A COMPARISON BETWEEN THE REQUIREMENTS OF PRESENT AND FORMER ROMANIAN SEISMIC DESIGN CODES, BASED ON THE REQUIRED STRUCTURAL OVERSTRENGTH

I.-G. Craifaleanu[1]

[1] *Associate Professor, Dept. of Reinforced Concrete Structures, Technical University of Civil Engineering & National Institute for Building Research, Bucharest, Romania*
Email: iolanda@incerc2004.ro, i.craifaleanu@gmail.com

ABSTRACT :

Due to several factors, the lateral resistance of a structure is, usually, greater than lateral force used in seismic design. As a result, a structure has a “reserve” strength (overstrength) that can be mobilized in variable amounts when the structure is subjected to lateral seismic loads. The available structural overstrength is difficult to evaluate analytically; a more promising quantity for research is the required overstrength, i.e. the overstrength necessary to a structure in order to withstand specified seismic forces. In the paper, the required overstrength was expressed through the ratio between the actual spectral ordinates and the corresponding values of the code-specified design spectra. The actual spectral ordinates were calculated for two selected sets of representative Romanian seismic records. Calculations were performed, separately, for both versions, old and new, of the Romanian seismic code. Based on the results, the severity of the requirements of the two codes was assessed comparatively.

KEYWORDS: overstrength, seismic design code, response spectra, design spectra

1. INTRODUCTION

As part of the process of harmonization with European standards, a substantial effort has been made in Romania in recent years to implement regulations concerning the seismic design of buildings. Most of the provisions of Eurocode 8 Part 1 (CEN, 2004), were adopted (with a number of required adjustments) in the new Romanian seismic design code, P100-1/2006 (MTCT, 2006). This new code introduces important changes in comparison with the previous one, P100-92 (MLPAT, 1992), one of the most significant being the evaluation of seismic forces.

A comparative analysis of behavior factors and seismic forces specified by the two versions of the code is made in this paper with reference to the provisions of Eurocode 8. Then, required overstrength is evaluated for both versions of the Romanian seismic design code. Based on the results, severity assessments are made.

2. DESIGN SPECTRA AND BEHAVIOR FACTORS IN THE NEW AND IN THE OLD ROMANIAN CODES FOR SEISMIC DESIGN

2.1. New Romanian seismic design code (P100-1/2006)

The structure of the provisions concerning behavior factors in the new Romanian seismic design code is, in general, similar to that of Eurocode 8. However, the values of q are different from those in the European norm.

The shape of the elastic response spectrum for the horizontal components of seismic action was modified, as compared with the one in the 1992 version of the code, in order to be compatible with that in Eurocode 8.

The new response spectra are specified for zones characterized by three values of the control (corner) period T_C, 0.7 s, 1.0 s and 1.6 s. For each of these three zones, a normalized acceleration response spectrum is specified. The normalized acceleration response spectrum $\beta(T)$ for the horizontal components of seismic action and for 5% damping follows the Eurocode 8 format and is given by the following equations:

$$T \le T_B: \qquad \beta(T) = 1 + \frac{(\beta_0 - 1)}{T_B} T \tag{1}$$

$$T_B < T \le T_C: \qquad \beta(T) = \beta_0 \tag{2}$$

$$T_C < T \le T_D: \qquad \beta(T) = \beta_0 \frac{T_C}{T} \tag{3}$$

$$T > T_D: \qquad \beta(T) = \beta_0 \frac{T_C T_D}{T^2} \tag{4}$$

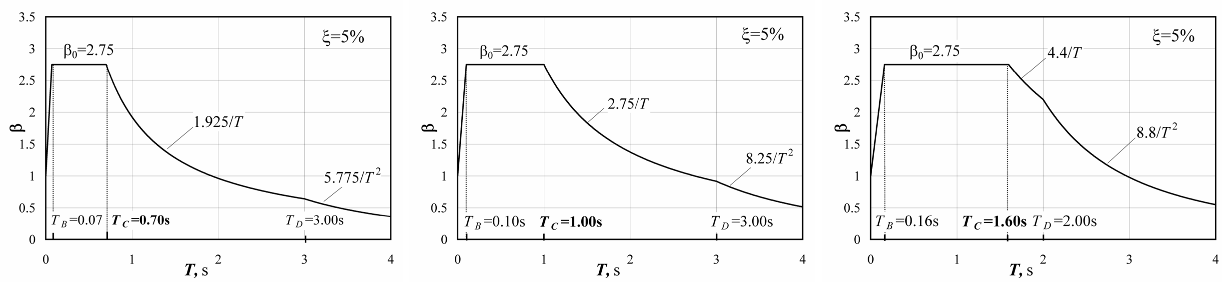
where:

$\beta(T)$ is the normalized acceleration response spectrum (elastic);

β_0 is the maximum dynamic amplification factor;

T is the fundamental period of vibration of a single-degree-of-freedom structure.

Figure 2.1 shows the spectra in the new Romanian seismic design code for all specified values of T_C.



a) $T_C = 0.7$ s b) $T_C = 1.0$ s c) $T_C = 1.6$ s

Figure 2.1. P100-1/2006. Elastic response spectrum normalized by PGA

The control periods T_B, T_C and T_D are given in Table 1.

Table 1. Reference periods of response spectra T_B, T_C, T_D, for horizontal ground motion

Control period	T_B, s	T_C, s	T_D, s
$T_C = 0.7$s	0.07	0.70	3.00
$T_C = 1.0$s	0.10	1.00	3.00
$T_C = 1.6$s	0.16	1.60	2.00

The elastic response spectrum for horizontal ground motion is defined as:

$$S_e(T) = a_g \beta(T). \tag{5}$$

One should note that the T_C value of 1.5 seconds in the 1992 code was modified to 1.6 in the 2006 release as a result of extensive studies performed by Lungu et al. (Lungu, 2004). As these studies also suggested that a higher value for the horizontal plateau of the spectra would be more adequate, the plateau was consequently raised to the value of 2.75.

Figure 2.2 shows an overall comparison between the normalized elastic response spectra in the two releases of P100.

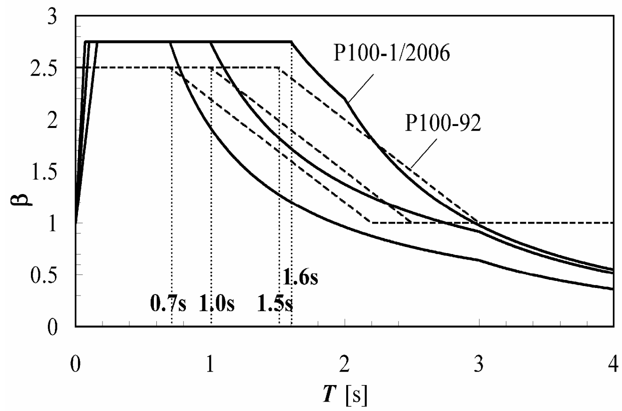
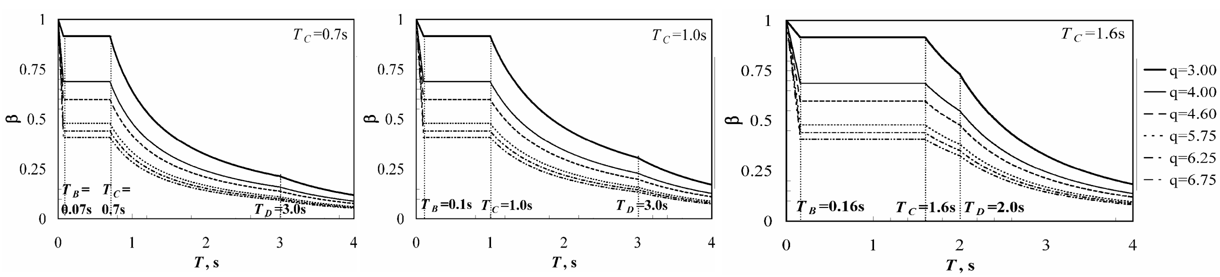


Figure 2.2. Comparison between the normalized elastic response spectra in the two releases of P100

A separate spectrum with $\beta_0 = 3$ and $T_C = 0.7$ s is given in the 2006 release for the crustal sources in the Banat area (in the southwestern part of Romania) where a series of significant seismic events occurred in 1991.

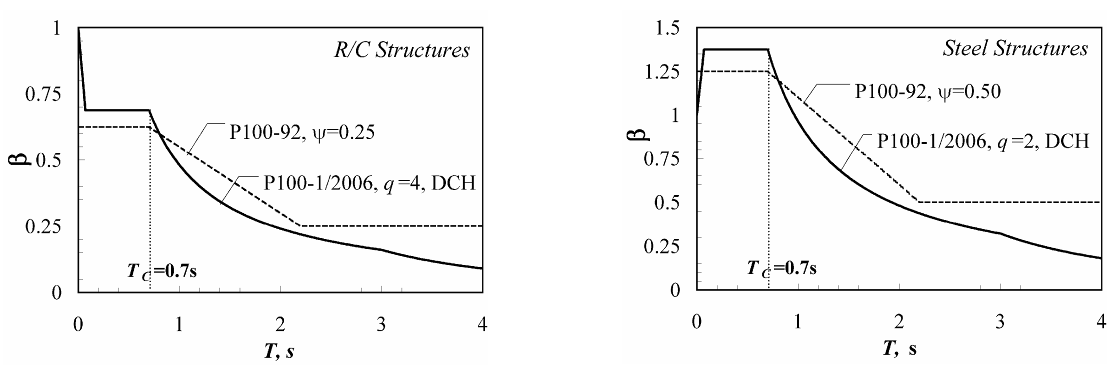
Figure 2.3 presents, for illustration, design spectra normalized by PGA as prescribed by P100-1/2006 for reinforced concrete structures of regular elevation and of high ductility class (DCH). The display order for the curves in all diagrams is the same as that given in the legend.



a) $T_C = 0.7$s b) $T_C = 1.0$s b) $T_C = 1.0$s

Figure 2.3. P100-1/2006. Design spectra normalized by PGA for reinforced concrete structures of regular elevation and of high ductility class (DCH)

A comparison between the requirements of the two versions of the Romanian seismic design code is shown in Figure 2.4 for two pairs of corresponding values of the ψ coefficient and of the *q* factor.



a) b)

Figure 2.4. Comparison between the requirements of the two versions of the Romanian seismic design code

Table 2. Values of the behavior factor, q, for reinforced concrete structures regular in elevation

Structural type		DCM		DCH		P100-92 (1/ψ)
		EC8	P100-1/ 2006	EC8	P100-1/ 2006	
Frames	one-story buildings	3 α_u/α_1 (3.30)	3.5 α_u/α_1 (4.025)	4.5 α_u/α_1 (4.95)	5 α_u/α_1 (5.75)	5.00; 6.66
	multistory, one-bay frames	3 α_u/α_1 (3.60)	3.5 α_u/α_1 (4.375)	4.5 α_u/α_1 (5.40)	5 α_u/α_1 (6.25)	4.00; 5.00
	multistory, multi-bay frames	3 α_u/α_1 (3.90)	3.5 α_u/α_1 (4.725)	4.5 α_u/α_1 (5.85)	5 α_u/α_1 (6.75)	4.00; 5.00
Dual systems.	frame-equivalent	3 α_u/α_1 (3.90)	3.5 α_u/α_1 (4.025; 4.375; 4.725)	4.5 α_u/α_1 (5.85)	5 $\alpha_u/\alpha 1$ (5.75; 6.25; 6.75)	-
	wall-equivalent	3 α_u/α_1 (3.60)	3.5 α_u/α_1 (4.375)	4.5 α_u/α_1 (5.40)	5 α_u/α_1 (6.25)	-
Walls	systems with only two uncoupled walls per horizontal direction	3	3	4 α_u/α_1 (4.00)	4 α_u/α_1 (4.00)	4.00
	other uncoupled wall systems	3	3	4 α_u/α_1 (4.40)	4 α_u/α_1 (4.60)	4.00
	coupled wall systems	3 α_u/α_1 (3.60)	3.5 α_u/α_1 (4.375)	4.5 α_u/α_1 (5.40)	5 α_u/α_1 (6.25)	4.00
Torsionally flexible system		2	2	3	3	-
Inverted pendulum system		1.5	2	2	3	2.86

Table 3. Values of the behavior factor, q, for steel structures regular in elevation

Structural type		DCM		DCH		P100-92 (1/ψ)
		EC8	P100-1/ 2006	EC8	P100-1/ 2006	
Unbraced frames / Moment resisting frames	one-story buildings	4	2.5; 4	5 α_u/α_1 (5.50)	2.5; 5 α_u/α_1 (2.50; 5.00; 5.50)	2.94; 3.46; 5.00; 5.88
	multi-story buildings	4	4	5 α_u/α_1 (6.00; 6.50)	5 α_u/α_1 (6.00; 6.50)	5.88
Frames with concentric bracings	bracing with tension diagonals	4	4	4	4	4.00; 5.00
	V-bracings	2	2	2.5	2.5	2.00; 2.50
Frames with eccentric bracings,		4	4	5 α_u/α_1 (6.00)	5 α_u/α_1 (6.00)	5.00
Inverted pendulum structures		2	2	2 α_u/α_1(6.00)	2 α_u/α_1 (6.00)	1.54; 2.00
Structures with concrete cores or concrete walls		2	2	3	3	-
Dual frames	moment resisting frame combined with concentric bracing	4	4	4 α_u/α_1 (4.8)	4 α_u/α_1 (4.8)	2.00; 2.20; 4.00; 5.00
	moment resisting frame combined with eccentric bracing	-	4	-	5 α_u/α_1 (6.00)	2.00; 2.20; 4.00; 5.00

Tables 2 and 3 (Craifaleanu, 2005) show comparisons between the values of the behavior factors in Eurocode 8 and the new and the old Romanian seismic design codes. Categories in the above tables are chosen in order to ensure the closest match between the different types of structures specified by the three codes. The values in parentheses show *q*-factors calculated based on α_u/α_1 ratios specified by the code.

3. Structural Overstrength

Due to several factors, the lateral resistance of a structure is usually greater than the lateral force used in seismic design. Among those factors that should be mentioned are the following: higher material strengths than those used in design, structural redundancy, member oversize resulting from story drift limitations or from detailing requirements, strength hardening, multiple load combinations, the effect of non-structural elements, larger reinforcement areas resulting from minimum reinforcement requirements and strain-rate effect. As a result, a structure has a reserve strength that can be mobilized in variable amounts when the structure is subjected to lateral seismic loads.

Moreover, for ordinary buildings the actual lateral strength is lower than the lateral force that would be induced in the structure if it behaved elastically. This is due to the current design concept that allows, under certain conditions, the occurrence of yielding in structural members.

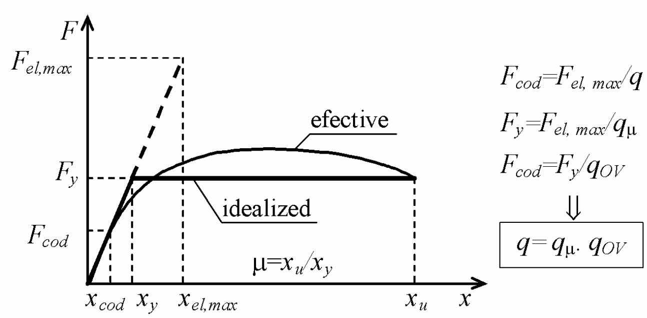


Figure 3.1. Simplified lateral force – deformation diagram of a structure

A simplified lateral force-deformation diagram of a structure is given in Figure 3.1 (Uang, 1991). The idealized (bilinear elasto-plastic) diagram corresponds to the meaning of the overstrength used in Eurocode 8 and in P100-1/2006 for defining the ratio α_u/α_1. The following notations were used:

- F_{code} = design seismic force;
- F_y = yield strength level;
- $F_{el,max}$ = maximum base shear that develops in the structure if it remains in the elastic range;
- x_{code} = deformation at design seismic force;
- x_y = deformation at yield strength level;
- $x_{el,max}$ = deformation at $F_{el,max;}$
- x_u = ultimate displacement (prior to collapse);
- $\mu = x_u/x_y$ = lateral displacement ductility.

With the above notations, it results that the behavior factor q in Eurocode 8 and in P100-1/2006 represents the product of two factors, one containing the effect of inelastic behavior (q_μ), and the other the effect of overstrength (q_{OV}):

$$q = q_\mu \cdot q_{OV} \ . \tag{6}$$

The overstrength factor q_{OV} is difficult to evaluate analytically. At present, the evaluation of q_{OV} is based mostly on existing experience for different structural types; however, the required overstrength for structures designed according to specified design forces to withstand given seismic actions can be determined quite simply.

The required overstrength is a simple criterion for the assessment of the severity of seismic design provisions. In the following, this criterion is used for a comparative evaluation of the requirements of the two versions, old and new, of the P100 code.

In order to perform the assessment, design spectra such as those shown in figures 2.1 and 2.2 were used to represent code requirements. Then, normalized acceleration spectra with 10% probability of exceedance were calculated for q_μ values equal to the q values specified by P100-1/2006 and for q_μ values equal to the $1/\psi$ values specified by P100-92 respectively.

Two sets of seismic motions were used to calculate these spectra: the first set consisted of 23 broad frequency band motions recorded in the northeastern part of the country (Moldavia) and the second set consisted of 8 narrow frequency band motions recorded in the soft soil conditions of Bucharest. All motions were recorded during the three strong Vrancea earthquakes that affected Romania in 1977, 1986 and 1990 respectively. The two sets were selected in order to be representative for the seismic zones characterized through the corner periods T_C specified by the above mentioned codes, i.e. $T_C = 0.7$ s and $T_C = 1.6$ s (1.5 s in the old code).

The spectra were determined by considering a 5% damping ratio, an elasto-plastic hysteretic model and a lognormal distribution of the spectral ordinates.

The required overstrength was expressed through the factor R_{OV} calculated as the ratio between the spectral ordinates with 10% probability of exceedance (determined by considering a lognormal distribution) and the design spectra. Both types of spectra were determined for the same specified q value. The R_{OV} factor was preferred to q_{OV} since it has a more comprehensive definition as it was obtained directly on the basis of seismic design forces.

The sets of behavior factors q used in the study correspond to structures of DCH of uniform elevation. The calculations were performed practically for all values given by P100-1/2006 for reinforced concrete structures and for steel structures, respectively.

Figures 3.2 and 3.3 show, for illustration, the spectral curves that were used as a basis for the determination of R_{OV} values for reinforced concrete structures. Similar curves were used for steel structures by considering the appropriate values of q and $1/\psi$.

Figures 3.4 and 3.5 show the diagrams of the R_{OV} factor for reinforced concrete structures corresponding to the new and to the old code.

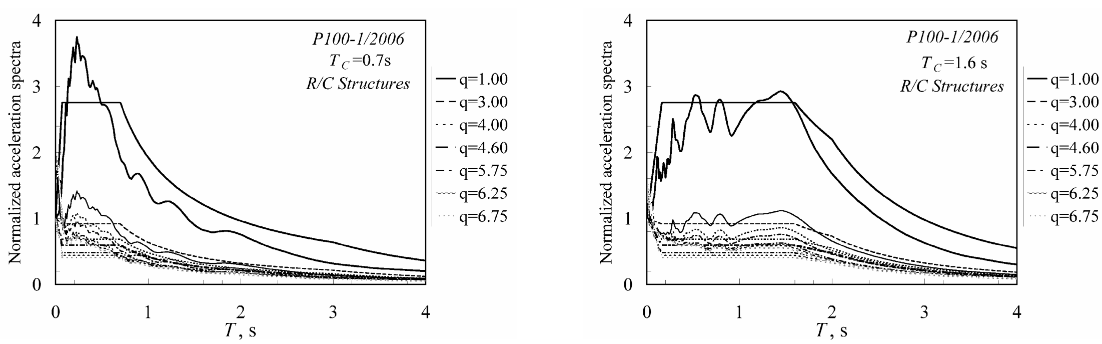
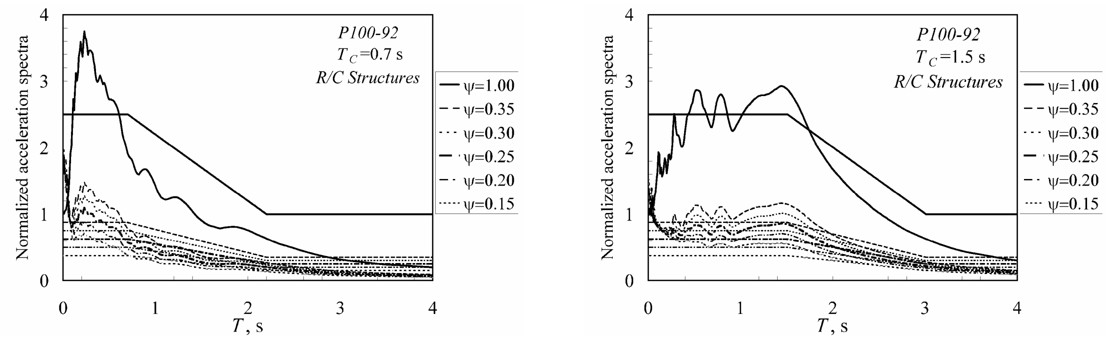


Figure 3.2. P100-1/2006. Spectral curves used as a basis for the determination of R_{OV} values for reinforced concrete structures of high ductility class (DCH)



a) b)

Figure 3.3. P100-92. Spectral curves used as a basis for the determination of R_{OV} values for R structures

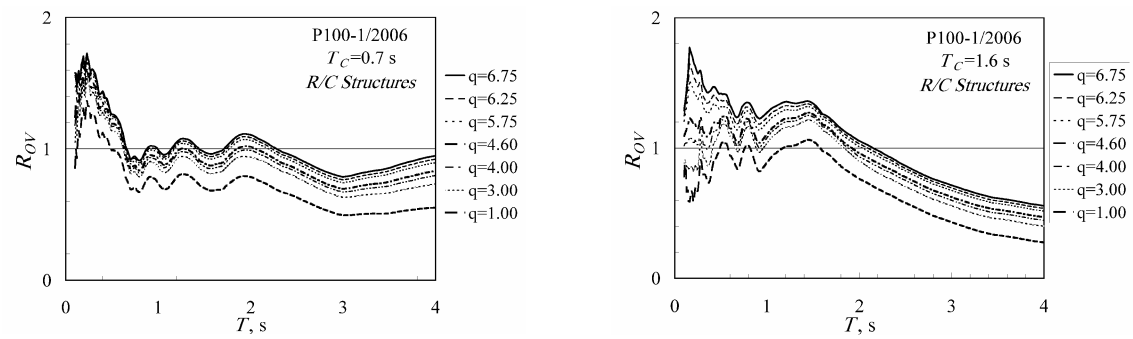


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Figure 3.4. P100-1/2006. R_{OV} values for reinforced concrete structures of high ductility class (DCH)

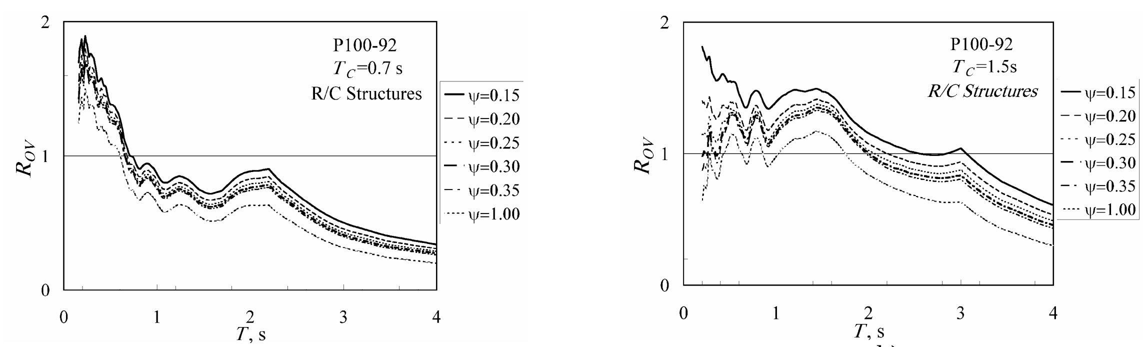

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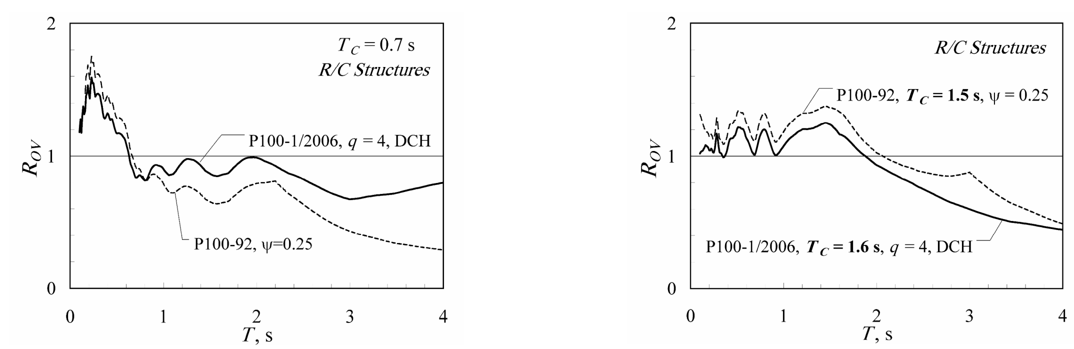
Figure 3.5. P100-92. R_{OV} values for reinforced concrete structures



a) b)

Figure 3.6. Comparative diagrams of R_{OV}

Similar diagrams were determined for steel structures by considering the appropriate sets of q and ψ values. Their shape is much similar to the diagrams for reinforced concrete structures, but the range of variation of the q and ψ values is narrower, as one can notice from Table 3.

The comparative diagrams in Figure 3.6 illustrate two particular cases in which the values of q and $1/\psi$ are equal for a specified structure type, in the two codes analyzed.

4. CONCLUSIONS

1. For the required overstrength expressed by the ratio between the demands imposed by Vrancea earthquakes and those imposed by the Romanian seismic design codes, R_{OV} varies significantly with the period of vibration and with behavior factors.
2. For both codes, required overstrength can reach values up to 1.8 times the seismic design force for behavior factors situated at the limit of the usual range. However, with the exception of periods shorter than 0.5 s, the values of R_{OV} are currently below 1.35–1.40 for the new seismic design code (P100-1/2006) and below 1.5 for the old code (P100-92).
3. By modeling seismic action through two sets of selected accelerograms recorded during strong Vrancea earthquakes in Romania considered as relevant for seismic zones with corner periods T_C of 0.7 and 1.6 (1.5) s, it resulted that the required overstrength for structures designed to resist the seismic forces specified by the new code was lower than that in the old code. This is an indication of the more conservative character of the new provisions.
4. Regarding the values of the behavior and overstrength factors, further studies should be performed for each structural type using detailed structural models and taking into account all the provisions in the new code in order to validate the currently calculated values of required overstrength.

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