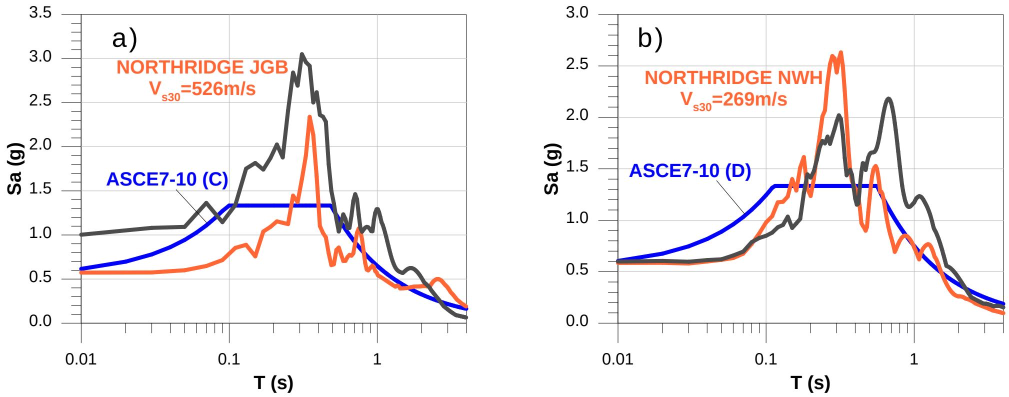

Fig. 9 – Response spectra from records of Northridge Earthquake. Soil Type C - ASCE7

a) JGB station and b) NWH station

On March 11, 2011, the Tohoku Earthquake of Magnitude Mw = 9.0 struck off Japan's northeastern shore. This mega earthquake was recorded by a dense array of accelerometers, one of which recorded a maximum PGA of 2.75g in N-S direction and 1.29g in the E-W direction. In particular, the response spectra computed from the TCGH16 and FKSH20 records are plotted in Fig. 10, from where it is evident that the instrumental spectra are significantly higher than the spectra proposed in the codes.

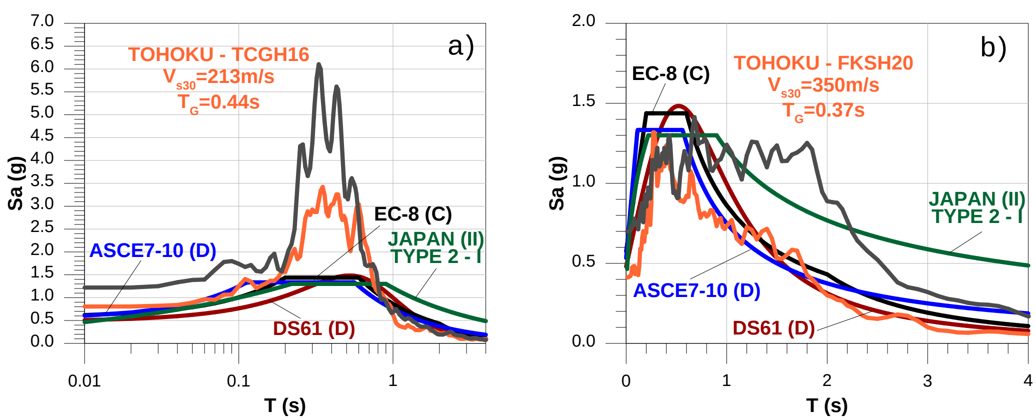

Fig. 10 – Response spectra obtained from records of Tohoku Earthquake. Soil Type II according to JC

a) TCGH16 station and b) FKSH20 station

Recently, on April 16, 2016, the Muisne Earthquake of Magnitude Mw = 7.8 hit the coast of Ecuador. The recorded maximum peak horizontal and vertical accelerations were 1.41g and 0.74g, respectively. The instrumental response spectra of the two acceleration histories recorded by the seismic stations APED and APDN are plotted in Fig. 11. In this figure, the spectra indicated by the Ecuadorian code for Seismic Zone VI (coast) are also presented. It is evident that the design spectra proposed in the Ecuadorian code were significantly exceeded by the ground motion.

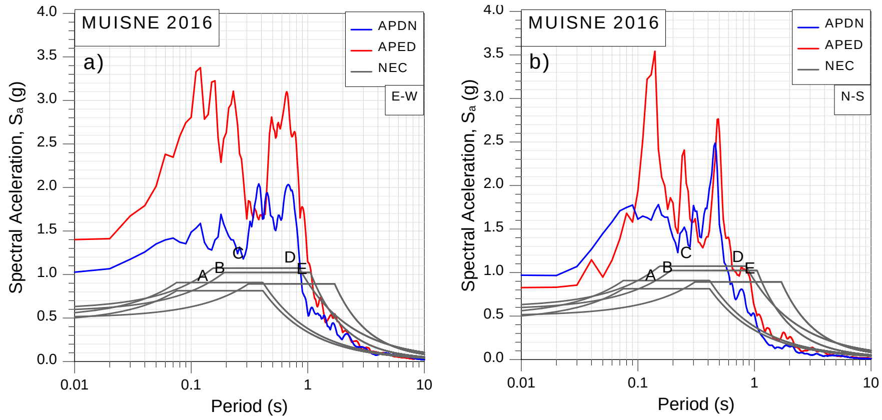

Fig. 11 – Response spectra obtained from Muisne Earthquake, Ecuador

It is important to mention that in all the presented cases, where the spectra of the codes were significantly exceeded by the spectra computed from the actual recorded ground motions, there are almost no damages that can be directly attributed to acting seismic loadings larger than the design loading. In general, the majority of the damages can be explained by incorrect designs and/or deficiencies in both materials and construction. Therefore, it is possible to indicate that most likely the spectra that are incorporated in the codes correctly satisfy the needs of a good engineering practice. Nonetheless, new insights should be introduced in this aspect of the seismic design in order to have a reasonable match between measured spectra and those spectra provided by seismic provisions. The first step for addressing this issue is related to the PGA, which should be a function of the soil conditions. In addition it seems necessary to re-define the amplification factor from PGA to the plateau of the pseudo-acceleration, Sa. Values well higher than 2.5 are usually encountered in the instrumental spectra.

6. Proposed Seismic Soil Classification

The seismic soil classification attempts to characterize a site in terms of its seismic response at the surface. Accordingly, the expected spectrum of a site, for a given level of seismic hazard (MCE ground motion), is directly associated with the seismic soil classification. In this context, the seismic soil classification is important because in practice it determines the acting seismic forces under which the structures are designed.

Most of the codes have developed a seismic soil classification mainly based on the shear wave velocity of the upper 30 m of the ground, as the case of ASCE7, EC8, and DS61, whereas other provisions, such as the Japanese codes, use the so-called critical period of the ground. In the first case, it is evident that any soil property that represents only the upper 30 m of the ground, is definitely unable to capture the complex seismic response of deep soil deposits constituted by several layers of different geomechanical properties. Nevertheless, considering that the parameter V_{S30} is related with the stiffness (or flexibility) of the top 30 m of the ground, where the amplification is more pronounced, it is expected that V_{S30} can have some influence on the seismic response at the ground surface.

On the other hand, the predominant period, Tp, of a soil deposit definitely provides important insights about the seismic response. Low values suggest rigid soil deposits, with important amplification, whereas large values of predominant period would be associated with flexible soil deposits, with a medium level of amplification.

Taking into account that these two parameters provide relevant information about the seismic behavior for a site, a seismic soil classification that includes basically these parameters is proposed. The current shear wave velocity of the upper 30 m, V_{S30}, corresponds to the equivalent shear wave velocity that reproduces the same propagation time in the upper 30 m. This implies that the sequence of different soil layers does not affect the value of V_{S30}. However, the seismic response at the ground surface can be strongly affected by the sequence of the soil layers. For example, in Fig. 12 the transfer function base-surface of two stratigraphic profiles, with identical V_{S30}, are presented, both including a 10 m-thick layer with Vs = 150 m/s. In the left profile this layer is on the surface, while in the right profile it is located at a depth of 20 m. It can be seen that when the soft layer (Vs =150 m/s) is not located at the surface, it practically acts as a seismic isolator, while at the surface it amplifies the response due to the large impedance ratio (square root of the ratio of shear wave velocities of two consecutive layers).

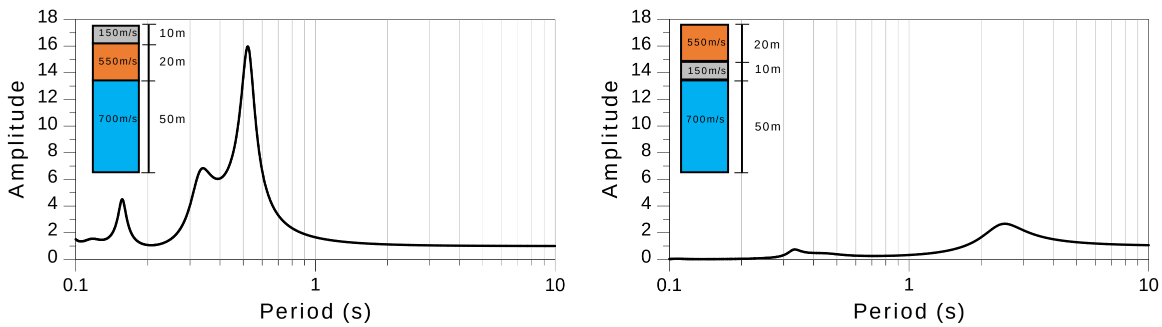

Fig. 12 – Transfer functions of layered grounds

Thus, it is proposed to incorporate the stratigraphic sequence of the upper 30 m by means of a modified V_{S30}, which corresponds to an equivalent shear wave velocity that would reproduce the same fundamental period of vibration of the top 30 m of real stratified ground. Knowing the stratigraphic sequence of Vs of the top 30 m of a site, the theoretical elastic fundamental period, $T_{f\text{-}30}$, can be numerically obtained, using for example an equivalent linear analysis. Then, the evaluation of the equivalent shear wave velocity, according to equation (1), is: $V_{S30\text{-}E} = 120/T_{f\text{-}30}$ (in m/s). The capability of $V_{S30\text{-}E}$ for assessing the seismic response of layered grounds has been evaluated by means of 1D analysis. Fig. 13 shows three different stratigraphic profiles, that have the same $V_{S\text{-}30}$ = 300 m/s. In each case five ground motions records obtained at rock outcrops during the Maule Earthquake were scaled to 0.2g and applied at the bottom of the models. The response spectra obtained at the ground surface are shown in Fig. 13; where the average values are shown in red. It can be observed that even though the three sites have the same value of $V_{S\text{-}30}$, the seismic responses on the surface are significantly different.

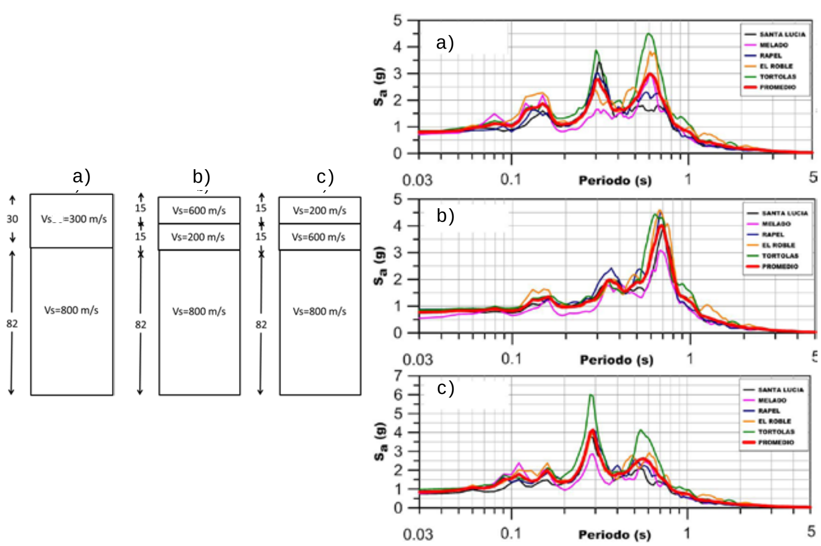

Fig. 13 – Layered soil profiles with same V_{S30} and response spectra at ground surface

The proposed parameter $V_{S30\text{-}E}$ has been computed in each case, and used to generate new soil models, which were subjected to the same five ground motions described before. The response spectra of the original layered ground and the new soil model with $V_{S30\text{-}E}$ are presented in Fig. 14. It is observed that the characterization of the top 30 m with the proposed $V_{S30\text{-}E}$, allows to adequately reproducing the seismic response on the surface.

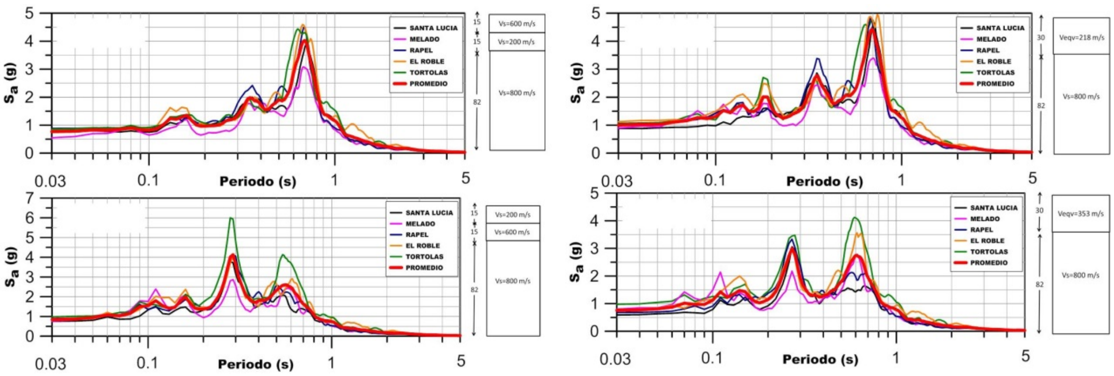

Fig. 14 – Response spectra of layered ground (left) and using $V_{S30\text{-}E}$ (right)

On the other hand, the second parameter to be used in the proposed seismic soil classification is the predominant period, which can be easily obtained using the H/V spectral ratio (Nakamura 1989). When this method is used on rigid soil deposits, as dense gravely materials, the evaluated H/V spectral ratios do not show any peak. Hence, soil type A and B (DS61) are recognized by flat H/V spectral ratios. When the H/V spectral ratios show a peak, the predominant period, T_N, of the site can be accomplished. This period should be consistent with the site classification obtained according to $V_{S30\text{-}E}$. If T_N is larger than what is expected, it means that the ground conditions below 30 m make the site more flexible, and therefore, its classification should be modified and corrected in that direction, degrading the soil type in one step. Accordingly, for soils type A and B, the H/V spectral ratio has to be flat, or with a weak peak indicating a value of T_N smaller than 0.3 s. For soil type C, the H/V spectral ratios should result with a value of T_N smaller than 0.4 s. For soil type D, the H/V spectral ratios should result with a value of T_N smaller than 0.8 s. Soil type E does not need to satisfy a condition about its expected predominant period. The proposed limited values of T_N (0.3 s for soil types A and B, 0.4 s for soil type C and 0.8 s for soil type D) resulted from the analyses of available information of the Maule and Tohoku Earthquakes, but it is a matter of discussion. The important issue is that the seismic soil classification based on Vs of the top 30 m, has to be reinforced with the inclusion of the predominant period of the site.

7. Concluding Remarks

The spectra established in the ASCE76, DS61, Ecuadorian and Japanese codes, for the worst seismic scenario, are in many cases significantly exceeded by the spectra computed from recorded ground motions. Nevertheless, the observed seismic damages are not directed attributed to this fact. Indeed, in the Chilean case, during the Maule and Illapel Earthquakes the seismic structural performance was excellent, although several acceleration records generated spectra that were well higher than the ones specified in the DS61. This empirical evidence implies that further research is needed in order to clarify which are the actual seismic forces that act on the structures.

A seismic soil classification is proposed that takes into account two ground conditions: the equivalent shear wave velocity, $V_{S30\text{-}E}$, of the upper 30 m of the ground and the predominant period of the soil deposit. The $V_{S30\text{-}E}$ attempts to incorporate the effect of the sequence of soil layers by means of reproducing the dynamic lateral stiffness of the upper 30 m of the ground. The predominant period of the soil deposit is easily estimated using the Nakamura´s method (H/V spectral ratio), except in the case of soil type A and B (DS61) where the H/V spectral ratios do not show a clear peak. Indeed when this occurs, it means that the site should classifies as A or B.

The presented soil classification uses $V_{S30\text{-}E}$, but additionally, the estimated predominant period has to corroborate the soil type. If not, the initial soil type is degraded in one step.

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9. References

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