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Behavior factors: a proposal for RC framed buildings designed according to current seismic codes

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Summary

Nowadays the seismic design is based on the results of elastic analysis of structures (indeed of the growth of the non-linear based procedures). In order to perform the analysis of structures which are modeled with elastic behavior, although the engineer knows that they could reach plastic behavior, seismic design codes prescribe the reduction of elastic design spectra using response reduction factors. These factors have been formulated based on engineering judgement. In this article it is proposed a new methodology in order to determine these response reduction factors by means of the results of the non-linear response of soils representative of the profiles of the Venezuelan seismic code simultaneously with reinforced concrete structures modeled as multi-degree of freedom systems. Computed values are greater than those the seismic code prescribes, then the design of buildings is controlled by displacements rather strength.

KEYWORDS: Design spectrum, response reduction factor, non-linear analysis, ductility, redundancy, overstrength.

1. INTRODUCTION

The procedure performed to obtain the design equivalent forces is based on the response spectra, which represent the geotechnical characteristics of the buildings' locations. Those spectra are reduced in order to take into consideration the inelastic behavior the structures may reach when are subjected to the action of a strong ground motion. The reduction is obtained using response reduction factors (R), named behavior factors (q) according the Eurocode-8 [1], which were proposed by Veletsos and Newmark [2] in the format that they are applied in most of the worldwide seismic codes.

In the past two decades a large number of works dealing with the study of the response reduction factors were presented [3], some of them were focused into determinate the components of R. In sake of the brevity, in this work only the ATC-19 [4] approach is referenced among other relevant works:

$$R = \left(R_S \cdot R_\mu\right) \cdot R_R \tag{1}$$

Where R_S is the strength-based reduction component, R_μ is the ductility-based component and R_R is the redundancy-based reduction component. The first two components are time-dependent, while the values of are used to assume a fix value, depending on the structural type. It is important to mention that obtaining values of is a difficult task, so they are usually associated with the strength-based component, in a single factor called overstrength-based component, defined according to:

$$R_\Omega = R_S \cdot R_R \tag{2}$$

2. METHODOLOGY

The new procedure to obtain the response reduction factors is performed in two steps. The first one consists into determine the ductility-based component by means of the procedure formulated by [5, 6, 7], see Equation 3:

$$\begin{aligned} R_\mu &= 1 + \frac{T}{T_g}\left(\frac{\mu}{\beta} - 1\right) \quad & T \le T_g \\ R_\mu &= \frac{\mu}{\beta} \quad & T > T_g \end{aligned} \tag{3}$$

In this equation T is the period (in sec.), T_g is the characteristic soil period, μ is the design assumed ductility and is a coefficient which depends on ductility and the dynamic non-linear response of the soil. Values of and can be seen in Table 1 and Table 2, respectively.

Table 1. Values of the characteristic period T_g

Soil type	μ=2	μ=4	μ=6
S1	0,12	0,19	0,25
S2	0,22	0,29	0,38
S3	0,34	0,47	0,74
S4	0,60	0,71	0,82

Table 2. Values of the coefficient β

Soil type	μ=2	μ=4	μ=6
S1	1,16	1,29	2,02
S2	1,24	1,35	1,50
S3	1,26	1,27	1,38
S4	1,28	1,27	1,38

Those values are used in Equation 3 for the determination of R_μ for the soil where the building is located. Note in Tables 1 and 2 that the soil profiles correspond to the spectral shapes the Venezuelan seismic code [8] prescribe. Results are shown in Figures 1 to 4 for those soil profiles.

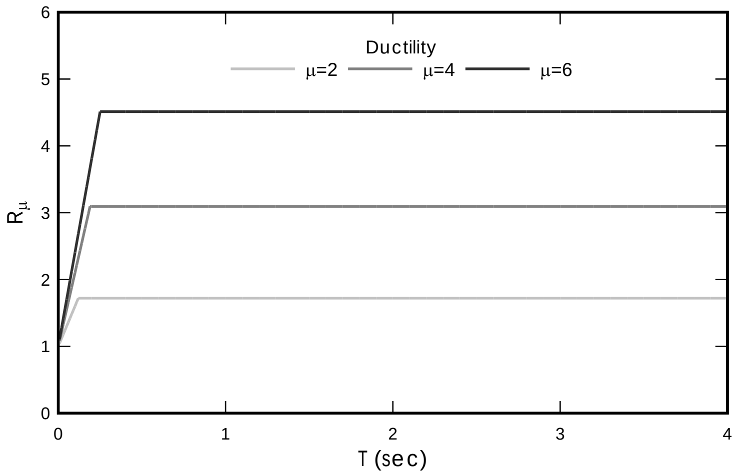

Figure 1. Ductility reduction component for soil S1

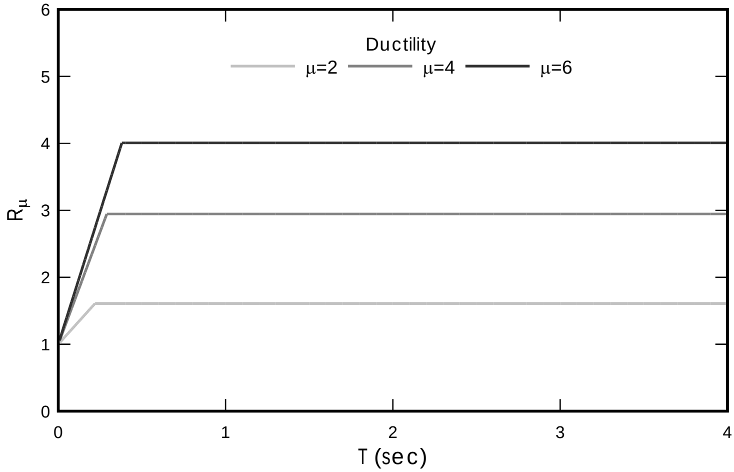

Figure 2. Ductility reduction component for soil S2

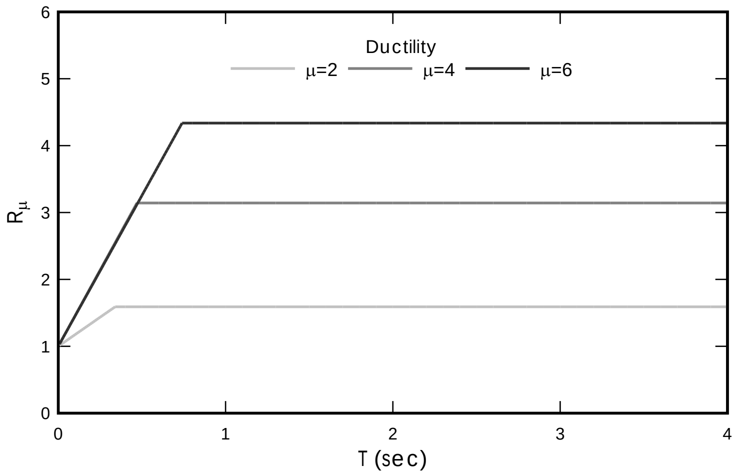

Figure 3. Ductility reduction component for soil S3

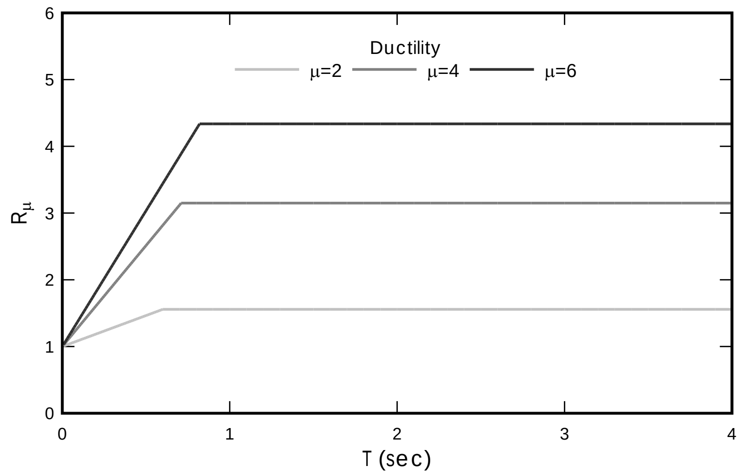

Figure 4. Ductility reduction component for soil S4

The next step consists in to obtain a reliable value for the overstrength component defined according to Equation (2). This goal is matched using the results proposed in , which were computed from non-linear analysis of RC framed buildings with various numbers of spans and stories, designed for high ductility according current codes [9]. Figure 5 shown the resulting values of R_{μ} plotted vs. the level numbers of the buildings; note the convenience to calculate the mean value in order to apply a unique value for the case studies that are presented in next section.

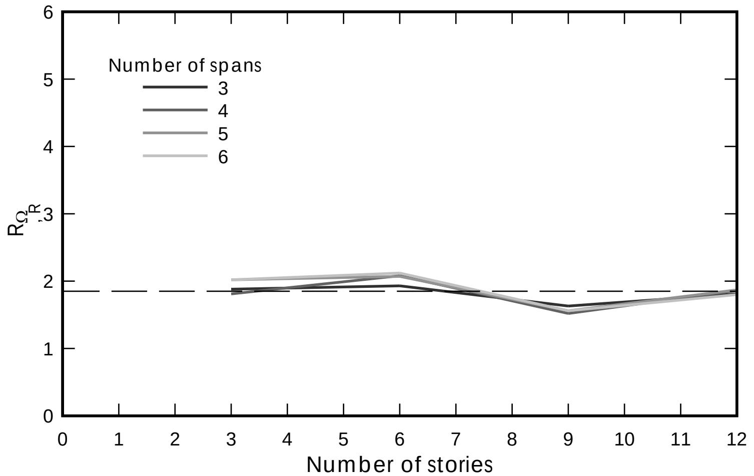

Figure 4. Ductility reduction component for soil S4

2. CASE STUDIED

In order to know how suitably the proposed methodology is, a set of RC regular framed buildings have been designed following the standard procedures the current Venezuelan seismic prescribes and the new methodology. The set consist in four buildings with different number of stories: 3, 6, 9 and 12. Columns and beams have square and rectangular shapes, the floors are two-way 15cm thick solid slabs. Members have been detailing and dimensioning using 25 MPa concrete and 420MPa reinforcement steel (longitudinal and transversal).

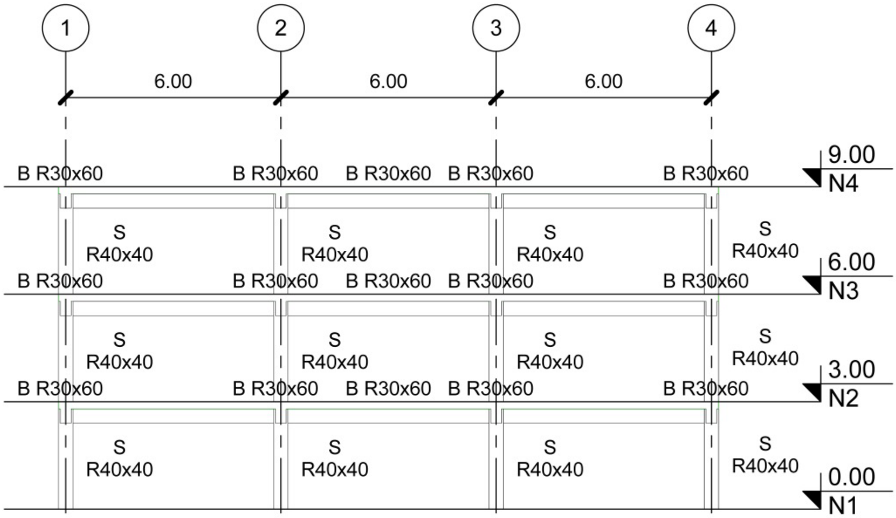

Figure 5. Lateral view of the 3 stories building

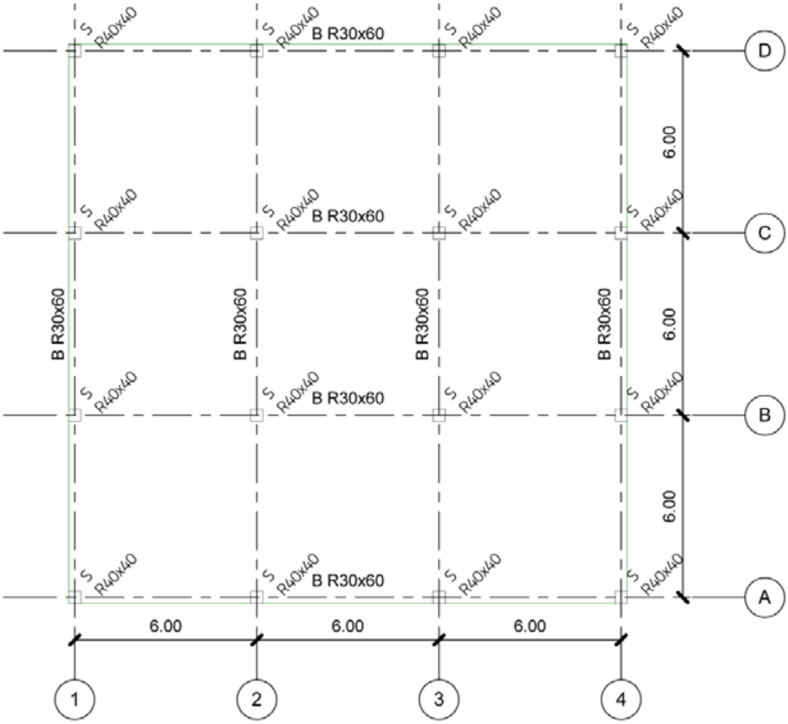

Figure 6. Typical plan view of the buildings

Figures 5 and 6 showed the lateral view of the 3 stories building and a typical plan view of the buildings, respectively. Note that the spans are 6,00m long, equispaced in both directions. Seismic parameters specific for a high-level hazard site selected for the analysis are shown in Table 3.

Table 3. Seismic parameters selected for the analysis

Parameter	value
Seismic zone	3
Design acceleration	0,3g
Design level	DL-3
Response reduction factor	6

Analysis process requires an initial value for the structural fundamental period. The approximate formulae let to calculate this period using the building total height h_n (in m) and an empirical coefficient C_t which depends on the main material that constitutes the structure:

$$T = C_t h_n^{3/4} \tag{3}$$

Values of the approximate fundamental period are used with the parameters contained in Tables 1 and 2 in order to determine the values of R_μ via Equation (3). Results are summarized in Table 4.

Table 4. Values of the coefficient β

Number of levels	T (sec)	R
3	0,36	7,12
6	0,61	7,42
9	0,83	7,42
12	1,03	7,42

Inelastic spectra are then obtained using those values jointly with the value corresponding to the standard procedure. Figure 7 shows the elastic and inelastic spectra computed according the guidance of the Venezuelan seismic code.

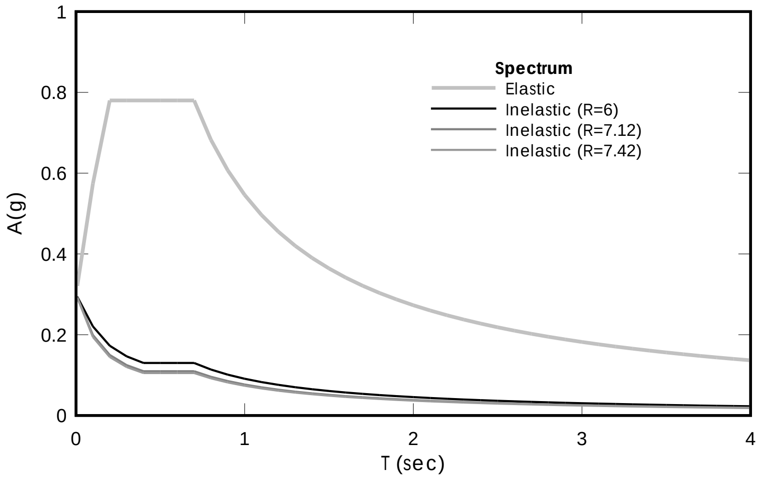

Figure 6. Elastic and inelastic design spectra

Elastic analysis of the models is performed using the forces resulting for modal-spectral analysis for the case of the buildings designed by the code's prescribed procedure and the alternative procedure described above. Derived displacements of the gravity centers in each floor computed from elastic analysis are then used to compute the inelastic displacements. Note that most of the seismic codes prescribe the amplification of the elastic displacements times R. In the case of the alternative procedure, those displacements are computed using amplification factors derived from energy-based relationships obtained from non-linear response of similar buildings [10, 11]. For buildings whose dynamic response is velocity-dependent, the displacement amplification factor is calculated trough:

$$C_\mu = \frac{\left(R_\mu^2 + R_\Omega^2\right)}{2R_\Omega} \tag{4}$$

While for buildings whose dynamic response is displacement-dependent, displacement amplification factor must be calculated using:

$$C_\mu = R_\mu R_\Omega \tag{5}$$

All the terms in Equation (4) and (5) are the same defined in past section. Inelastic displacements allowing calculating inter-story drifts, in order to check the dimensioning of the whole structure by comparing against a maximum inter-story drift that the code prescribes, 1,8% according to the Venezuelan seismic code. In Figure 7 are shown the inters-story drifts calculated vs. the height, for the three levels building, using standard procedure (Figure 7a) and alternative procedure (Figure 7b).

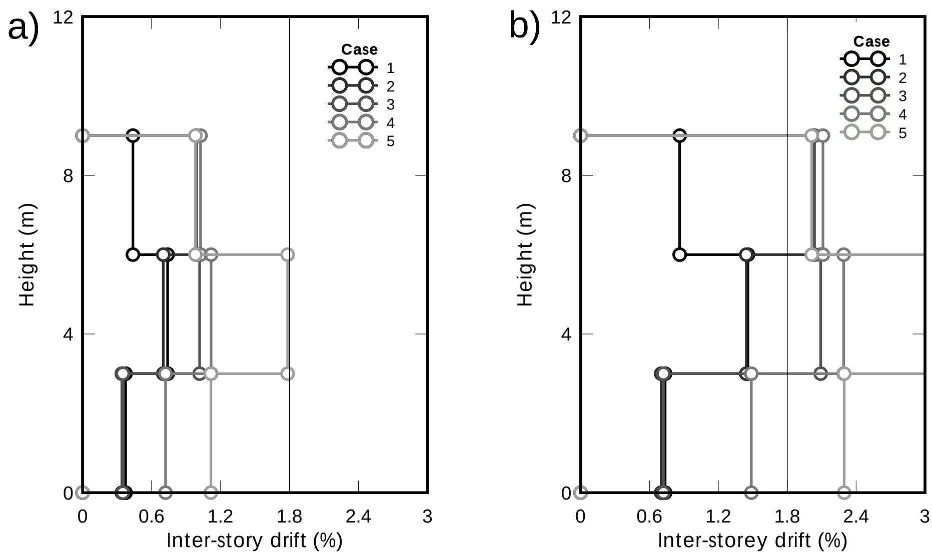

Figure 7. Inter-story drifts of the three levels building

The reader may note that for each procedure five cases have been plotted. Those cases correspond to different building's configurations in which the cross sections of the columns have been modified in order to satisfy that their interstorey drifts are not greater than the maximum value of 1,8%. Selected configurations received a numeration that range from the most flexible (case 1) to the stiffer (case 5). It is evident the difference between the results obtained with the standard procedure and the alternative one, because in Figure 7a all the cases have interstorey drifts lesser than 1,8%, while from Figure 7b only the case 5 fulfill the adopted criterion.

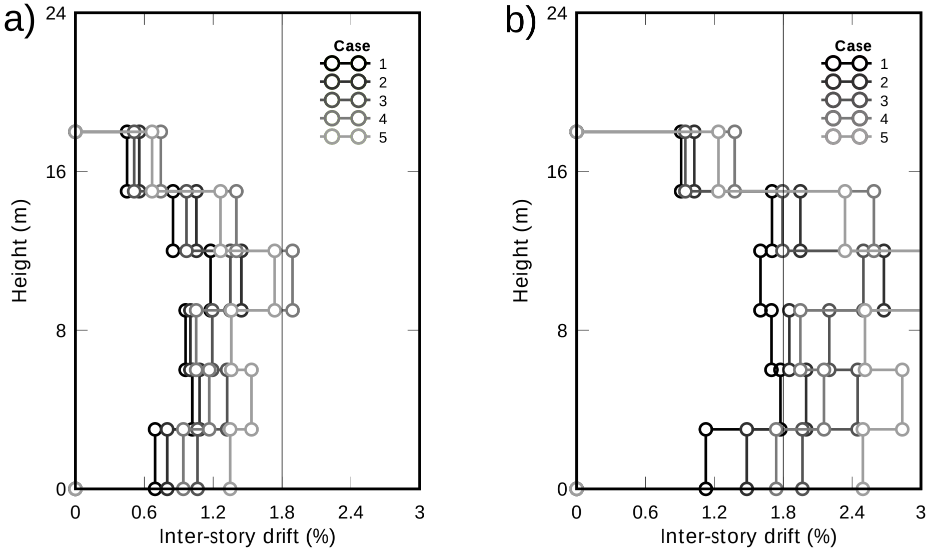

Figure 8. Inter-story drifts of the six levels building

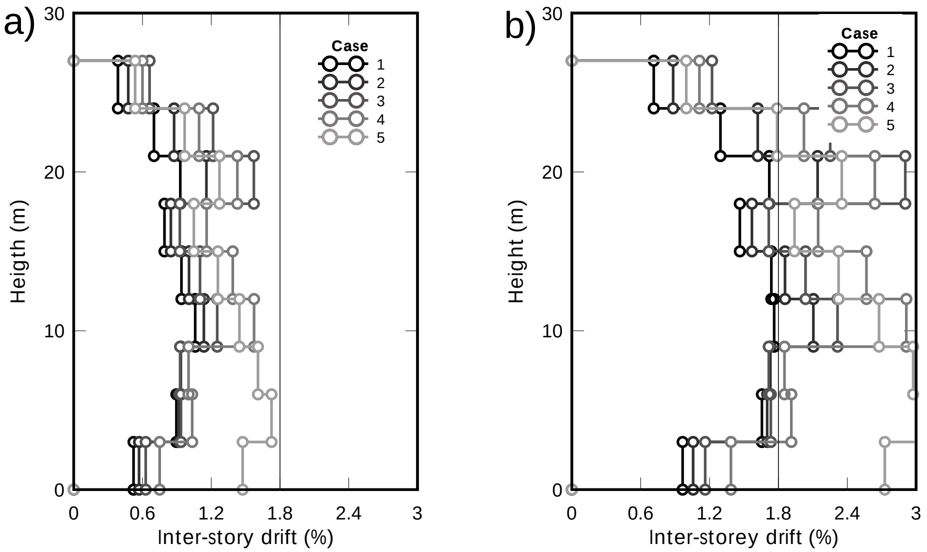

Figure 9. Inter-story drifts of the nine levels building

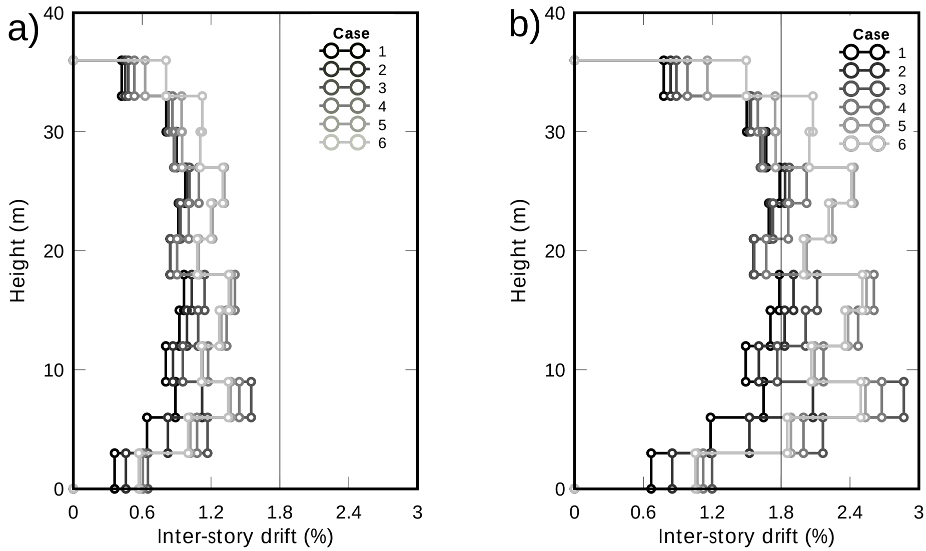

Figure 10. Inter-story drifts of the twelve levels building

Results for 6, 9 and 12 stories buildings can be seen in Figures 8, 9 and 10, respectively. Those figures reveal that according to standard procedure every configuration can be considered for the next stage in the design process: the detailing of structural members. But the resulting longitudinal reinforcement ratio exceed the maximum value that the Venezuelan RC design code [12] prescribes for columns (6%) or have values too large which are technically impossible to place in cross sections. Only case 5 is able to satisfy both the maximum interstorey drift and reinforcement ratio. In contrast, the cases calculated using the alternative procedure, exhibit results which demonstrate that only the stiffer case (case 5) satisfy the maximum inter-story drift criteria, and also satisfy the reinforcement ratio requirement.

According to obtained results, it can be concluded that standard procedure is strength-controlled, while alternative procedure is displacement-controlled, this last feature produce more realistic designs with a desirable performance when buildings are under the effect of strong ground-motions [13, 14, 15, 16]. By the other hand, it is evident that the design process will demand less computational effort if the alternative process is applied, because it reduce the number of iterations for determine the appropriate configuration.

3. CONCLUSIONS

In this paper an alternative procedure has been presented. The procedure wants to present a rational approach in order to determine the response reduction factors for RC framed buildings in a compatible format with the factors used in the current version of the Venezuelan Seismic Code.

The procedure can be extended to other structural typologies used for seismic design.

The procedure is suitably and easy to apply in order to let seismic engineers to obtain inelastic response design spectra they need to perform the analysis and design of new buildings. The procedure includes determining inelastic displacements using equations derived from energetic-based methods, whose components are the same than the used for conduct the analysis.

Dimensioning of buildings using the alternative procedure led to reach a configuration which not only satisfy the maximum inter-story drift that the Venezuelan seismic code prescribes (displacement-controlled design) but also satisfy the maximum reinforcement ratio for structural elements (strength-controlled design). This feature led to reduce the computational effort, because it requires less iteration to find an adequate solution for the structural dimensioning.

Acknowledgements

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