# Seismic response and torsional effects of RC structure with irregular plant and variations in diaphragms, designed with Venezuelan codes

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## Abstract

The objective of this study is to determine the seismic response and torsional effects of an existing Reinforced Concrete building with irregular plant and five levels projected according to an older version of Venezuelan seismic design code. Two structures were analysed: the original building and a redesigned version. Nonlinear static analysis and nonlinear 3D dynamic analysis were applied, based on registers of three synthetic accelerograms compatible with the elastic design spectrum for the used code. In 3D analysis, four structures were simulated, with and without rigid diaphragms so as to compare the seismic behaviour of the buildings. Through this nonlinear analysis parameters were determined that define the behaviour of the structure, torsional moments and rotations in columns reached for simulated buildings. Also, to obtain damage fragility curves for five states damage were generated. Results show that the original structure has an inadequate resistant behaviour and a high probability of exceeding the moderate damage state, while the redesigned structure presents good performance under seismic events according to the existing code. It was also observed that maximum torsional effects occur in the entrant corners of the irregular plant, which are reduced in mid-rise buildings by using a rigid diaphragm.

*Keywords: RC structure, nonlinear analysis, rigid diaphragm, torsional effects.*

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# 1 Introduction

In Venezuela, a few destructive seismic events have produced severe damages to a significant number of reinforced concrete framed buildings whose plants were characterized predominantly by irregularities like flexible diaphragms, structural discontinuities or L or H shapes, among others. These characteristics contributed in a decisive way to the collapse of buildings designed and built according to old versions of Venezuelan construction and seismic codes. Buildings with high torsional risk and discontinuity in diaphragms collapsed during the earthquake of Cariaco, Venezuela (1997), as it was the case of a commercial building and a school building collapsed during this earthquake (see Figure 1).

Figure 1: Buildings collapsed during the earthquake of Cariaco (1997).

Venezuela is a country of high seismic activity, and an important number of existing buildings are characterized by plant irregularities; many of these buildings are framed low-rise residential structures built during the 1950–1970 period, with deficient seismic provisions and structural detailing.

This document presents a procedure for calculating the fragility curves of this typology of buildings by using methods established in the Risk-UE project, and the proposal of a dynamic 3D analysis that provide values useful for evaluating the effect of torsion to the seismic performance of these structures, indicating columns where the greatest torsional moments occur. The overall results demonstrate very low seismic capacity of the building studied compared with a redesigned model according to the current version of the Venezuelan seismic code.

# 2 Case studied

The structure analysed consists on a low-rise reinforced concrete residential building, built in 1968, characterized by its geometrical irregularity. This

 
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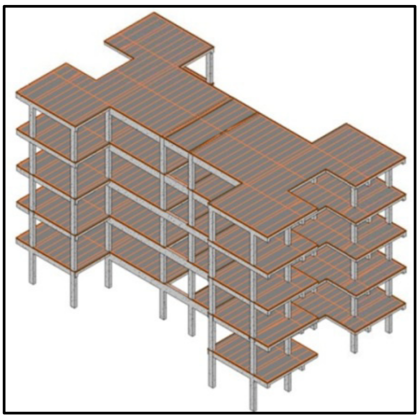
irregularity consists into re-entrants corners in the plan of the structure, see Figure 2a). This characteristic led to classify the building as a plant irregular building, according to ASCE-7 [1], Guevara [2] and Taranath [3]. The use of the building is residential and it represents an interesting case because there are a significant number of similar structures built along Venezuelan cities, see Figure 2b). It has also a irregularity type present in many low rise residential buildings in the country: re-entrants corners that configure L, H or U plan shapes, which are usually selected by architects and structural designers, even though they are not recommended by the current seismic design code Covenin 1756 [4] to zones with a high seismic hazard.



a)

b)

Figure 2: a) 3D view of the building in study; b) Actual building view and satellite view.

The building has six and four structural axes in x and y directions respectively, all of them with different lengths. The building also has five levels of 2.70 m height each. The slabs system consist in one way ribbed slab, which is a typical solution present in this type of building in Venezuela. This characteristic define that the main direction of transmission of service loads is in y axes direction; therefore beams oriented according to x axis are responsible to sustain and transmit service loads to the columns. The other beams are designed for sustain seismic loads during service life of the structure. Those beams have another important characteristic: these are flat beams, not recommended for high seismic hazard level zones (see Figure 3). The stairways are in the circulation core that is located at the middle of the plant, avoiding modifications of the gravity center of each level.

Geometrical properties of structural members in the original building are summarised in Table 1. It is important to note the few steel bars transverse reinforcement, which is less reinforcement than the current version of the seismic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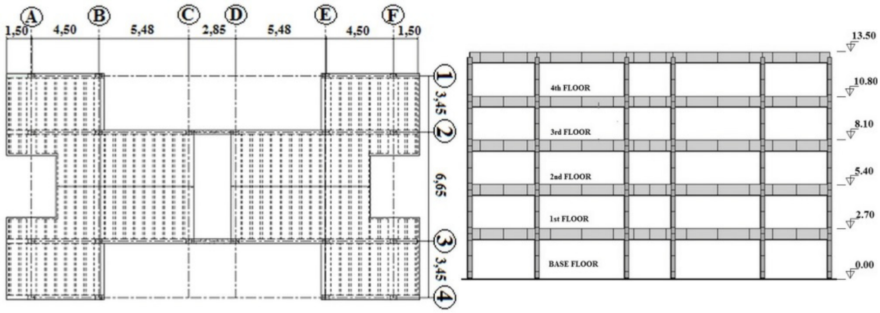


a) b)

Figure 3: Plan and lateral view (m) of the building.

code previsions. Bars used for the transverse reinforcement have diameter lower than the minimum provided by the current version of the seismic code (3/8”). Another issue is the longitudinal reinforcement of the y axes beams, which is greater than the placed in the other direction beams, with no special confinement zones in the original elements.

Table 1: Geometrical details of the structural members of the original building.

| Structural member | Cross section (cm) | Longitudinal reinforcement | Transverse reinforcement |
|---|---|---|---|
| Columns | 25 X 40 | 8 Φ 1/2" | 1 Φ 1/4" @ 0.20 |
| X beams | 25 X 45 | 2 Φ 1/2" y 2 Φ 5/8" | 1 Φ 1/4" @ 0.15 |
| Y beams | 60 X 25 | 4 Φ 5/8" y 4 Φ 3/4" | 1 Φ 1/4" @ 0.15 |

With the purpose of evaluate the performance of the structure designed according to the current version of the Venezuelan code, a new structure model was analyzed. The main difference between the original and the redesigned one is that redesigned model has bigger transversal sections in the structural elements. In Figure 4 it is possible to observe the elastic and inelastic design spectrum for a rigid soil (S2) prescribed by Venezuelan seismic code [4], this spectrum was computed using a response reduction factor R=6. The seismic forces were computed by applying this spectrum for modal periods obtained from modal analysis.

The general procedure includes the verification of the maximum inter-storey drift obtained from the combination of the modal displacements. For this purpose the displacements must be amplified by means of the reduction factor used to calculate the inelastic design spectrum. Inelastic displacements were used to compute the inelastic inter-storey drifts, and then they were compared with the maximum value prescribed by seismic code [4]. Other criteria applied for the design and detailing of the new columns was the verification of the maximum reinforcement ratio, which was not greater than 2.5%, (Vielma *et al*. [5]). Finally

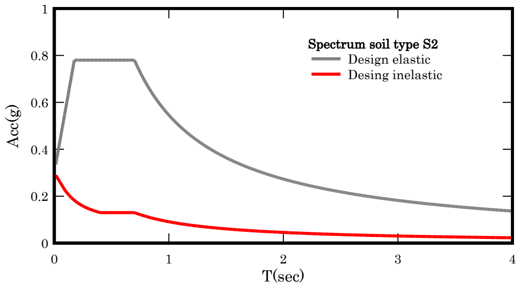


Figure 4: Elastic and inelastic design spectra.

all structural members were checked to satisfy “strong column–weak beam” criterium that characterize a stable behaviour during the occurrence of a strong motion (Fardis [6]). In table 2 it is possible to appreciate the geometrical characteristics of resulting members of the redesigned building.

Table 2: Geometrical details of the structural members of the redesigned building.

| Structural member | Cross section (cm) | Longitudinal reinforcement | Transverse reinforcement |
|---|---|---|---|
| Columns | 60 X 60 | 10 Φ 7/8" | 1 Φ 3/8" @ 0.10 y 1 Φ 3/8" @ 0.13 |
| X beams | 30 x 60 | 6 Φ 7/8" | 1 Φ 3/8" @ 0.14 Y 1 Φ 3/8" @ 0.28 |
| Y beams | 30 X 55 | 8 Φ 7/8" | 1 Φ 3/8" @ 0.13 Y 1 Φ 3/8" @ 0.26 |

This “strong column–weak beam” criteria is a conceptual design procedure introduced in the current version of the Venezuelan reinforced concrete design code Covenin 1753 [7]. Also, there is an important difference in the transverse reinforcement of structural members from the redesigned building because there were considered special confinement zones with greater concentration of shear resistant steel bars to guarantee ductile behaviour and avoiding the fragile failure of the structural members near the columns-beams joints.

## 3 Structural analysis

The structural analyses applied to the studied structures consist in non-linear static and dynamic analyses. The first set of analyses comprises the planar modelization of the structure divided by each frames in each direction. This type of analysis (Pushover analysis) is carrying on applying a set of lateral forces which simulate the seismic actions with a distribution that increases with the height of the building.

The computational tool used to perform these analyses was the Zeus NL software (Elnashai *et al.* [8]). This program is suitable for the assessment of the

 
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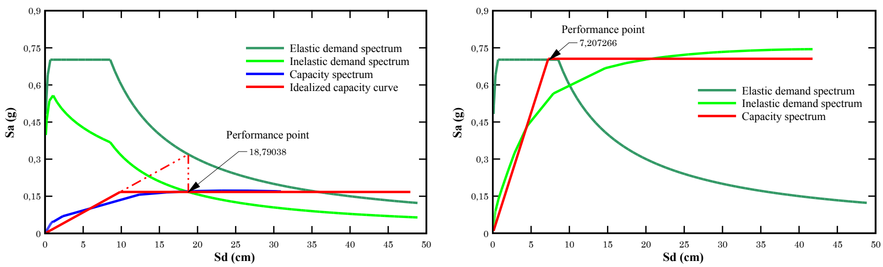
seismic response of complex structures like the case studied. It led to model all the structural members using characteristics obtained from the original design and detailing. The contribution of the transverse reinforcement was obtained using the model formulated by Mander *et al*. [9].

Nonlinear static analysis was performed using a lateral load distribution that corresponds to a first vibration mode shape; these seismic loads were applied after the gravity loads. This distribution of loads produces lateral displacements that were computed and post processed to calculate the global and inter-storey drifts. Results of the analysis were plotted showing the displacement of a control node located on the gravity center of the roof level vs. the base shear force, resulting in the capacity curve or pushover curve.




a) b)

Figure 5: Capacity and demand spectra allowing the determination of the performance point by the method N2 of building, a) original b) redesigned.

From capacity curves obtained from nonlinear static analysis, it is possible to compute a set of structural parameters that characterize the seismic response of the building: global ductility, over strength factor and the response reduction factor. Also, using the method proposed in Fajfar N2 [10], it was determined the “performance point” that represents the condition where the capacity equals demand, which means the maximum response of the structure to seismic action (see Figure 5). Results obtained for the frames of the original and redesigned buildings are shown in Table 3.

Results show that all frames satisfy one basic condition of the seismic design, to possess an over strength value greater than one. This condition is necessary but not sufficient in order to guarantee a structure capable to resist the minimum seismic forces that the code prescribes. Other important parameter is the global ductility, which indicates how the failure mode of the structure is. For the original building some of the values are lightly greater than one, reflecting that the structure would have a fragile failure mode if they are subjected to a strong motion. Note that reduction factors computed of the original frames are lower than the expected in the design process of similar buildings, while the redesigned building’s ones are greater than the factor assumed (R=6). It is evident that the original structure does not have enough displacement capacity to develop a ductile behaviour.

Table 3: Structural parameters computed from non-linear static analysis.

| Frame | Original building | | | | Redesigned building | | | |
|---|---|---|---|---|---|---|---|---|
| | Ductility | Over strength | Reduction factor | Performance point | Ductility | Over strength | Reduction factor | Performance point |
| 1 = 4 | 1.62 | 3.00 | 4.86 | 14.16 | 4.05 | 3.10 | 12.56 | 5.64 |
| 2 = 3 | 1.27 | 1.88 | 2.39 | 18.79 | 3.56 | 3.06 | 10.89 | 7.21 |
| A = F | 1.53 | 3.10 | 4.74 | 15.19 | 3.81 | 4.44 | 16.92 | 3.71 |
| B = E | 2.15 | 2.69 | 5.87 | 15.69 | 3.58 | 5.66 | 20.26 | 3.89 |
| C = D | 3.06 | 2.78 | 8.51 | 19.37 | 2.81 | 3.84 | 10.79 | 5.56 |

Values observed in the spectral shifts corresponding to the performance point of the original building are greater compared to building redesigned, this condition in the nonlinear response of the buildings is considered to relate the performance point and the final displacement, indicating whether the behaviour of a structure is ductile or fragile. Note in Figure 5a the closeness between the performance point and the ultimate shift corresponding to the original building; Figure 5b shows the redesigned building capacity, achieving a ductile behaviour.

In Table 3 the 2D analysis does not include the influence of torsion in the seismic response of the structure. To determine how this effect can be taken into consideration, in the last section of this paper, a new procedure is explained and applied to the case studied.

## 4 Fragility curves and damage

Upon knowing spectral maximum displacement studied for buildings it can be estimated whose frames will suffer significant damages. For this has been developed fragility curves based on spectral methods (Milutinovic and Trendafiloski [11], Barbat *et al.* [12], Lantada *et al.* [13]), considering five damage states: *None, Slight, Moderate, Extensive, Complete*. For each damage state, these cumulative lognormal distribution functions provides the probability of reaching or exceeding a given state of structural damage based on a maximum seismic response. In this case the parameter that defines the maximum response is spectral displacement Sd.

$$F(S_d) = \frac{1}{\beta_{ds} S_d \sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{1}{\beta_{ds}} \ln \frac{S_d}{S_{d,ds}}\right)^2\right] \quad (1)$$

where $S_{d,ds}$ is the mean value of spectral displacement for which the building reaches damage state threshold $d_s$ and $\beta_{ds}$ is the standard deviation of the natural logarithm of spectral displacement for damage state $d_s$. Conditional Probability $P(S_d)$ of reaching or exceeding a particular damage state $d_s$, given the spectral displacement $S_d$, is defined as:

$$P(S_d) = \int_0^{S_d} F(S_d)\, d\, S_d \quad (2)$$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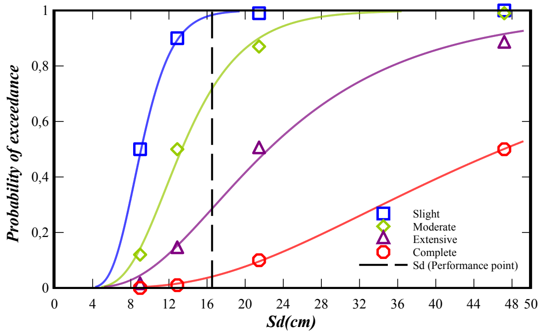
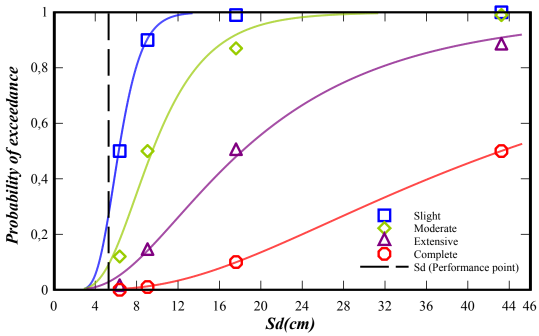
Figures 6a) and 6b) show the fragility curves calculated for the original and redesigned considered in this analysis.




a)




b)

Figure 6: Fragility curves for a) original building, b) redesigned building.

Damage probability matrices are calculated by entering the spectral displacement corresponding to the performance point into the fragility curves. The obtained values represent the probability of exceedance of a damage state and are given in Table 4 for each model studied, building original (OB) or redesigned building (RB). This Table shows that for the considered demand, there is a high probability that OB exceed the moderate damage state. This moderate damage state exceedance probability is 43.5% for OB and 5.8% for RB. It can be also seen that for RB the exceeding probabilities for these damage states are lower, and are expected higher probabilities for slight damage state (see Figure 7).

Table 4: Damage probability matrices (in %) for the buildings.

| Damage state | Original building | Redesigned building |
|---|---|---|
| | *OB* | *RB* |
| None | 1.5 | **74.8** |
| Slight | 26.2 | 16.6 |
| Moderate | **43.5** | 5.8 |
| Extensive | 24.7 | 2.6 |
| Complete | 4.1 | 0.1 |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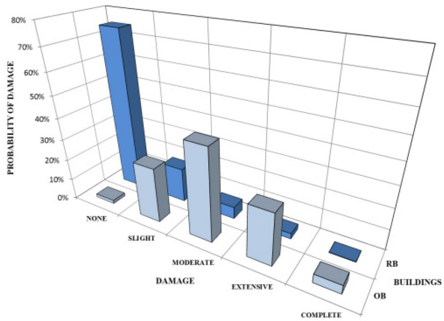


Figure 7: Graph of damage probability matrices.

## 5 Torsional effects

Torsional effects are decisive in the seismic response of in plan irregular buildings, (Fajfar *et al.* [14], Herrera *et al.* [15]). An important number of seismic codes along the world reflect the need to take into account this type of irregularity to avoid unexpected failures induced by torsion, which may affect the outer frame's columns. But there are only qualitative recommendations for the design, sizing or even splitting of the structures, regardless of the irregularity. To evaluate the effect of plant irregularity in concentration of stresses in columns, a 3D model of the original and redesigned buildings was subjected to a set of combinations of dynamic excitations applied at ground level of the buildings.

Buildings were analyzed as a skeleton structure, without the rigidity contributed by masonry walls. By the other hand, the buildings were modeled taken into consideration the contribution of the diaphragm defined by the building's slabs; for this purpose it was necessary to define flat elements that provide lateral stiffness but not add flexural strength. So, it were defined and analysed four 3D structures, two original and redesigned structures without diaphragm, and two original and redesigned structures with diaphragm (see Figure 8).

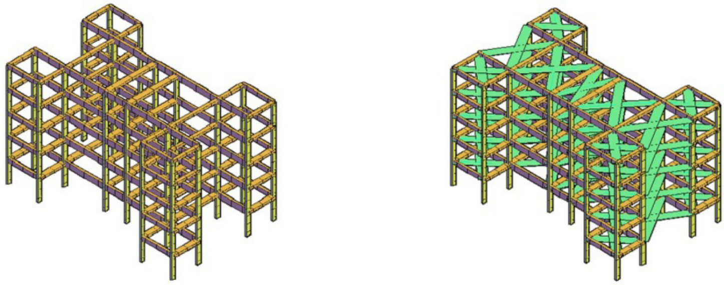

Figure 8: Buildings without diaphragm and with diaphragm.

These analysis were performed using a set of three different synthetic accelerograms compatible with the design spectrum corresponds to a stiff soil type that is the soil type present in the main cities in Venezuela. This set was obtained using the PACED program [16]. Resulting accelerograms have different durations (60 sec, 80 sec and 100 sec) and also have different frequency content (see Figure 10). This procedure was defined to comply with a minimum number of three accelerograms, to include the effect of resonance (Kappos and Stefanidou [17]). Selected combinations consider the directionality prescribed in [4] to perform modal combinations. Nonlinear dynamic analysis was performed for each obtained accelerogram. Table 5 shows these combinations.

Combinations shown in Table 5 are typically prescribed by seismic codes for linear combination of modal analysis results, to calculate the maximum accelerations and displacements of buildings (Elnashai *et al.* [18]). These combinations were applied to dynamic analysis simultaneously, providing results that allow evaluating the torsional moments and rotations in the base of the columns at ground level.

Figure 9 is a plant view which contains a quantitative representation of the torsional moments computed in the base of the columns obtained from the dynamic analyses using the combination 1 of Table 5. Values of these moments have been normalized respect to the maximum value reached. It can be seen that columns of outer axis suffered the maximum torsional moments in comparison with the columns located in the circulation core. This result confirms the presumption of the conceptual design that tries to reduce the stress concentration in those structural members. Columns placed in re-entrant corners have high values too, it allowing concluding that this type of geometrical distribution of the structure could affect the global behaviour of the structure.

Table 5: Combinations of the accelerograms according to their directions.

| ***Combination*** | ***Directions*** |
|---|---|
| **1** | 100% X |
| **2** | 100% X+ 30% Y |
| **3** | 100% X |
| **4** | 100% X+ 30% Y |

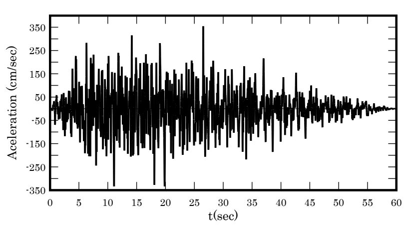


Figure 9: Synthetic accelerogram duration (60 sec).

 
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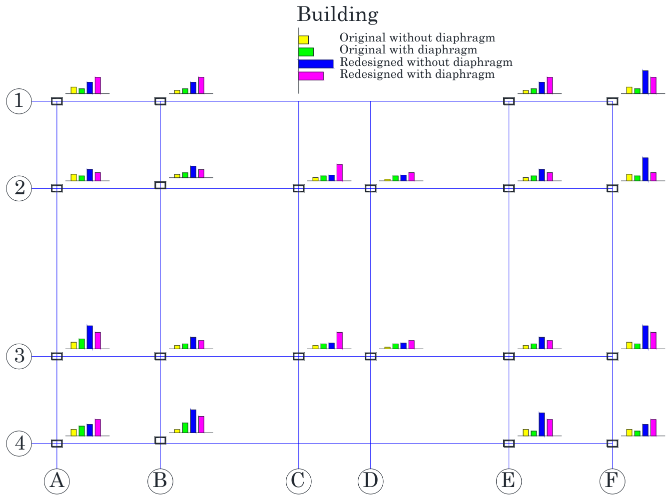


Figure 10: Comparison of the torsor moments computed for the original and redesigned buildings with and without rigid diaphragm.

## 6 Conclusion

Research conducted through this study allow to determine the seismic response and torsional effects of an existing Reinforced Concrete building with irregular plant and five levels which is a typical residential building in Venezuela. Advanced analyses were necessary to assess the seismic response of this type of irregular building. These analyses included static nonlinear analysis and a proposed dynamic nonlinear analysis in 3D; all of these were performed using the advanced computational tool Zeus NL.

Results demonstrated that the expected behaviour of the original structure is unsuitable respect the seismic requirements of the current version of the Venezuelan seismic code. A new structure, called redesigned structure, was also analysed in order to compare the behaviour of the same structure designed according to the current seismic code. The response of this "redesigned structure" satisfies the global design goals. Fragility curves were computed by using an established procedure from the maximum spectral response. It was demonstrated that the original building is likely to reach higher damage states in comparison with the redesigned building.

Finally, it is very useful to extend this type of research and include the results and recommendations in current Venezuelan seismic codes to avoid catastrophic collapses like it has been occurred in buildings with similar irregularities.

## References

[1] ASCE 7, *Minimum Design Loads for Buildings and Other Structures*, American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI). Reston, USA, 2005.

[2] Guevara, T., *Configuraciones urbanas contemporáneas en zonas sísmicas*. Fondo editorial Sidetur. Caracas, Venezuela, 2012.

[3] Taranath, B., *Reinforced concrete design of tall buildings*. CRC Press. Boca Raton, USA, 2010.

[4] COVENIN 1756:01., *Earthquake Resistant Design Code*. Part 1. Caracas, Venezuela, 2001.

[5] Vielma, J. C., Barbat, A. H., Oller, S., Dimensionado sísmico de edificios de hormigón armado mediante factores de amplificación de desplazamientos con base en el balance de energía. *Hormigón y acero*. **63(263)**, 83–96. Madrid, Spain, 2012.

[6] Fardis, N. M., *Seismic design, assessment and retrofitting of concrete buildings*. Springer Verlag. Heilderberg, Germany, 2009.

[7] COVENIN 1753:06, *Design and construction of buildings with structural concrete*. Caracas, Venezuela, 2006.

[8] Elnashai A., Papanilolau V., Lee, D., ZEUS-NL, A system for Inelastic Analysis of Structures, User Manual. Mid-America Earthquake Center report no. MAE, Illinois University. Urban, Champagne, Illinois, 2011.

[9] Mander JB, Priestley MJN, Park R., Observed stress-strain behaviour of confined concrete. *J Struct Eng* (ASCE) 114, pp. 1827–1849, 1988.

[10] Fajfar, P., Nonlinear analysis method for performance based seismic design. *Earthquake Spectra*, EERI, USA, **16(3)**, 573–591, 2000.

[11] Milutinovic, Z., Trendafiloski, G., RISK-UE. An advanced approach to earthquake risk scenarios with applications to different European towns. WP4: Vulnerability of current buildings. RISK-UE – EVK4-CT-2000-00014, 2003.

[12] Barbat, A. H., Pujades, L. G., Lantada, N., Seismic damage evaluation in urban areas using the capacity spectrum method: application to Barcelona. *Soil Dyn Earthquake Eng*, **28**, 851–865, 2008.

[13] Lantada, N., Pujades, L. G., Barbat, A. H., Vulnerability index and capacity spectrum based methods for urban seismic risk evaluation. A comparison. *Nat Hazards*, **51**, pp. 501–524, 2009.

[14] Fajfar, P., M Marusic, D., Perus, I., Torsional effects in the pushover-based Seismic analysis of buildings, *Journal of Earthquake Engineering*, 9(6), pp. 831–854, 2005.

[15] Herrera, R., Vielma, J., Ugel, R., Martínez, Y. and Barbat, A., Optimal design and earthquake-resistant design evaluation of low-rise framed RC structure. *Natural Science*, **4(8a)**, pp. 677-685, California, USA, 2012.

[16] Vielma, J. C., PACED: Program for the generation of spectrum compatible accelerograms. Lisandro Alvarado University. Barquisimeto, Venezuela, 2009.

[17] Kappos, A., Stefanidou, S., A deformation-based seismic design method for 3D R/C irregular buildings using inelastic dynamic analysis. *Bulletin of Earthquake Engineering*, Springer. **8(4)**, pp. 875–895, 2010.

[18] Elnashai, A., Di Sarno, L., *Fundamentals of earthquake engineering*. John Wiley and Sons, Chichester, United Kingdom, 2008.