SEISMIC RESPONSE OF REINFORCED CONCRETE BUILDINGS IN CARACAS, VENEZUELA

Safina Salvador[1], **Andrade Mónica**[2], **Schmitz Michael**[2]
Jraige Cristian[1], **Espinosa Luis**[1]

[1] *SISMOMETRIC, C.A. – Seismic Safety, Caracas, Venezuela*
[2] *FUNVISIS - Venezuelan Seismological Research Foundation, Caracas, Venezuela*
Email: safinasal@gmail.com

ABSTRACT :
As part of the Seismic Microzonation Project of the Caracas city, coordinated by the Venezuelan Seismological Research Foundation (*FUNVISIS*), a study on the seismic response of nine (9) real moment-resisting reinforced concrete frame buildings located in Caracas – Venezuela, has been developed. The selected buildings includes low (3-4 stories), medium (6-8 stories) and high-rise (13-17 stories) RC frame structures, designed according to the different Venezuelan Standard Construction, i.e. buildings designed before 1967 year (Pre-67), between 1967 and 1982 year (67-82) and buildings designed after 1982 year (Post-82). Each building was modeled using a computer 3D model with SAP2000, taking into account dimensions and reinforcement details of the primary system components (columns and beams), and then subjected to simplified nonlinear analysis to evaluate seismic performance. Using Pushover analysis, the capacity curve is obtained for each analysis direction of the model, then different physical damage levels are defined and associated with Peak Ground Acceleration (PGA) throug ATC-40 simplified procedures. The seismic hazard is characterized using a typical site acceleration spectrum. Based on analysis results, preliminary fragility curves are generated for each building and each analysis direction, showing the significant vulnerability of those buildings constructed before 1967 (Pre-67), specially high-rise buildings, which performance is unacceptable for hazard levels expected in Caracas city . The fragility curves also show a good performance, regardless of height, for buildings constructed after 1982 (Post-82), while the performance of buildings constructed between 1967 and 1982 (67-82) do depend on building height, showing that low-rise buildings perform better than medium and high-rise buildings, the last ones revealing seismic strength below normative according to current Venezuelan seismic code.

KEYWORDS:
Pushover analysis, Capacity curve, Simplified nonlinear analysis, Fragility curves, Seismic vulnerability

1. BUILDINGS DESCRIPTION

A study on the seismic response of nine (9) real moment-resisting reinforced concrete frame buildings located in Caracas – Venezuela, has been developed as part of the Seismic Microzonation Project of the Caracas city (Schmitz et al., 2006). The selected buildings includes low (3-4 stories), medium (6-8 stories) and high-rise (13-17 stories) RC frame structures, designed according to the different Venezuelan Standard Construction, i.e. buildings designed before 1967 year (Pre-67) used MOP(1955), buildings designed between 1967 and 1982 year (67-82) used MOP(1967) and buildings designed after 1982 year (Post-82) used Covenin(1982), as table 1 shows. Each building was modeled using a computer 3D model, taking into account dimensions and reinforcement details of the primary system components (columns and beams), and then subjected to simplified nonlinear analysis to evaluate seismic performance (FEMA 440, 2005).

Table 1. Models Identification

	Pre-**67**	**67-82**	Post-**82**
	MOP(1955)	*MOP(1967)*	*Covenin (1982)*
Low-Rise	*L67*	*L82*	*L98*
Medium-Rise	*M67*	*M82*	*M98*
High-Rise	*H67*	*H82*	*H98*

2. STRUCTURAL MODELING

Each building was modeled using a computer 3D model with SAP2000 v.10 (SAP2000, 2005), taking into account the most relevant arquitectural and structural features, specially those related to beams and columns reinforcement details, in order to estimate the strength and ductile capacity of each element for nonlinear analysis purposes. Beams and columns was modeled as framing type elements and beam-column joints were considered rigid along the whole intersection between frame elements. Floors are considered as rigid diaphragms were aplicable and structure supports are totally restrained without considering soil-structure interaction and the effect of other nonstructural elements (masonry-infilled, walls, stairs and others) in the structural response. Beams are uniformly loaded considering 100% dead load and 25% live load.

Inelastic behavior was modeled using discrete hinges at the end of frame elements. Two types of hinges are used for the models: Moment hinge for beams and PMM interaction hinge for columns.

Beam moment hinges are modeled using a moment-rotation relation following ATC-40 (1996) recomendations as figure 1 shows, wherein A, B, C, D and E are characteristics points totally describing the mentioned curve moment-rotation and IO, LS and CP are rotation limits associated with performance levels defined in ATC-40 as IO: Immediate Occupancy, LS: Life Safety and CP: Collapse Prevention. Based on available reliable data, each of this points (A,B,C,D and E) was rationally defined according the building design year as shown in table 2 and table 3 shows the adoptted values for the corresponding rotation limits IO, LS and CP.

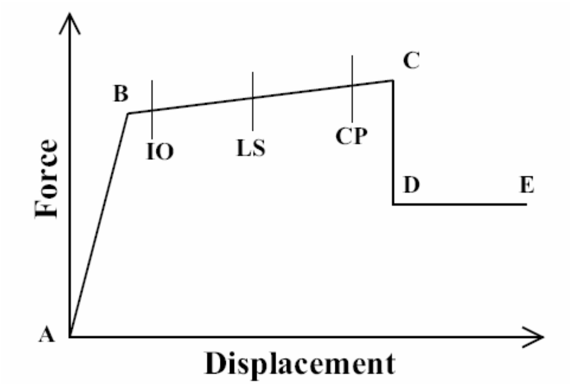

Figure 1. ATC-40 hinge typical curve

Table 2. Moment-rotation values according to building construction year

	B		C		D		E	
	M/My	θ/θy	M/My	θ/θy	M/My	θ/θy	M/My	θ/θy
Post-82	1.00	1	1.25	6	0.20	6	0.20	8
67-82	1.00	1	1.15	4	0.20	4	0.20	6
Pre-67	1.00	1	1.10	2	0.20	2	0.20	4

Table 3. Rotation limits values adoptted for the different performance levels defined in ATC-40

θ/θy	IO	LS	CP
Post-82	2	4	6
67-82	1.5	3	4
Pre-67	1.1	1.5	2

Column hinges (PMM) are default type and automatically generated by SAP2000 taking into account the distribution of longitudinal steel along the column section, axial load level, acting shear force and the level of confinement provided by the transverse reinforcement. Following proposed FEMA 356 (2000) plastic rotation values and recomendations, two types of post-yield column behavior are established in terms of two limits in the axial load level (0.10Ag f'c and 0.40Ag f'c) and the transverse reinforcement is classified as non-conforming for those buildings constructed before 1982 year and conforming for the ones constructed later.

3. ANALYSIS METHODOLOGY

For each model and for the two main directions of analysis a capacity curve is obtained from a "pushover" analysis. Figure 2 show a H82 model representation (17 stories) and capacity curves for X and Y directions. The lateral load pattern is according to the first mode in the analysis direction considered. Following FEMA 356(2000) recomendations, a bi-linear representation of the capacity curve is constructed so that the yield base shear and displacement (Vy, δy) in the top of the building can be estimated. The structure is pushed until collapse displacement (δu) occurred. These displacements can be used to estimate the building global ductility. The ratio between the mentioned displacements and the height of the building constitute estimates of the global drift which can be associated to damage levels and as a guide to establish levels of performance.

Damage States (DS) are defined as follows: *No Damage* for those buildings subjected to elastic displacements (displacements smaller than yielding displacement δy), *Light Damage* for displacements slightly above δy, *Colapse* for the maximum displacement δu and finally, two intermediate equally spaced damage states between yielding and collapse defined as *Moderate* and *Severe Damage*, respectively. The capacity-demand spectrum method proposed in ATC-40 (1996) and based in the equivalent linearization of non-linear response of the system is used to correlate demand and damage states, so that for a particular spectrum it is posible to estimate the peak ground acceleration (PGA) required to reach a target displacement or drift associated to a particular damage state.

Three different site class spectrum named S1, S2, S3 are defined in the venezuelan seismic code COVENIN 1756-1:2001 (Covenin, 2001) and used for the analysis. Finally, in order to implement the ATC-40 method a structural behavior type B is established for those structures constructed after 1982 year and a type C for those constructed before.

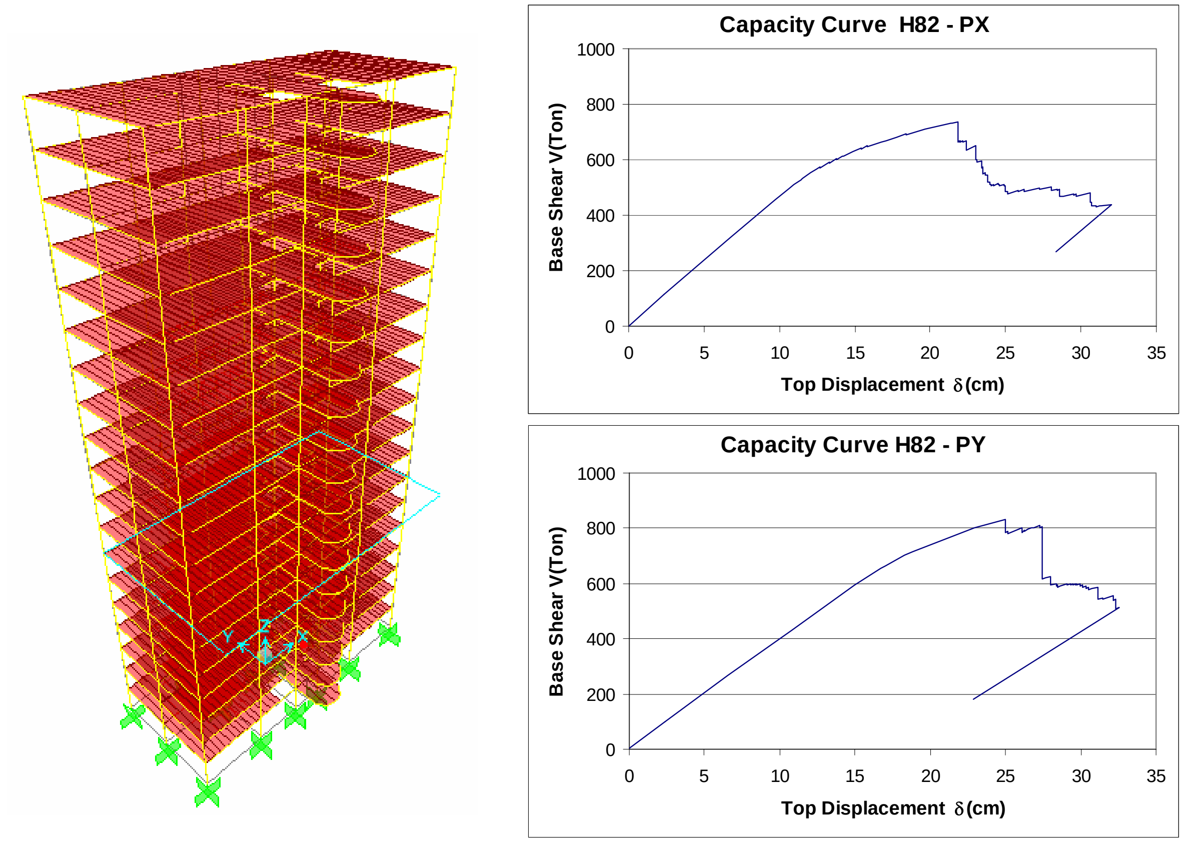

Figure 2. H82 model - Graphic representation and capacity curves for X and Y directions

4. RESULTS

For each model and each analysis direction, peak ground acceleration values associated to the different damage states are summarized in tables like table 4, which shows results corresponding to H82 model for the X direction and S2 spectrum type (Safina et al., 2007). In particular this table shows the seismic weight and height of the building (W, H), the period of vibration of the first mode in the direction of analysis (To) and the structural damping (βo). Aditionally the table shows for each damage state (light, moderate, severe and collapse) the corresponding top displacement of the building (δ), global drift (δ/H), base shear (V), base shear - building weight ratio (V/W), peak ground acceleration (PGA), effective period (Teff) and equivalent damping coefficient (βeff).

Table 4. H82 model results – X Direction – S2 Spectrum type

W (Ton) = 7268 To(seg)= 1.82
H (m) = 53.20 βo = 0.05

DS	δ (cm)	δ/H (%)	V (Ton)	V/W	PGA (g)	Teff (seg)	βeff
Light	13.20	0.25	592	0.08	0.12	1.90	0.060
Moderate	16.10	0.30	654	0.09	0.15	2.02	0.078
Severe	19.00	0.36	702	0.10	0.17	2.09	0.086
Collapse	21.90	0.41	740	0.11	0.19	2.17	0.100

Results obtained allow the generation of preliminary fragility curves for each model and each direction of analysis. Although results correspond only to a particular case and the absence of results statistics of similar buildings, the generation of the fragility curves is based on the following: (i) Peak ground accelerations obtained are mean values ($P\overline{G}A$), (ii) A log-normal distribution of probabilities $\Phi[.]$ is assumed in order to describe the fragility curves (Eqn 4.1), (iii) Considering the lack of analysis results required for the estimation of the dispersion of the PGA mean values (β_{PGA}), dispersion values obtained from other studies of similar buildings (Bonett, 2003) are used in this research and summarized in table 5.

$$FD = \Phi\left[\frac{1}{\beta_{PGA}}\ln\left(\frac{PGA}{P\overline{G}A}\right)\right] \tag{4.1}$$

$P\overline{G}A$ - Mean value of peak ground acceleration
β_{PGA} - *PGA* Standard deviation
$\Phi[.]$ - Log-normal distribution of probabilities
FD = P[DS≥DSi|PGA] – Probability for a damage state DS to be bigger than a limit damage state DSi conditioned to a peak ground acceleration PGA.

Table 5. PGA Standard deviation values (β_{PGA}) used in this research

	Light	Moderate	Severe	Collapse
Post-82	0.25	0.30	0.35	0.35
67-82	0.30	0.35	0.40	0.45
Pre-67	0.35	0.40	0.45	0.50

These preliminary fragility curves allow the characterization of the seismic vulnerability of each building modeled and the relative vulnerability between them. Figure 3 show H82 model fragility curve for X direction and S2 Spectrum type. Figure 4 show a comparison of fragility curves associated to severe damage state for S2 Spectrum type.

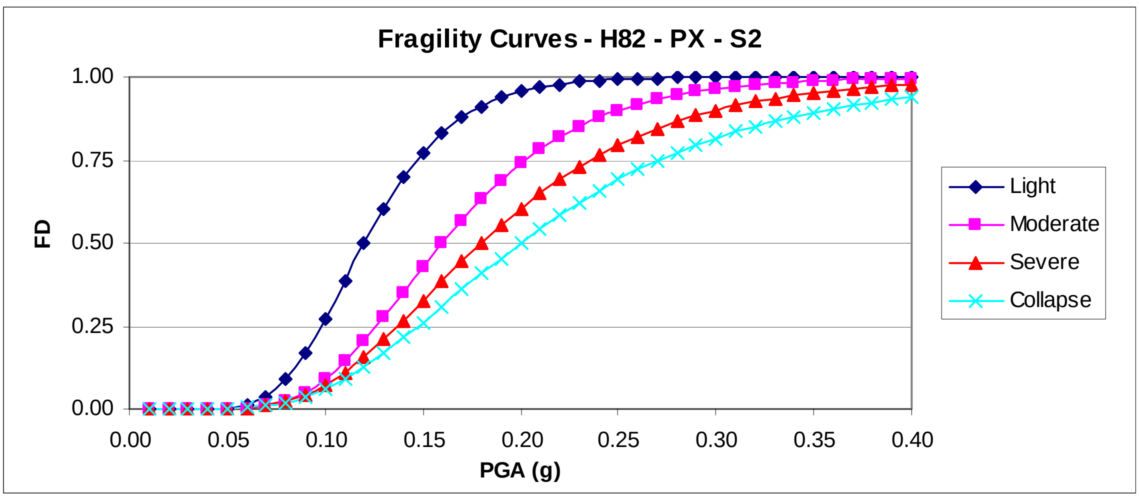

Figure 3. H82 model fragility curve – X direction – S2 Spectrum type

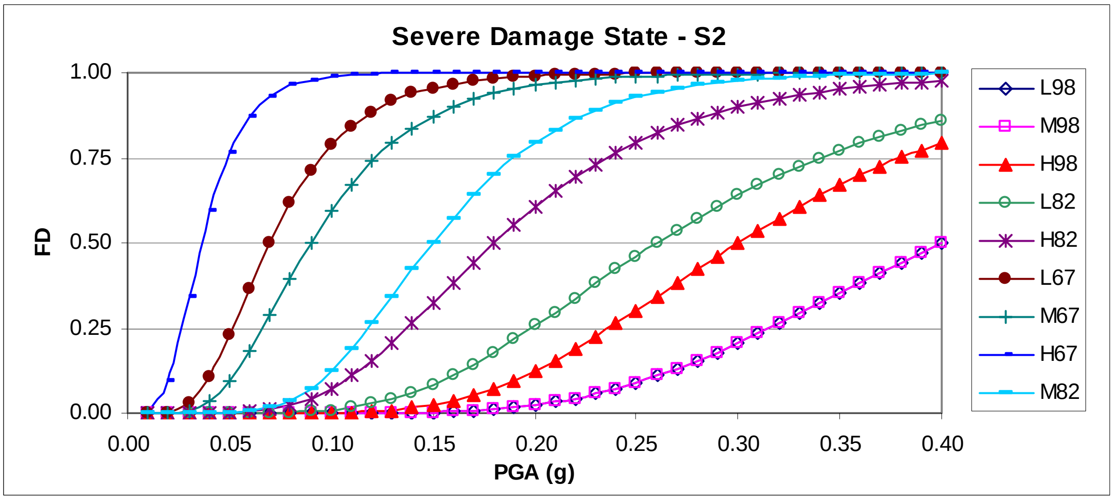

Figure 4. Fragility curves for severe damage state - S2 Spectrum type

5. CONCLUSION

The complexity of assess seismic performance of existing buildings is confirmed again in this study, wherein analysis results were obtained from a set of suppositions or hypothesis and simplified assessment models. However, the systematic use of this simplifying hypothesis for the analysis of the buildings under study make possible the visualization of typical pattern of seismic response, classification of seismic performance and hierarquization of relative vulnerability.

Results obtained point out a significant vulnerability in the buildings constructed before 1967 year (Pre-67), which becomes worse the taller the building is. Buildings constructed before 1967 year shows mean peak ground acceleration values associated to a severe damage state below PGA design values specified for Caracas city (PGA=0.30g) and preliminary fragility curves show high probabilities of reaching significant damage for low ground accelerations associated to frequent earthquakes.

In opposition to buildings constructed before 1967 year, results show that those buildings constructed after 1982 year (Post-82) require ground accelerations bigger than design acceleration for Caracas city to reach a severe damage state. In particular, H98, M98 and L98 show base shear – building weight ratios between 10% and 13%, 13%-18% and 18% - 30% respectiveley, and bigger deformation capacity.

Finally, behavior of buildings constructed between 1967 and 1982 year (67-82) depends on the building height. Results show that low-rise buildings have adequate lateral strength for PGA close to design acceleration for Caracas city, while medium and high-rise buildings show a level of lateral strength below normative for severe or collapse damage states and ground accelerations between 0.20g and 0.25g.

Comparison of fragility curves associated to each damage state show that in all cases fragility curves for buildings constructed before 1967 year are always located at the left of the rest of curves, which means that this group of buildings (Pre-67) require low ground acceleration values to reach a specific damage state, which make these buildings more vulnerable than others. Post-82 buildings fragility curves are located at the extreme right of the comparison graphic (figure 4) meaning that these buildings are the ones with the lowest vulnerability and fragility curves for those buildings constructed between 1967 and 1982 year are located at the center of the graphic implying an intermediate vulnerability. For a same period of construction, fragility curves tend to locate at the left of the graphic when building height increases, which means that the high-rise buildings are the most vulnerable ones.

ACKNOWLEDGEMENT

This research was funded by the FONACIT of the Ministry of Science and Technology, under Project No. 200400738. The support of FUNVISIS is appreciated.

REFERENCES

ATC-40 (1996). Seismic evaluation and retrofit of concrete building. Volume 1. Applied Technology Council. Redwood City. California. USA. November 1996.

Bonett R. (2003). Vulnerabilidad y riesgo sísmico de edificios. Aplicación a entornos urbanos en zonas de amenaza alta y moderada. Tesis Doctoral. Universidad Politécnica de Cataluña UPC. Barcelona. España. Octubre 2003. (In Spanish)

Covenin (1982). Norma venezolana para edificaciones antisísmicas. Covenin 1756-82. Ministerio del Desarrollo Urbano. Fondonorma. Caracas. Venezuela. (In Spanish)

Covenin (2001). Norma venezolana para edificaciones sismorresistentes. Requisitos y Comentarios. Covenin 1756-1:2001. Ministerio de infraestructuras. Fondonorma. Caracas. Venezuela. (In Spanish)

FEMA 356 (2000). Prestandard and commentary for the seismic rehabilitation building. Prepared by American Society of Civil Engineers ASCE. Reston. Virginia. USA. November 2000.

FEMA 440 (2005). Improvement of nonlinear static seismic analysis procedures. Prepared by Applied Technology Council (ATC-55). Redwood City. California. USA.

MOP (1955). Norma para el cálculo de edificios. Ministerio de Obras Públicas.. Caracas. Venezuela. 1955. (In Spanish)

MOP (1967). Norma provisional para construcciones antisísmicas. Ministerio de Obras Públicas. Dirección de edificios. Caracas. Venezuela. Noviembre 1967. (In Spanish)

Safina S., Jraige C., Espinosa L., Andrade M. (2007). Estudio de la respuesta sísmica de edificaciones reales de Caracas. Informe Técnico para Funvisis. Mayo 2007. (In Spanish)

SAP2000 (2005). SAP2000 – CSI Analysis reference manual. Computer and Structures, Inc. Berkeley. California. USA. October 2005.

Schmitz M., Hernandez J.J., Audemard F., Malavé G., González, Granado C., Sánchez J., Andrade L. y Klarica S. (2006). Avance de los proyectos de microzonificación sísmica en las principales ciudades de Venezuela. VIII Congreso Venezolano de Sismología e Ingeniería Sísmica, Valencia, Venezuela, 2006. Memorias en CD. (In Spanish)