# SEISMICITY AND DESIGN CODES IN CHILE: CHARACTERISTIC FEATURES AND A COMPARISON WITH SOME OF THE PROVISIONS OF THE ROMANIAN SEISMIC CODE

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**ABSTRACT**

A brief account on the characteristics of the seismic region and events in Chile reveals interesting indices in understanding the present day Chilean seismic design code. The present article points out some of the most important provision in the Chilean code that could have led to the relatively small number of casualties at the seismic event of February 27th 2010. By comparing the Chilean code to the Romanian one, the goal is to underline the differences and the similarities regarding both the conceptual and formal aspects. Observations are pointed out by means of comparative graphs of significant parameters. Based on statistics of recorded damage published after the earthquake, some comments are made about the importance of the quality of seismic codes and of the effectiveness of their enforcement.

*Key-words*: Chile, seismic code, earthquake, behavior / reduction factor, seismic coefficient, design spectra

**REZUMAT**

Un scurt istoric, precum şi caracteristicile zonei seismice şi cutremurelor de pământ din Chile relevă indicii interesante în înțelegerea actualului cod de proiectare seismică chilian. Articolul de față evidențiază unele dintre cele mai importante prescripții din codul chilian care ar fi putut conduce la numărul relative redus de victime la evenimentul seismic din 27 februarie 2010. Prin compararea codului chilian cu cel românesc, se urmăreşte evidențierea deosebirilor şi asemănărilor, din punct de vedere conceptual, dar şi formal, dintre cele două coduri. Observațiile efectuate sunt puse în evidență pe baza graficelor comparative ale unor parametri semnificativi. Pe baza statisticilor privind avarierile înregistrate la cutremurul menționat, sunt efectuate unele comentarii privind importanța calității codurilor de proiectare seismică şi a eficacității aplicării acestora.

*Cuvinte-cheie*: Chile, cod seismic, cutremur, factor de comportare / reducere, coeficient seismic, spectre de proiectare

## 1. SEISMICITY OF CHILE

Chile represents a point of special interest to the earthquake engineering, as on the territory of this country there are regions with one of the highest degrees of seismicity in the world.

The main seismic source in Chile is the Nazca subduction zone. In this area, Nazca tectonic plate subducts with a relatively high velocity (80 mm / year) to the South America tectonic plate (Fig. 1.). As a consequence of this collision, the models of the seismic source which affects Chile can be described as: subduction interface and intra-slab, crustal faults and background seismicity [1]. All these lead to shallow crustal earthquakes, typical for this area.

The earthquake that led to the greatest number of casualties was the one on January 24th 1939, at local hour 23:32, with $M_w$=7.8, $M_L$=8.3 and the epicenter at Chillán. The earthquake was a shallow crustal one (60 km deep) and caused the loss of 30.000 human lives. Approximately 3500 buildings

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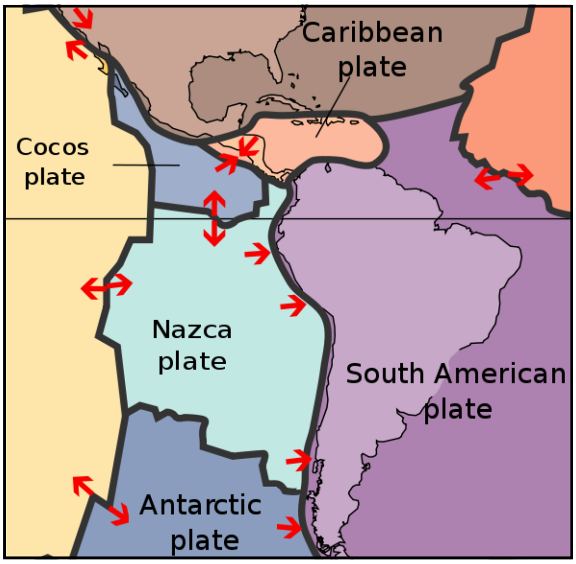


**Fig. 1.** Pacific Tectonic plates from South America and Pacific Ocean [2]


**Fig. 2.** The epicenter of the February 27th 2010 Chile earthquake [5]

collapsed at the initial shock; after the aftershocks, 95 % of the city was destroyed. The electrical power went down, the drinkable water supplies were seriously damaged and most of the representative buildings in the town were destroyed [3].

The earthquake with the highest magnitude in Chile is, at the same time, the strongest ever recorded in the world, with a moment magnitude $M_w$=9.5. It occurred on May 22nd, 1960, at the local hour 14:11, with the epicenter at Cańete. It affected mainly Valdivia city, hence its name: "The Great Valdivia Earthquake". As a consequence of the initial shock, tsunami waves formed and affected the south of Chile, the Hawaii Archipelago, Japan, the Philippine Islands and the west coast of the U. S. The damages could not be estimated accurately because of the large surface affected by the earthquake. Different sources mention casualties of 2231, 3000 or 5700; as regarding the economic losses, sums between 400 and 800 million USD were estimated. In Valdivia, 40 % of the housings were destroyed. The most affected were concrete structures, one of the main causes of the damage being the lack of a seismic design. The traditional wood dwellings behaved better; most of them were still standing, although some of them were not safe enough for occupancy [4].

From 1973 until nowadays, in Chile occurred 13 seismic events with magnitude over 7 (USGS). Among these, one of the most important are the one in Santiago, on March 3rd 1985, with a magnitude $M_w$=8.0, and 177 casualties, and the one on July 30th 1995, in Antofagasta, with only 3 lost human lives.

Given the numerous seismic events and due to the development of earthquake engineering and seismic design, efforts have been made in Chile to implement advanced design codes. After the earthquake in 1960, the Chilean government financed the research work for a seismic design code for new buildings and, in 1993, according to *AIR Worldwide*[6], all the Chilean codes were reviewed to include the newest methods and techniques available. Starting with year 2003, the local authorities have gradually introduced, with the assistance of the Association of Chilean Structural Engineers, a regulation by which seismic and structural computations were to be verified by an independent professional, authorized by the Construction Ministry.

These measures could represent one of the main reasons for which the number of casualties was relatively low (about 300 according to some reviewed estimations [7]) at the magnitude $M_w$=8.8 seismic event of February 27th 2010. The initial shock took place at the local hour 03:34, the epicenter being located at about 325 km south-west from the capital, Santiago de Chile, and at approximately 115 km north from Concepción, the second largest city in the country, having over 200,000 habitants. The latter was the most affected one (Fig. 2).

The seismic motion of the initial shock lasted for approximately 3 minutes. During the month that followed, 257 aftershocks (until March 20th), 18 of which had a moment magnitude greater than 6. The epicenters of these aftershocks extend on a very large area, along the rupture surface (Fig. 3).

The depth of the hypocenter of the initial shock was estimated at 35 km. Initially, a tsunami warning was issued for Chile and Peru, which then was extended for the entire Pacific area, with the exception of the west coast of the USA and of Alaska. The extension of the tsunami waves was, fortunately, smaller than expected. In Concepción city, strongly damaged buildings and fires were reported and the access of rescue teams was partially hampered due to damaged infrastructure [9].

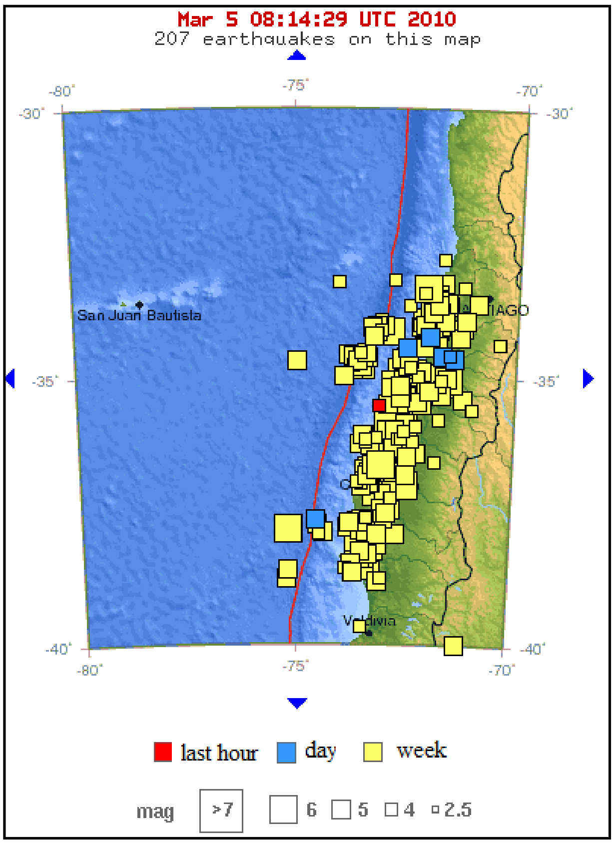


**Fig. 3.** The epicenters of the aftershocks following of the February 27th 2010 Chile earthquake [8]

## 2. THE PRESENT CHILEAN SEISMIC DESIGN CODE

### *2.1. General*

The present seismic design code in Chile (with the indicative NCh433.Of96) [10] was implemented in 1996 and has not been updated since then. For the industrial buildings, as well as for base isolated systems, separate codes have been enforced in 2003. As a consequence of the effects of the last recorded earthquake, the one on February 27th 2010, nowadays research is being carried out in order to update the NCh433.Of96 code [11].

According to the Chilean code, the territory of the country is divided into three seismic zones (1, 2 and 3), which specify the peak ground accelerations, $A_0$ (Fig. 4 and Table 1).

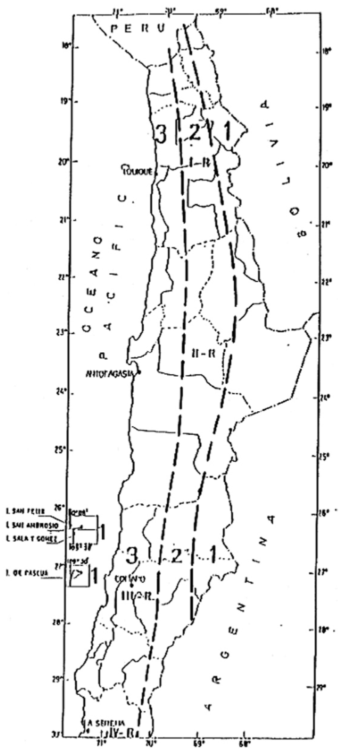


**Fig. 4.** The NCh433.Of96 Chilean code: seismic zonation map of the PGA values for the North region of the country

**Table 1.**

| Seismic zone | $A_0$ |
|---|---|
| 1 (at the border with Argentina)<br>2<br>3 (shores) | 0.20 g<br>0.30 g<br>0.40 g |

The code provides a classification of the soil types into four categories (Table 3), as well as of the buildings, according to their importance, also into four categories (Table 2).

As regarding the seismic design procedures, the code specifies two methods: the static method (corresponding to the method of the static equivalent lateral forces in the Romanian code) and the spectral modal analysis method.

**Table 2.**

| The importance class of the building | $I$ |
|---|---|
| A (high importance)<br>B<br>C<br>D (low importance) | 1.2<br>1.2<br>1.0<br>0.6 |

## *2.2. The static method*

The static method is applicable to structures which satisfy the following conditions:

- are built on soil type C (unsaturated sands and gravels, cohesive soils with the undrained shear, $s_u$, between 0.025 and 0.10 MPa) or D (cohesive soils with $s_u \geq 0.025$ MPa) and are located in the seismic zone I (with the PGA = = 0.20g – Table 1);
- do not exceed 5 stories or 20 m height;
- for the structures having 6 to 15 stories, the application of the method is permitted provided that: (1) the ratios between the total building height and the modal vibration periods with the highest translational equivalent mass in "x" and "y" directions are at least equal to 40 m / s and (2) the distribution of the horizontal seismic forces of the static method is such that shears and overturning moments at each level shall not differ with more than 10 % with respect to those obtained through a spectral modal analysis with the same base shear force [11].

Additionally, the applicability of the method is limited to the zones with $A_0$=0.20g.

The base shear force, $Q_o$, is determined according to the formula:

$$Q_0 = CIP \tag{2.1}$$

where $C$ = seismic coefficient; $I$ = coefficient taking into account the importance class of the building (Table 2); $P$ = the total weight of the superstructure.

The seismic coefficient, $C$, is given by:

$$C = \frac{2.75 \cdot A_0}{g \cdot R} \cdot \left( \frac{T'}{T^*} \right)^n \tag{2.2}$$

where $n$, $T'$ = parameters depending on the foundation soil (Table 3); $A_0$ = peak ground acceleration; $R$ = seismic response reduction factor; $T^*$ = the vibration period of the mode with the highest translational equivalent mass in the direction of analysis.

**Table 3.**

| Foundation soil | $S$ | $T_0$ [s] | $T'$ [s] | $n$ | $p$ |
|---|---|---|---|---|---|
| I | 0.90 | 0.15 | 0.20 | 1.00 | 2.0 |
| II | 1.00 | 0.30 | 0.35 | 1.33 | 1.5 |
| III | 1.20 | 0.75 | 0.85 | 1.80 | 1.0 |
| IV | 1.30 | 1.20 | 1.35 | 1.80 | 1.0 |

The value of the seismic coefficient is limited to a minimum value of $A_0 / 6g$ and to maximum values according to Table 4.

In Table 4, $S$ is a coefficient depending on the soil type.

**Table 4.**

| $R$ | $C_{max}$ |
|---|---|
| 2.0 | 0.90 $S \cdot A_0$ / g |
| 3.0 | 0.60 $S \cdot A_0$ / g |
| 4.0 | 0.55 $S \cdot A_0$ / g |
| 5.5 | 0.40 $S \cdot A_0$ / g |
| 6.0 | 0.35 $S \cdot A_0$ / g |
| 7.0 | 0.35 $S \cdot A_0$ / g |

The distribution of the seismic forces over the height of the structure is proportional with the mass and the height of each floor with respect to the base of the building, as follows:

$$F_k = \frac{A_k \cdot P_k}{\sum_{j-1}^{N} A_j \cdot P_j} \cdot Q_0 \tag{2.3}$$

where:

$$A_k = \sqrt{1 - \frac{Z_{k-1}}{H}} - \sqrt{1 - \frac{Z_k}{H}} \tag{2.4}$$

and $Z_k$ = the distance from floor $k$ to the base of the building; $H$ = the height of the structure.

According to NCh433.Of96, the structures with two or more levels and with no rigid diaphragm at the uppermost storey may be computed as if there was such a diaphragm. Nevertheless, in order to design a floor which cannot play the role of a rigid diaphragm, each element resisting seismic forces must be designed to resist a horizontal acceleration of $1.20F_N g / P_N$ times the corresponding mass.

The results given by the static method, determined by applying independent seismic forces on two directions, must be combined with those obtained from the accidental torsion analysis. Thus, at each level, torsional moments, computed as the product between the equivalent static lateral load at the given storey and the accidental eccentricity, should be applied:

$\pm 0.10 b_{ky} Z_k / H$ for the seismic action in $Y$ direction

$\pm 0.10 b_{kx} Z_k / H$ or the seismic action in $X$ direction

where $b_{kx}$ and $b_{ky}$ are the largest dimensions of the structure, in the $X$ and $Y$ directions, respectively, at story level $k$.

This prescription is similar to that in the Romanian code P 100-1 / 2006, with a difference in the computation of the accidental eccentricity. In the Romanian code, the accidental eccentricity is computed with the relation:

$$e_{1i} = \pm 0.05 L_i \qquad (2.5)$$

where $e_{1i}$ = the additional eccentricity of the mass at level $i$ with respect to the position of the gravity center, applied on the same direction at all levels; $L_i$ = the dimension of the floor perpendicular to the direction of the seismic action.

### *2.3. The spectral modal analysis method*

The **spectral modal analysis method** can be applied to structures with regular vibration modes and critical damping ratios of approximately 5 %.

In this method, the design spectrum is determined with:

$$S_a = \frac{I \cdot A_0 \cdot \alpha}{R^*} \qquad (2.6)$$

where $I$ and $A_0$ have the values given in Tables 1 and 2 and α (fig. 5) is an amplification factor determined for each vibration mode, $n$, with the formula:

$$\alpha = \frac{1 + 4.5\left(\frac{T_n}{T_0}\right)^p}{1 + \left(\frac{T_n}{T_0}\right)^3} \qquad (2.7)$$

in which $T_n$ = the vibration period for the $n$-$th$ vibration mode; $T_0$, $p$ = parameters depending on the type of foundation soil (Table 3).

The reduction factor $R^*$ (Fig. 6) is computed as

$$R^* = 1 + \frac{T^*}{0.1 \cdot T_0 + \frac{T^*}{R_0}} \qquad (2.8)$$

where $T^*$ = the vibration period of the mode with the highest translational equivalent mass in the direction of analysis; $R_0$ = the global reduction factor of the structure, given in tables by the Chilean code.

For structures with reinforced concrete walls or with walls and frames, NCh433.Of96 allows for a simplified computation of the reduction factor, according to the expression below:

$$R^* = 1 + \frac{N \cdot R_0}{4 \cdot T_0 \cdot R_0 + N} \qquad (2.9)$$

where $N$ = number of stories of the structure.

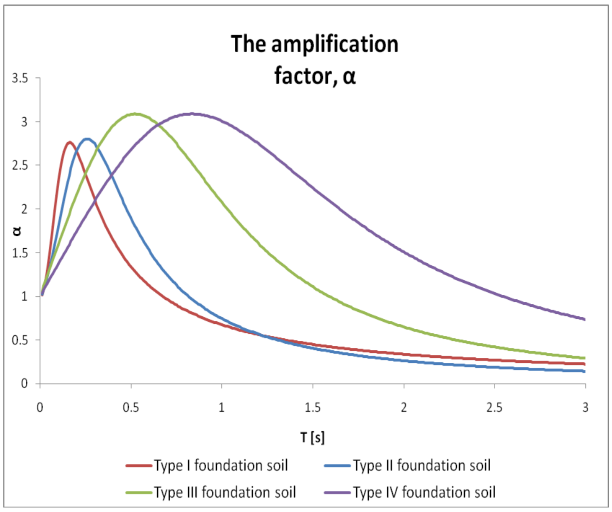


**Fig. 5.** The amplification factor, α, for the four soil types in the Chilean code

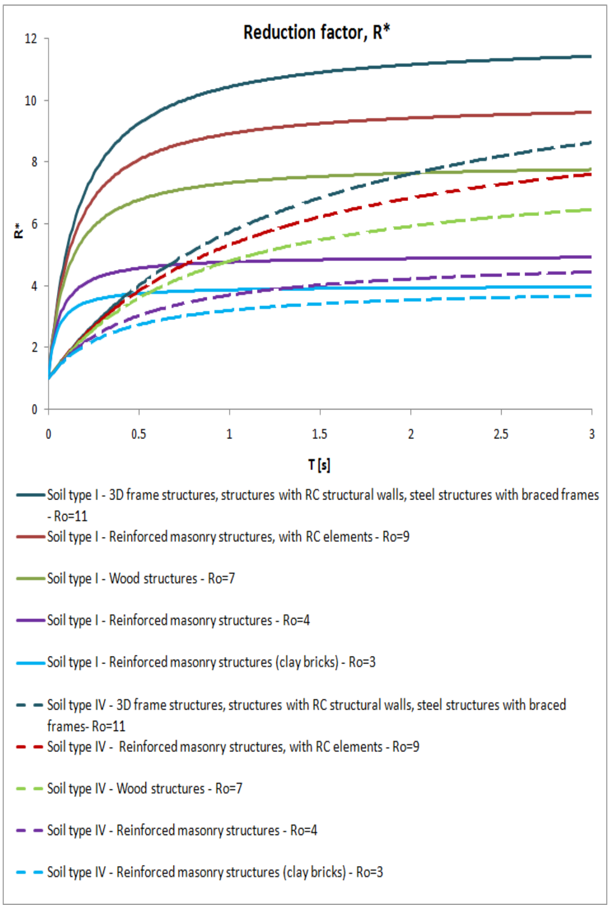


**Fig. 6.** The reduction factor, *R**, for foundation soil types I and IV and different types of structural systems

Depending on the amount of the contribution of walls or frames to the lateral force resisting system, $R_0$ can take the values 4 or 9.

The variation of the reduction factor, $R^*$, with vibration period and structural system type is shown in Fig. 6, for the soil types I and IV. The diagrams in Fig. 7 were obtained by plotting the simplified formula of the reduction factor for soil types I and IV. For the intermediate types II and III, the values of $R^*$ are in between.

The upper and lower limitation of the seismic coefficient in the Chilean norm, mentioned previously, in the paragraph about the static method, is also valid for the modal response analysis method.

Fig. 8 shows, for comparison, design spectra computed according to the Chilean code, for reinforced concrete structures, by using both analysis methods. The following values were considered: foundation soil type IV; $A_0 = 0.40$g. As for the values of the factors $R_0$ and $R$, they were correlated in order to correspond to the same types of structures ($R_0 = 11$ and $R = 7$ for reinforced concrete structures, $R_0 = R = 3$ for masonry structures). One can notice the small differences between the spectra computed with the formulae given by each method.

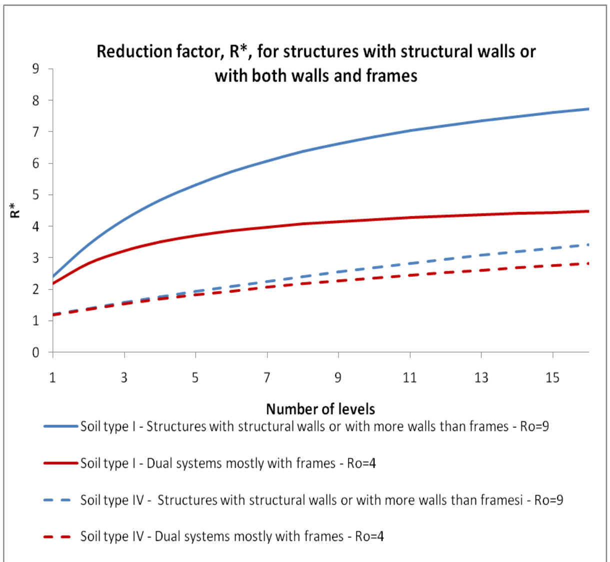


**Fig. 7.** The reduction factor, *R**, in the Chilean code for structures with reinforced structural walls or with both structural walls and frames (dual structures)

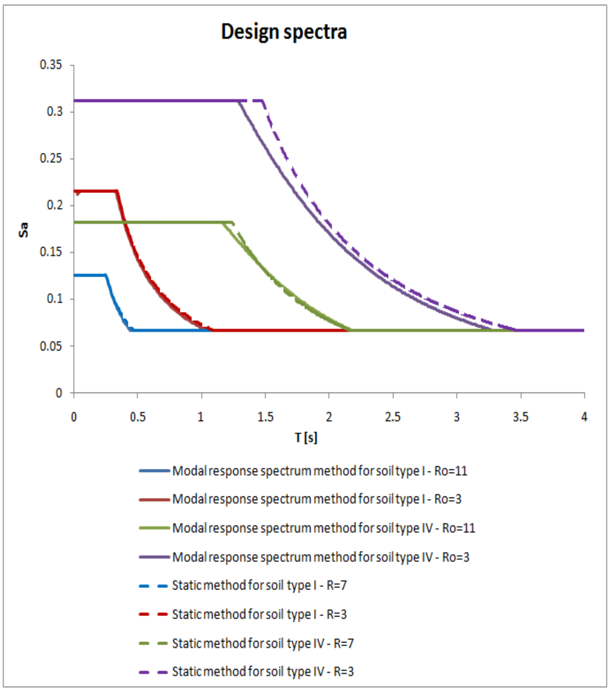


**Fig. 8.** The Chilean code: design spectra for two seismic analysis methods

## 3. SOME COMPARISONS WITH THE ROMANIAN SEISMIC CODE, P100-1 / 2006

Despite the totally different seismic contexts of Chile and Romania, the basic concepts of the two codes are fundamentally the same. This allows for some interesting comparisons to be made.

### *3.1. Static method*

In order to eliminate the influence of the peak ground acceleration, $A_0$, the values of the Chilean seismic coefficient $C$, as computed in the static method, were divided with $A_0 / g$. A normalized design spectrum, denoted by $C^*$, was thus determined, for different $R$ values. The lower and upper limits of $C$, specified by the Chilean code, were also divided by $A_0 / g$. The resulting spectrum is analogous, as significance, to the normalized design response spectrum in the Romanian code, P100-1 / 2006, which will be denoted here by β* ($\beta^*(T) = S_d(T) / a_g$). By plotting the Chilean normalized design spectrum for $R = 7$ and for soil types I and IV, the continuous line diagrams in Fig. 9 were obtained. The small circular markers on the graph, located at the extremities of the constant value zones, show positions which are analogous to the ordinates at the corner periods, $T_C$, in the Romanian code. The normalized design acceleration spectra, according to P 100-1 / 2006, were plotted on the same graph, with dashed lines, for the maximum and minimum values of the corner period, $T_C$, i.e. 0.7 and 1.6. The normalized design spectrum for the Banat zone, with $T_C = 0.7$, was also plotted.

Fig. 9 shows the differences between the normalized elastic response spectra specified by the two codes. These differences are given by the specific characteristics of each seismic zone in the two countries, but also by the different approach in considering the effects of ground conditions on code spectra. Accordingly, the spectra in the Romanian code reflect the characteristics of the subcrustal Vrancea earthquakes, respectively of the shallow crustal Banat seismic events. In the Romanian code, the normalized elastic response spectra do not depend on the soil type, their shape being established

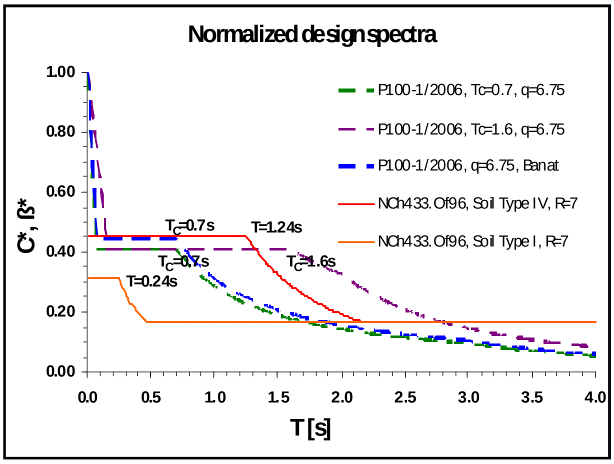


**Fig. 9.** Normalized design spectra, $C^*$ (according to NCh433.Of96, static method: continuous lines) and β* (according to P100 1/2006: dashed lines)

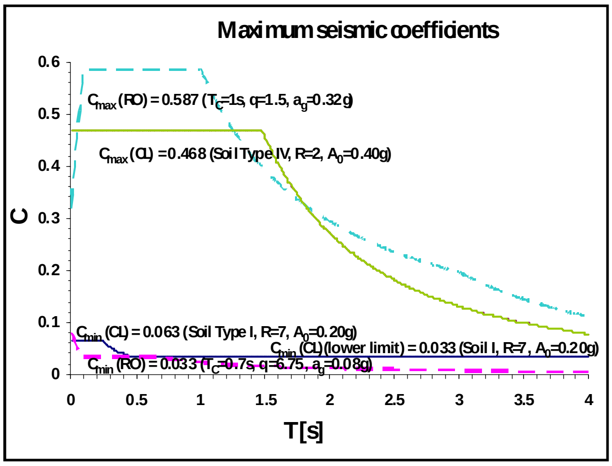


**Fig. 10.** Maximum and minimum seismic coefficients according to the Chilean code (continuous line) and to the Romanian code, with λ = 1 (dashed line)

only based on the corner period, $T_C$. On the other hand, the Chilean code reflects the specificity of the shallow crustal seismic activity affecting this country, classifying the spectra function based on a factor depending on the ground conditions, $S$ (Table 3). A significant feature of the Chilean spectra is the absence of the linear part situated between the origin of the spectrum and the horizontal upper limit. Thus, the Chilean spectrum has a shape somehow similar to the spectra in the previous Romanian seismic codes, P13-63 and P13-70.

Fig. 10 shows, with continuous line, diagrams of the seismic coefficient, $C$, for the highest and lowest values of the reduction factors given by the Chilean code, i. e. $R = 7$ (the maximum value corresponds to reinforced concrete structures) and $R = 2$ (the minimum value corresponds to special structures that cannot be included in any other category) and for soil types I and IV.

On the same diagram, seismic coefficients according to the Romanian code were plotted with dashed line, for the maximum and minimum values of the behavior factor specified by the code, $q$, i.e. $q = 6.75$ (maximum value for reinforced concrete structures of ductility class $H$) and $q = 1.50$ (minimum allowed value for reinforced concrete structures). The $A_0$ and $a_g$ values in the two codes were chosen accordingly, in order to obtain the maximum and minimum possible values of the seismic coefficients. Due to lack of data, it cannot be confirmed whether the chosen combination of Chilean parameters is actually met on a real site. The considered value for $T_C$ is that for the city of Focşani ($T_C = 1$ s), which was chosen as it is located in the zone with the highest $a_g$ specified by the Romanian code. The computation of the seismic coefficients according to P100-1/2006 was made by considering $\lambda = 1$ in the formula of the seismic base shear force, according to the equivalent lateral static force method. For both codes, computations were made for the case of ordinary buildings (importance factors equal to 1).

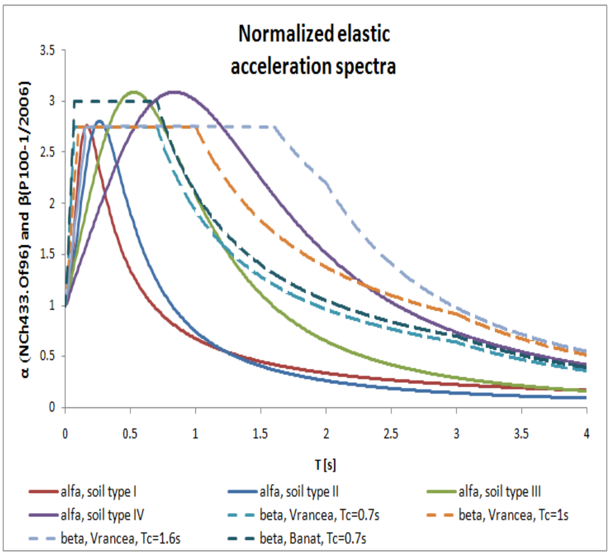


**Fig. 11.** The amplification factor, α, in the Chilean code (continuous line), as compared to the normalized elastic acceleration spectra, β (*T*) in P 100-1 / 2006 (dashed line)

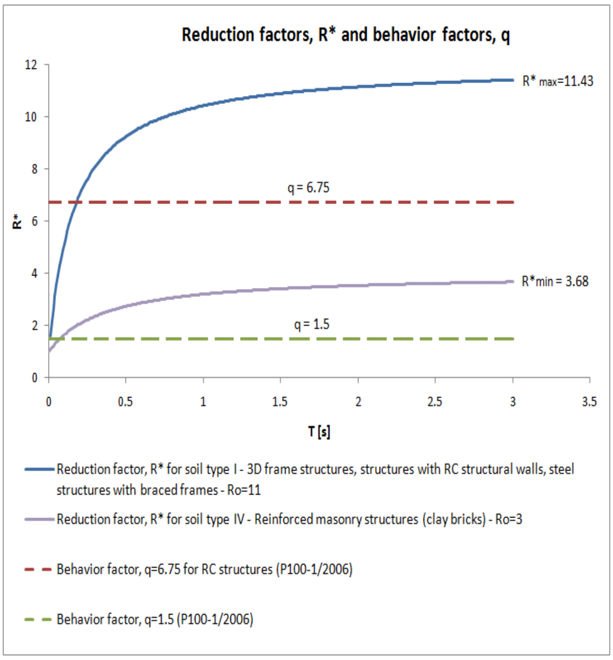


**Fig. 12.** Reduction factors, *R** (Chile, spectral modal analysis method) and behavior factors, *q* (Romania): minimum and maximum values

The values of the seismic coefficients in the two codes lie between the corresponding highest and lowest curves. It can be observed that both the highest and the lowest values of the seismic coefficient (0.587 and 0.033, respectively) are obtained for the Romanian code.

It should be mentioned that the Romanian code does not specify a lower limit for the seismic coefficient, as does the Chilean code. Such a limit was introduced in 2010 by the adoption of the Eurocode 8 (EN 1998-1:2004) as a national standard. The Romanian National Annex to this European norm specifies that the values of the normalized design spectrum, β*, should not be below 0.2. In terms of the comparison in Fig. 10, this would mean a lower limit of 0.016 for *C*.

### *3.2. Spectral analysis method*

By analyzing the spectral analysis method in the Chilean code, it can be noticed that the amplification factor, *α*, has, as a correspondent in the Romanian code, the elastic design spectrum, β (*T*). Fig. 11 displays a comparison between the variation of α and *β* with vibration period.

In what concerns the reduction factor in the mentioned method, $R^*$, its correspondent in the Romanian code is the behavior factor, *q*. Unlike the Chilean code, the Romanian code (in a similar way to Eurocode 8 and to the American codes), does not specify a variation of the behavior factor *q* with the vibration period of the structure.

The maximum and the minimum values of the reduction factor in the Chilean code, and of the behavior factor, *q*, in P 100-1 / 2006, respectively, are shown in Fig. 12.

Further analysis of the February 27, 2010 Chile earthquake, including processing of some significant ground motions recorded during the event, can be found, among others, in [17].

## 4. FINAL REMARKS

The recent seismic event in Chile on February 27th 2010 brought once again into attention, due to the relative reduced number of casualties given its magnitude ($M_w$=8.8), the importance of the seismic design codes in the mitigation of earthquake effects.

The NCh433.Of96 Chilean seismic code provides a set of coherent instructions for earthquake design, including detailed procedures for the evaluation of seismic design forces according to the specific seismicity and soil conditions of the country. However, Chile was not spared of building damage and collapse during the earthquake. According to preliminary reports, among the causes could be identified: very large peak ground accelerations that reached 0.56g locally [15], much larger effective spectral ordinates at certain stations, as compared to those predicted by the code [15], poor detailing of reinforced concrete shear walls [18], [19] etc.

Apart from the causes pertaining to uncertainties inherent to earthquake hazard assessment, building damage and collapse, especially those of high-rise reinforced concrete wall structures occurred also due to some deficiencies of the NCh433.Of96 Chilean seismic code. For instance, the satisfactory behavior of these structures during the March, 1985 Chile earthquake encouraged Chilean code-writers to pay less attention to the specific provisions concerning their design and detailing. As a consequence, even though the clauses B.1 and B.2 of the 1996 code specified that the appropriate U.S. codes (ACI, AISC and AISI) should be used, until the revision of the national codes, for the dimensioning and detailing of concrete and steel structures, this requirement was waived for reinforced concrete wall structures by clause B.2.2 of the same code. Subsequently, most of the observed damage in structural walls was due to the lack of confinement of boundary elements at wall ends and to insufficient measures for ductile detailing [19]; requirements which were included in the ACI 318-95 code, but not in the Chilean code. It is worth noting that, according to a study cited in the above reference, from 640 Chilean buildings with more than 10 stories and built after 1950, 76.7 % were reinforced concrete wall structures, while 21.6 % used wall-frame systems. Taking into account the previously presented regulatory context, such a categorical predominance of wall or dual structures would suggest an extreme vulnerability of the medium- and high-rise building stock in Chile. However, according to the statistical data collected after the 2010 earthquake, that was not the case. An estimation made by Rene Lagos, cited in [19], reveals that, if only structures built between 1985 and 2009 are considered, just 4 buildings collapsed and about 50 had to be demolished. This is equivalent to a percent of 0.5 % failures for buildings with 3 or more stories and of 2.8 % for buildings with 9 or more stories, from the total building stock in the analyzed category. Moreover, if all engineered structures in Chile are taken into account, it results that less than 2.5 % of those suffered damage and that, out of about 400 casualties, less than 20 occurred in engineered structures. The explanation of these low figures can be found in the advanced nationally available know-how in the field of structural design and in the rigorous structural and seismic review of building designs, required by the Chilean laws [19].

The above aspects point out once again the importance, for the mitigation of earthquake effects, of the quality of seismic design codes and of the effectiveness of their enforcement.

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