Chapter 12

# Seismic Evaluation of Low Rise RC Framed Building Designed According to Venezuelan Codes

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Additional information is available at the end of the chapter

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## 1. Introduction

Along its history, Venezuela has been severely affected by destructive earthquakes [1]. Approximately 80% of the population lives in seismically active areas, where have occurred destructive earthquakes even in recent times [2]; The seismic hazard, inadequate design and construction of buildings as well as the damage occurred from previous earthquakes, demonstrate a high vulnerability in existing buildings. Then it is essential to continuously make progress and research in the field of earthquake engineering and upgrade the seismic design codes. Seismic upgrade requires the evaluation or predictions of the expected damage to structures at the time of an earthquake of a certain severity occur. From this prediction it can be defined solutions for the reduction of structural vulnerability [3].

The damage occurred in buildings after an earthquake indicates the need for reliable methodologies for the evaluation of seismic behavior of the existing buildings. According to current technical and scientific advances, seismic evaluation of reinforced concrete (RC) structures can be done by two different approaches: empirical methods and mechanical methods [4]. The current tendency of earthquake engineering in the evaluation of structural behavior is the application of simplified mechanical methods based on performance, involving the capacity spectrum [5], because there are developed refined models and detailed analysis.

This study used a mechanical method that involves non-linear analysis with deterministic and probabilistic approaches, as well as procedures of analysis based on Limits States defined by displacements [6], in order to evaluate the behavior of a low rise RC building with plan irregularity, designed according to Venezuelan codes [7]-[9] and subjected to seismic action effect. Through the use of mathematical models and computational tools, seismic behavior of the building is obtained in a suitable way. Among these tools any procedure was chosen: the quadrants method, which leads to the rapid assessment of the seismic capacity of a structure through its non-linear response [10]. Results of the research shown that the current design of this kind of structures is not safe when they are under the maximum seismic actions prescribed by codes, then it is necessary to review the design procedures in order to find more realistic designs that fulfill the goals of the performance-based design.

## 2. Case studied

A two story RC framed building was studied, (Figure 1a), which contains internal staircase and 220 m$^2$ total plan area. This structure represents a common typology used for residential buildings in Venezuela, for prone seismic zones. The structure was designed and detailed for a high ductility value (response reduction factor of 6).

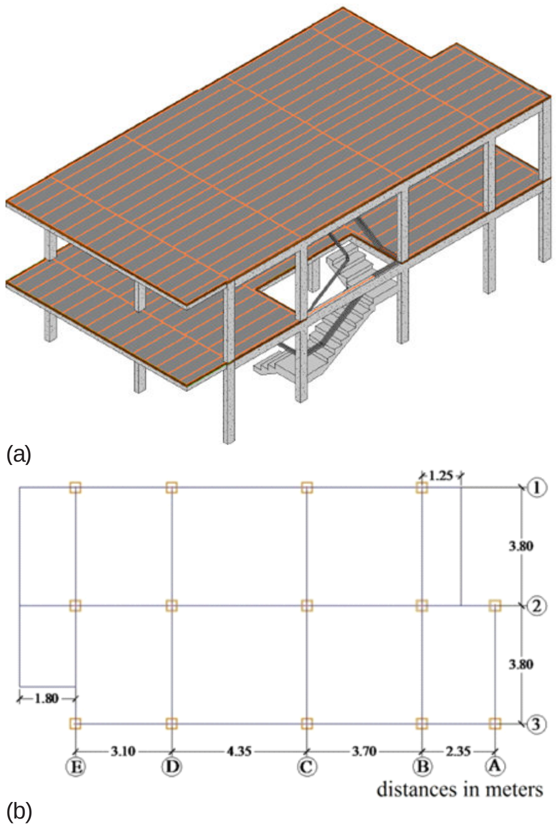


**Figure 1.** Low rise RC building(a) 3D view (b) Plan view

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The building was modeled according its original design, called *original building (OB)*, with plan asymmetry (Figure 1b) and one way 25 cm depth slabs in *X* direction. A second model was designed adjusted to seismic performance requirements formulated by Herrera *et al.* [12], called *resizing building (RB)*, which presents equal geometrics and mechanics characteristics than *OB* model, but considering the "strong column-weak beam" condition. It was also used the displacement-based seismic design procedure of *Priestley et al.* [13] in order to design of a third model, called *displacement-based design building (DBDB)*. These three models differ only in the dimensions of its structural elements (Table 1).

| Building | Axis X beams (cm) | Axis Z beams (cm) | First level columns (cm) | Second level columns (cm) |
|---|---|---|---|---|
| OB | 20x35 | 20x35 | 20x30 | 20x30 |
| RB | 20x45 | 20x35 | 30x30 | 30x30 |
| DBDB | 20x40 | 20x40 | 35x35 | 30x30 |

**Table 1.** Geometric characteristics of elements from each modeled building

## 3. Assessment method

The Quadrants Method is based on the results of the non-linear static analysis (Pushover analysis). This analysis results are plotted in a displacement vs. base shear format, this generate the capacity curve which represents the overall capacity of the whole structure against lateral forces. In order to evaluate the capacity curve two of the main structural parameters are taken into account. The first one is the design elastic shear, obtained from the elastic analysis of the structure using the elastic design spectrum. The second parameter is the threshold that defines the Repairable Limit State, obtained from [14] for RC framed buildings with similar characteristics to the studied ones. The thresholds have been computed from characteristic values of three levels of damage proposed in [15] and are showed in Table 2. Both values are used to define two axes over the capacity curve, the elastic base shear defines an horizontal axis and the damage threshold defines a vertical axis, then Capacity Curve is divided in four spaces or quadrants, see Figure 2.

| Limit State | Damage type | Seismic hazard | Probability event | Inter-storey drift in (%) |
|---|---|---|---|---|
| Service | Non-structural | Frequent | 50% in 50 years | $0,2<\delta<0,5$ |
| Damage control | Moderate structural | Occasional | 10% in 50 years | $0,5<\delta<1,5$ |
| Collapse prevention | Severe structural | Rare | 2% in 50 years | $1,5<\delta<3,0$ |

**Table 2.** Inter-storey drifts adopted for the damage thresholds determination

The performance point is a common procedure accepted among the scientific community to evaluate the seismic performance of a structure under a specific demand. It is usually obtained from the idealized shape of the Capacity Curve as is shown in the Figure 2 [16]. The Quadrants Method also uses this parameter in order to define the roof displacement of the case studied, defined according to the N2 method [5]. If the performance point is under the axis defined by the elastic base shear (Quadrants III or IV), the design does not meet the basic objective of the seismic design because the building does not have enough lateral strength. If the performance point is on the right side of the vertical axis (Quadrant I) means that the building has adequate stiffness, otherwise (Quadrant II) it means that the stiffness is very low and the displacements can be longer than the displacements that can produce advanced structural damage, technically or economically irreparable. These lateral displacements are usually computed from the dynamic response of the structure submitted to a strong motion with a return period of 475 years, or an occurrence probability of 10% in 50 years [17].

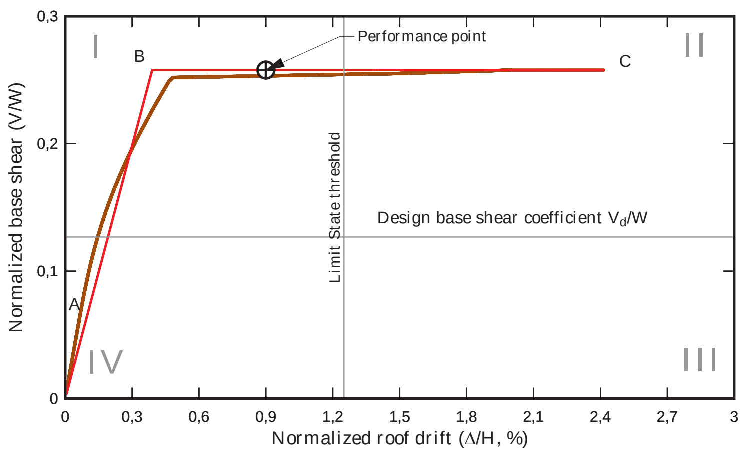


**Figure 2.** Capacity curve and the axis that define the Quadrants Method

The Quadrants Method can provide an objective criterion in order to upgrade the seismic capacity of a structure. If the performance point is on the Quadrant I, the structure has enough lateral strength and stiffness, so does not need to be reinforced. If the structure is on the Quadrant II, it is necessary to provide additional stiffness by using conventional procedures like RC or steel jacketing. If the performance point is on Quadrant III, the structure requires a more radical intervention, adding stiffness and lateral strength. In this case it is possible to combine some traditional reinforcement techniques with new ones like FRP jacketing. In this case the columns are the subject of the main intervention. Finally, if the performance point is on the Quadrant IV, the structure does not has enough lateral strength and then the reinforcement technique must be FRP jacketing.

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## 4. Nonlinear analysis

The structures are modeled by incorporating the structural response when it incurs in the material and geometrical non-linear range, produced by high deformations caused by accidental excitations (earthquakes) [11]. The analyses were performed using ZEUS-NL software [18], which allows to model complex structures with "n" number of finite elements, thus to know the elements in the building which are most vulnerable to damage. Each building is modeled in two dimensions, spitting each frame to get a more detailed response for the seismic behavior of each frame; a 3D dynamic analysis was applied to the ER model.

The static Pushover analysis is performed once the frames have been subjected to action of gravity loads, based on the pseudo-static application of lateral forces equivalent to displacements of seismic action [5]. The pattern of lateral seismic loads consist in increasing loads with height (triangular distribution) applied in a monotonic way until the structure reaches its maximum capacity [20].

This procedure applies a solution of equilibrium equations in an incremental iterative process form. In small increments of linear loads, equilibrium is expressed as:

$$K_t \Delta_x + R_t = \Delta F \tag{1}$$

Where *Kt* is the tangent stiffness matrix, *Rt* is the restorative forces at the beginning of the increased load. These restorative forces are calculated from:

$$R_t = \Sigma\, K_t, K\Delta_u \tag{2}$$

While this procedure is applied, the strength of the structure is evaluated from it is balance internal conditions, updating at each step the tangent stiffness matrix. Unbalanced loads are applied again until it can satisfy a convergence criterion. Then, a new load increase is applied. The increases are applied until a predetermined displacement is reached or until the solution diverges.

From the capacity curve provided in this analysis, it is determined the structural ductility (μ) by the quotient between the ultimate displacement and cadence point displacement, as shown in the following expression:

$$\mu = \Delta_u / \Delta_y \tag{3}$$

Where $\Delta_u$ is Ultimate displacement and $\Delta_y$ is the global yield displacement. Both values are computed from the idealized capacity curve of the structure.

By the other hand, the dynamic analysis is an analysis method that can be used to estimate structural capacity under seismic loads. It provides continuous response of the structural system from elastic range until it reaches collapse. In this method the structure is subjected to one or more seismic records scaled to intensity levels that increase progressively. The maximum values of response are plotted against the intensity of seismic signal [21-22]. The procedure to perform the dynamic analysis from the seismic signal is:

- To define a seismic signal compatible with the design scenario;
- To define the scaled earthquake intensity a monotonic way;
- To define the extent of damage or damage Limit States;
- To study a seismic record for the dynamic analysis of a structural model parameterized to measure earthquake intensity;

The non-linear dynamic analyses provide a set of curves which are a graphical representation of the evolution of the drifts respect time. Results let to compute the damage lumped in specific elements of the structure, but these results are beyond the objective of this Chapter.

| **Analysis earthquake** | **Limit State** | **Return period (years)** | **Occurrence probability in 50 years** | ***Interstorey drift δ (%)*** |
|---|---|---|---|---|
| Frequent | Serviceability | 95 | 50 % | $\delta < 0,5$ |
| Rare | Reparable damage | 475 | 10 % | $\delta < 1,5$ |
| Very Rare | Collapse Prevention | 2475 | 2 % | $\delta < 3,0$ |

**Table 3.** Limit States and seismic hazard level

For the dynamic analysis the structures were subjected to seismic action (see Table 2) defined by accelerograms built on the basis of a likely value of maximum acceleration of the soil and the hazard level associated with the location of the structure and other seismic characteristic design parameters [16]. These accelerograms called "synthetic accelerograms" are generated through the implementation of a set of earthquakes with wide frequency content, using the PACED program [17], based on the Venezuelan code's elastic design spectrum. For the dynamic analyses of the three buildings (*OB*, *RB*, *DBDB*), it were used 3 synthetic accelerograms with duration of 60sec.

Non-linear dynamic analysis was applied to all buildings in order to verify if the performance evaluated by the Quadrants Method is reliable in order to evaluate the fulfilment of the thresholds defined in the precedent section. For this purpose they has been computed three synthetic elastic design spectrum-compatible accelerograms by means of the PACED program [23]. In Figure 3 are shown the Venezuelan rigid-soil elastic design spectrum with the response spectra obtained from the synthetic accelerograms.

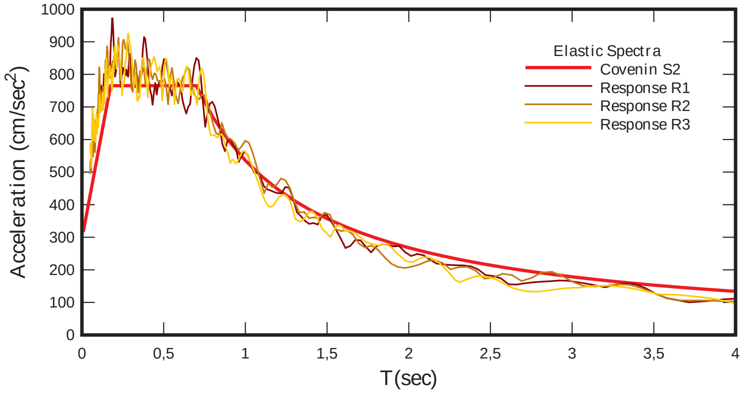


**Figure 3.** Elastic response spectra from elastic design spectrum-compatible accelerograms

These three earthquakes were applied to all frames from the three buildings evaluated, in order to obtain maximum displacement that can be reached by each one. In the software used [18], it was required the implementation of dynamic loads in direction X and the assignation of a control node located in the gravity center of the roof level.

The 3D non-linear dynamic analysis is based on the procedure explained in [20]. The RB building is analyzed, defining its geometry, materials and sections, serviceability loads in Y direction in all beams-columns joints, and dynamic loads on outer nodes with directions and combinations shown in Table 4. One direction ribbed slabs were modeled as rigid diaphragms in its plane by using additional elements with no flexural capacity (Figure 4).

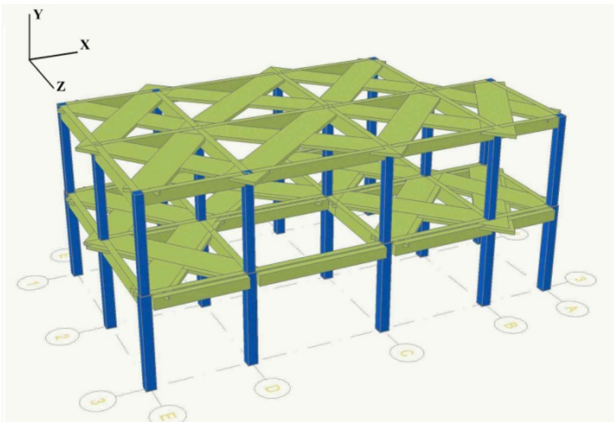


**Figure 4.** Rigid diaphragms in 3D *RB* framed building

Once built the model, there were applied all the accelerograms with the combinations shown in Table 3, for the interstorey drifts and maximum torsional moments on supports. These combinations are based on the Venezuelan seismic code [7] and following established by [24] about the seismic response of asymmetric structural systems in the inelastic range.

| Nº | Seismic combination |
|---|---|
| 1 | 100 % (X) |
| 2 | 100 % (Z) |
| 3 | 100% (X) y 30% (Z) |
| 4 | 100% (Z) y 30% (X) |

**Table 4.** Applied seismic combinations.

## 5. Results

From classic elastic analysis, the verification of interstorey drifts of the *OB* building, shows that they exceed the limit established in [7], while in the *RB* model it was obtained that it meets the code's parameters, which limits the inter storey drift to 0,018. By the other hand, in the *DBDB* building were not performed drifts verifications, since it was designed based on the method performed in [13], where the generated seismic forces are originally limited to not exceed the limit value of drift specified in the applied code.

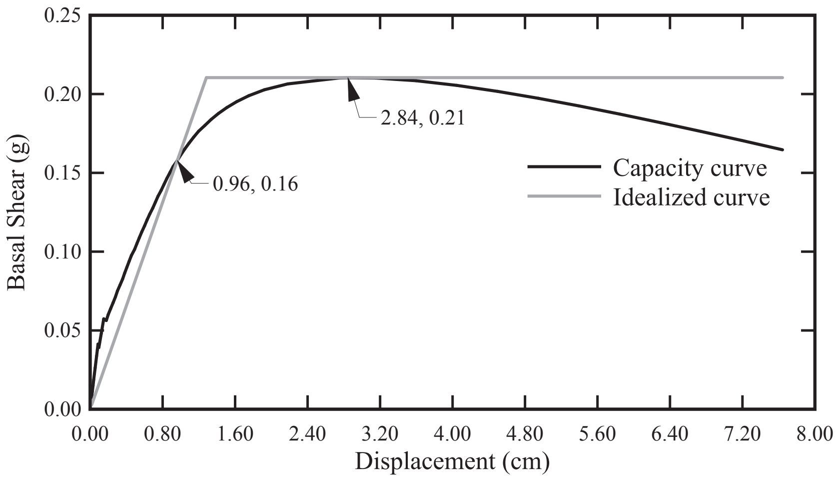


**Figure 5.** Normalized and idealized capacity curves. Frame C. *OB* building

To determine the values of structural ductility it was necessary to plot the idealized curve in function of the capacity curve obtained from non-linear pseudo-static analysis (pushover analysis), in order to know the point at which the structure begins to yield. Figure 5 shows an example of the normalized capacity curve with the idealized (bi-linear) curve of Frame C of *OB*. Structural ductility for each evaluated building values are presented in Table 5. From this Table it is evident that the original building, designed according to current Venezuelan codes has ductility values lower than the redesigned and the displacement-based buildings.

| | **Frame** | **Building** | | |
|---|---|---|---|---|
| | | EO | ER | DBDB |
| Structural Ductility | **A** | 5,56 | 5,52 | 4,77 |
| | **B** | 2,22 | 6,04 | 5,38 |
| | **C** | 2,17 | 4,69 | 5,25 |
| | **D** | 2,21 | 5,54 | 5,59 |
| | **E** | 2,23 | 7,07 | 6,06 |
| | **1** | 2,66 | 5,29 | 6,69 |
| | **2** | 2,20 | 4,17 | 5,92 |
| | **3** | 2,83 | 5,95 | 6,24 |

**Table 5.** Structural ductility results

From the obtained capacity curves there were computed the Performance point (Pp) of every frame of each evaluated building. Table 5 presents the values of Pp of all the frames of evaluated buildings. Figure 6 shows the determination of the Pp of Frame C from *OB* building using the N2 procedure proposed by Fajfar [5].

| **FRAME** | **Pp (cm)** | | |
|---|---|---|---|
| | EO | ER | *DBDB* |
| A | 5,94 | 2,42 | 2,52 |
| B | 13,89 | 9,47 | 7,43 |
| C | 15,22 | 9,50 | 9,38 |
| D | 14,01 | 9,50 | 7,57 |
| E | 13,45 | 9,55 | 6,60 |
| 1 | 12,62 | 9,35 | 6,07 |
| 2 | 15,74 | 11,48 | 9,29 |
| 3 | 10,92 | 7,57 | 4,23 |

**Table 6.** Performance points (Pp) of studied buildings frames

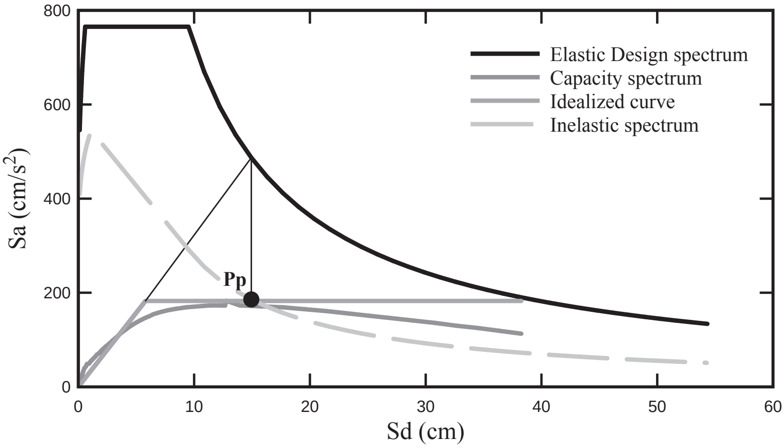


**Figure 6.** Performance point of Frame C. *OB* Building, determined by N2 procedure

From dynamic analyses, there were determined global and interstorey drifts of each frame from all three models studied. Both types of drifts were calculated on the basis of the application of synthetic accelerograms with different intensities, representing the lateral forces applied to frames in order to generate their respective maximum displacements. Figures 7 to 9 show the evolution of the global (Δ/H) drifts expressed as a percentage, respect to time (sec) of the frame C from *OB*, *RB* and *DBDB* models for a peak ground acceleration of 0,3 g, respectively.

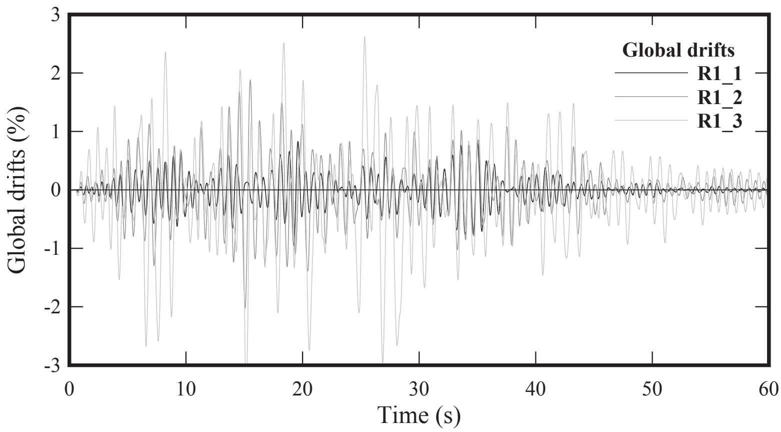


**Figure 7.** Global drifts. R1 earthquake. Frame C. *OB* building.

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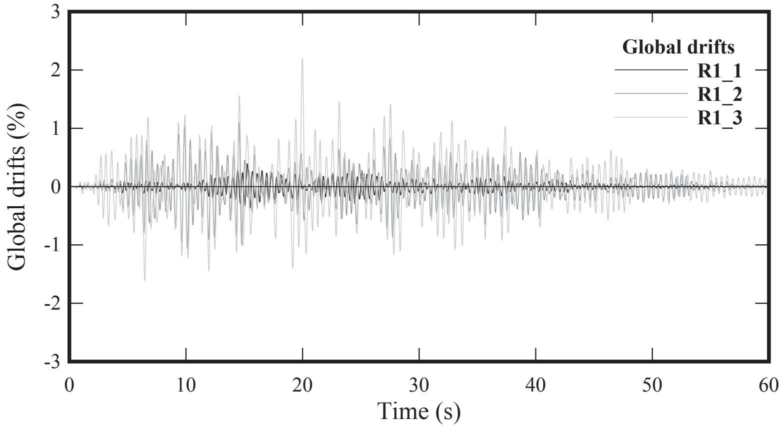


**Figure 8.** Global drifts. R1 earthquake. Frame C. *RB* building

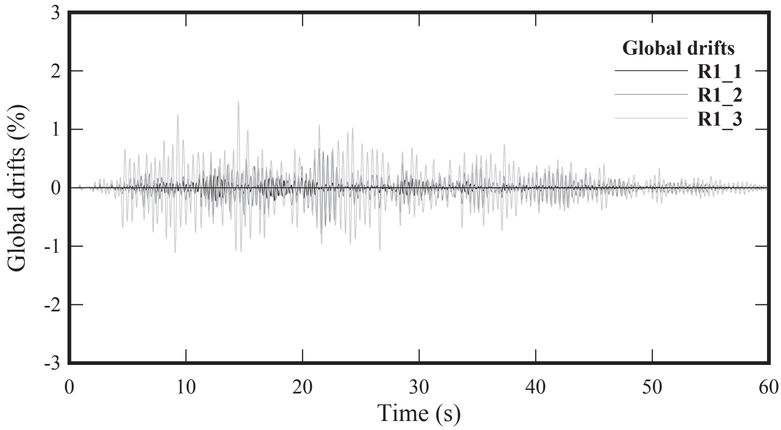


**Figure 9.** Global drifts. R1 earthquake. Frame C. *DBDB* building

Figures 10 to 12 show the results for interstorey drifts of frame C from *OB*, *RB* and *DBDB* buildings, taking into account the R1_3 earthquake with duration of 60 seconds. Similarly, interstorey drifts for applied earthquakes, R1, R2 and R3 with its three intensities, were obtained. It were verified for each Limit State considered in this study. Table 6 reflects the values of interstorey drifts of buildings in study for earthquake R1, taking into account the three levels of hazard, 0,5%, 1,5% and 3%, for the Limits States considered.

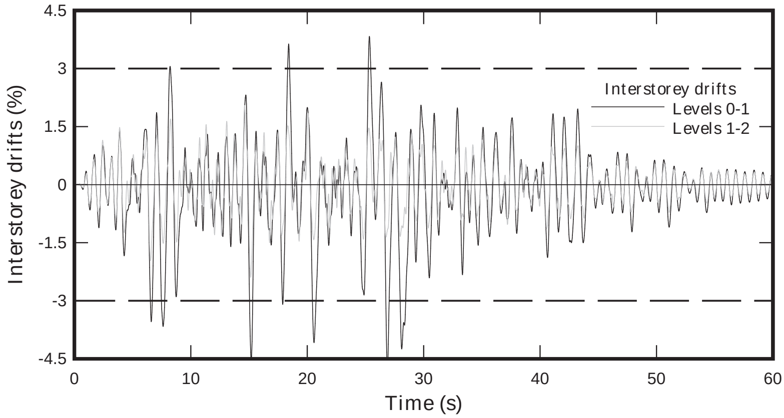


**Figure 10.** Interstorey drifts. R1_3 earthquake. Frame C. *OB* building

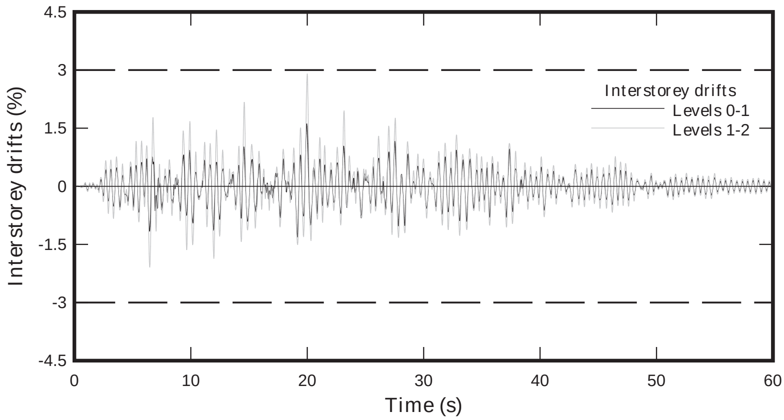


**Figure 11.** Interstorey drifts. R1_3 earthquake. Frame C. *RB* building

## 3D Nonlinear dynamic analysis

Interstorey drifts in frames of *RB*, were obtained by applying the R1_3 earthquake for the combinations 1 and 2 (Table 7). According to results obtained, interstorey drifts in 2D and 3D modeled buildings differ greatly from each other, for this reason it is important to take into account the 3D analysis in order to evaluate the drifts of buildings, because irregularities can produce lateral displacements that does not match with the obtained in 2D analysis.

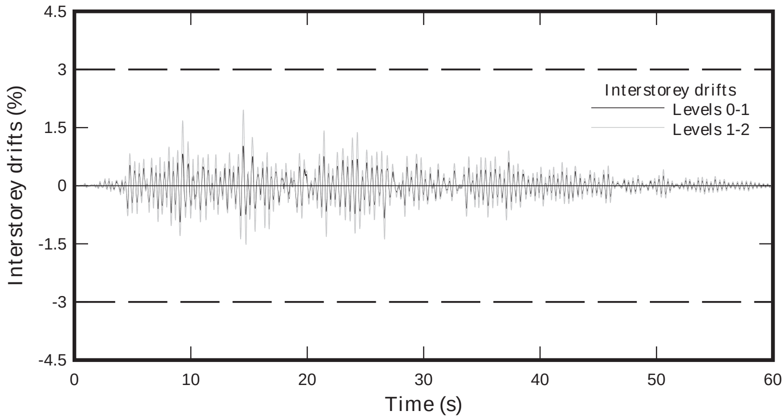


**Figure 12.** Interstorey drifts. R1_3 earthquake. Frame C. *DBDB* building

| FRAME | LIMITS STATES | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | ***OB*** | | | **RB** | | | ***DBDB*** | | |
| | **SLS** | **RDLS** | **PCLS** | **SLS** | **RDLS** | **PCLS** | **SLS** | **RDLS** | **PCLS** |
| A | - | - | - | - | OK | OK | OK | OK | OK |
| B | - | - | - | OK | OK | OK | OK | OK | OK |
| C | - | - | - | OK | OK | OK | OK | OK | OK |
| D | - | - | - | - | OK | OK | OK | OK | OK |
| E | - | - | - | - | OK | OK | OK | OK | OK |
| 1 | - | - | - | - | OK | OK | OK | OK | OK |
| 2 | - | - | - | - | OK | OK | OK | OK | OK |
| 3 | - | - | - | - | OK | OK | OK | OK | OK |

**SLS**: Serviceability Limit State, **RDLS** Reparable damage Limit State, **PCLS**: Prevention of Collapse Limit State; -: No meet the norm,+: Checks the Venezuelan seismic code

**Table 7.** Interstorey drifts verification. R1 earthquake. *OB*, *RB* and *DBDB* building

In the 2D model greater drifts were obtained, while in 3D model the drifts were reduced by the contribution of the diaphragms. Also it were determined the maximum torsional moments in each column before the implementation of R1_3 earthquake in all supports for the four combinations described in Table 8. In Figure 13 have been plotting torsional moments in function of time for the four combinations, where nodes appointed by n111 until the n513 are corresponding to supports, while Figure 14 shows the maximum torsional moment range for each column from three-dimensional analysis. It is evident that for the accelerograms used, the maximum torsional moments occurs in the extreme columns and in the columns located in the intersection of the structure. This is an important feature that confirms the negative effect of the irregularity combined with the seismic action. The torsional moments for the other seismic combinations used in this study were obtained using the same procedure.

| Seismic combination | Node-column Description | Max. Torsional Moment. (Nxm) |
|---|---|---|
| 1 | Corner column. n513 | 64225 |
| 2 | Corner column. n512 | 76000 |
| 3 | Corner column. n513 | 41000 |
| 4 | Corner column. n512 | 65000 |

**Table 8.** Maximum torsional moments for seismic combinations.

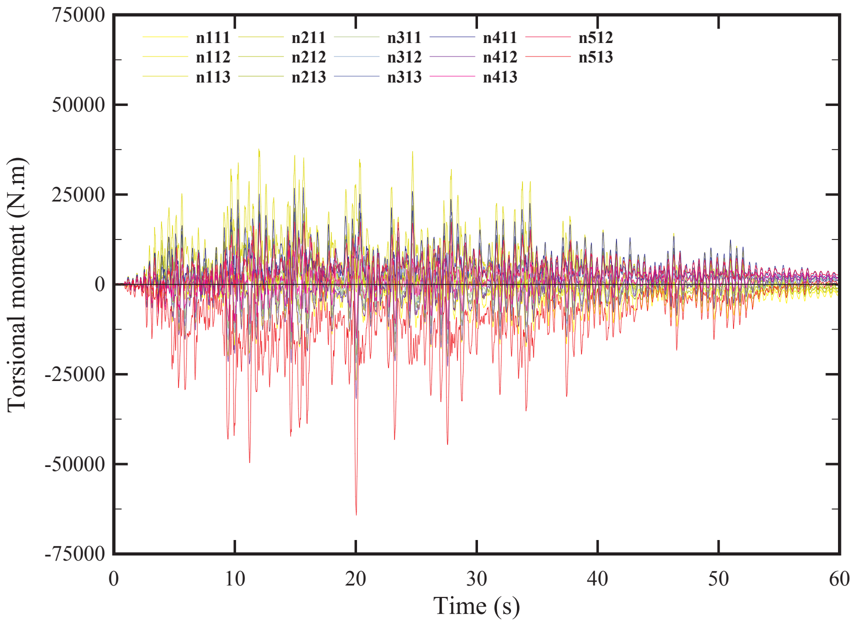


**Figure 13.** Torsional moments for earthquake 100% (X.)

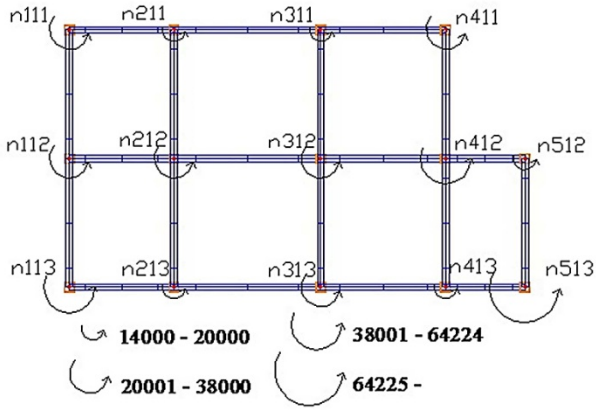


**Figure 14.** Torsional moments for earthquake 100% (X). Plant detail.

## 6. Conclusions

In order to know the seismic response of the studied building it were used analytical methods considering the seismic hazard level and structural regularity criteria. The elastic analysis applied to the *OB* building identified elastic displacements greater than maximum value of interstorey allowed by Venezuelan seismic code. From the resizing model *RB* it was obtained interstorey drifts that satisfied the maximum value established in the code. Thus, the sections of the structural elements of *OB* are insufficient to properly control the damage caused by seismic forces.

From dynamic analysis there were computed the global and interstorey drifts for all three evaluated models determining the dynamic response of these structures and controlling the damage level reached in them. With the global drifts, it was evaluated the threshold of the collapse Limit State, which corresponds to the maximum value of 2.5%. *RB* and *DBDB* buildings reached drifts values below this limit, proving good seismic performance on both buildings; *OB* presented drifts values which exceeded this limit. In the verification of interstorey drifts it was generally noted that interstorey drifts of *OB* building were longer than the considered by hazard levels, while the two resized buildings reached values within the thresholds established for each Limit State.

Three-dimensional dynamic analysis applied to *RB* building allowed determine that interstorey drifts values were under the threshold of the Limit States considered. On the other hand, in order to know the maximum torsional moments for each column in this model, there were applied four seismic combinations where it was noted that there was greater torsion in the case of the component of the earthquake in Z-direction. Based on these results it was demonstrated the structural asymmetry of the assessed building since the center of mass does not coincide with the center of rigidity, determining that the greatest torsional moments are on outer columns and inner corners.

Interstorey drifts of *RB* building obtained from 2D and 3D nonlinear dynamic analysis, it was noted that 2D model provided greater drifts values than the 3D model drifts. This is a logical and expected result since the 3D dynamic analysis considers the rigid diaphragm, which introduces restrictions to the number of degrees of freedom in the structure.

Inelastic static analysis is more reliable than linear methods in the prediction of the parameters of response of buildings, although this method has no response on the effects of higher modes of vibration. A more reliable and sophisticated method is the 2D no linear dynamic analysis, where it can be better determined the likely behavior of the building in response to the earthquake. However, the uncertainties associated with the definition of accelerograms used in these analysis and properties of coplanar structural models can be reduced with the implementation of the dynamic 3D analysis because there are considered factors associated with structural redundancy and are used more actual values in terms of rigidity of resistant structural lines.

The Quadrants Method presented in this paper is suitable for rapid and reliable evaluation of structures, with a low calculation effort. The cases studied demonstrated that the method can provide a reliable criterion to predict if any structure would have an inadequate seismic performance based on the results of static non-linear analysis. Results obtained from dynamic non-linear analysis confirmed the results obtained from the application of the Quadrants Method.

Despite the plan irregularity of the studied building, the Quadrants Method was suitably in order to predict that its lateral stiffness was not enough. Dynamics analysis confirmed this feature, then the cross sections of this building was resizing and details of the confinement were improved in order to meet the regulations of the current version of the Venezuelan seismic code. The seismic performance of the new designed building was tested with the Quadrants Method and dynamic analysis, showing that the resizing structure met all the Limit States used in this research.

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