Assessment of the energy dissipation capacity of Steel buildings with eccentrically braces

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SUMMARY:
The design of steel-frame buildings with eccentric braces can provide reasonable buildings solutions for developing countries. Therefore, it is important to know the earthquake-resistant capacity of buildings with eccentric braces and compare it with the capacity of normal frames, concentrically braced frames and concentrically frames configuration in "V". In this study, "V" shaped eccentric braces are used, with horizontal links at beam level or vertical links perpendicular to the central beam. Six study cases are analyzed on building structures of 4, 6, 8 and 10 stories, with square plants and three six-meters spans. Braces are located in the central span of each external frame around the perimeter in order to improve the torsional strength and obtain one noninvasive condition. Lateral drifts, shear forces, response factor R, overstrenght, performance points, ductility, and dissipated energy are determined in the 24 cases analyzed. Pseudo-static and dynamic analyses were performed with simulated acceleration histories. The study provides a better understanding the benefits of eccentrically braced systems.

Keywords:Seismic assessment, nonlinear analysis, braces, eccentrically braces.

1. INTRODUCTION

It is necessary to design economical steel building with some conditions to obtain energy dissipation under seismic actions in developing countries, because there are little possibilities to use base isolated or energy dissipaters, and it is convenient to have other practical solutions to design new structures or the rehabilitation of existing buildings. The eccentrically braced frames (EBF) can be one viable solution to obtain sufficient strength and stiffness on steel buildings, if a desirable link length and web stiffening are used (Popov et al, 1987). Afterward, the research works recommended to apply "capacity design" to assure the better design of links, beams and braced in EBF systems, where it is stablished that the "inelastic activity under severe cyclic loading is restricted primarily to the links". Also, the elements must be designed for the maximum forced including the link overstrength, that can be attributed to strain hardening, effects of composite floor systems and actual yield strength of steel (Engelhardt M. D. and Popov E. P., 1989).

When the buckling of braces it is present, the eccentrically braced frames (EBF) and the concentrically braced frames (CBF) can no to develop energy dissipation, and to design an alternative approach buckling restrained brace (BRB) called "unbounded brace, which concept it is not in this paper (Uang Ch-M. and Nakashima M., 2004). In some papers it is convenient to make the capacity curve pushover of frames with braced building and take into account the performance levels: immediate operation and life safety, and to obtain the target value of period and the energy dissipation (Teran-Gilmore and Virto-Cambray, 2010). In other paper many braced frames with different configurations were studied and the factors of response, ductility and overstrength were derived from capacity curves obtained using nonlinear static analysis (P. Shademan H. et al, 2010). In the pushover curve, we obtain the ductility factor with the yield displacement and ultimate displacement, and the response factor R can be performed by strength demand reduction factor, and the overstrength factor using the following expressions (Shademan et al, 2010):

$$Rd = \text{Elastic Strength Demand} / \text{Real Strength} \qquad (1)$$

Ω = Real Strength / Design Strength (2)

R = Rd . Ω (3)

These equations has been also utilized to know the restraint of the lateral displacement of buildings on concentrically and eccentrically braces in X, V and Y shapes, and for to obtain the R factor (Colasante Pérez G. M., 2011). The influence of the location of braces within the perimeter of buildings has been studied, and gave innovative configurations, creative layout and irregular morphologies (Mezzi M., 2010).

Taking into account the energy absorbed or work done by the external forces and the energy - displacement capacity represented on pushover curves, it is possible to obtain a performance point of energy or displacement demand by intercepting the curves of energy absorbed and the demand. That is an important method based on balance of energy to obtain the energy dissipation capacity (Leelataviwat et al, 2008).

The objectives of this paper are the following:

- To solve different models of buildings with concentrically (CBF) and eccentrically braces (EBF) of 4, 6, 8 and 10 stories. These cases are to buildings on moderated heigh according to Covenin Code that accept to consider concentrically braces for over five stories buildings.
- The dimensioning of the elements correspond to the normal steel building frame without braces and the changes will be imposed by the braces.
- The braces are located in the central span of each external frame around the perimeter, in order to improve torsional strength and stiffness and to obtain one noninvasive condition.
- To design the buildings attending the Seismic Covenin Code of Venezuela (1756-2001) and the Steel Structures Covenin Code of Venezuela (1618-1998).
- The other conditions are: Seismic zone 5, Ao = 0.30g, Soil S1 (hard), Importance factor α =1, Group B2 Housing, Structural type I (frame), level of design ND3 (maximum), regular structure. The spectra are shown in Fig. 1.
- To solve elastic modal analysis, nonlinear static analysis and dynamic analysis to obtain the significantly effects of the braces in the structural response.
- To take into account the stiffeners spacing and the strain hardening for the design of the capacity of links to sustain cyclic loads.
- To show the responses in values, factors, rotation angles, displacements and graphics; and discuss it, to have a knowledge of the energy dissipation of links, the behavior of braces the benefits of eccentrically braced systems.

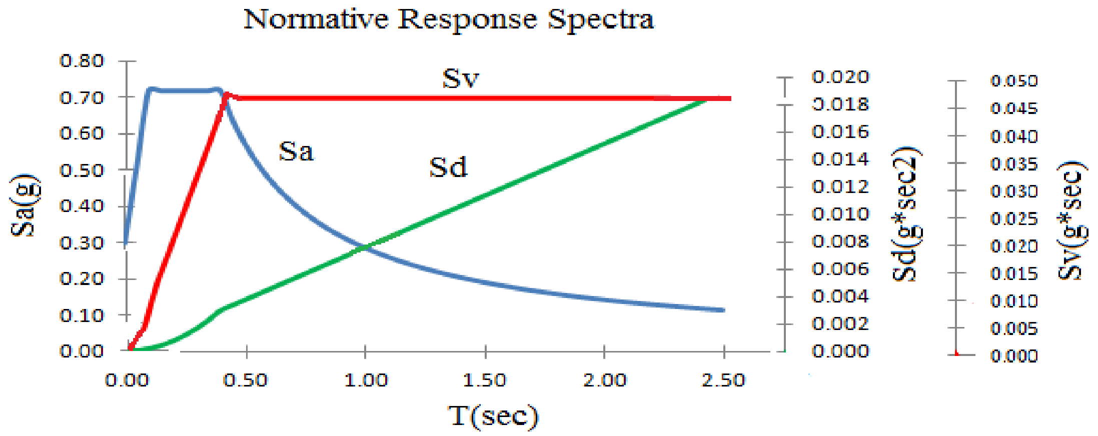


Figure 1. Response Spectra (Sa, Sv, Sd) of Covenin 1756-2001.

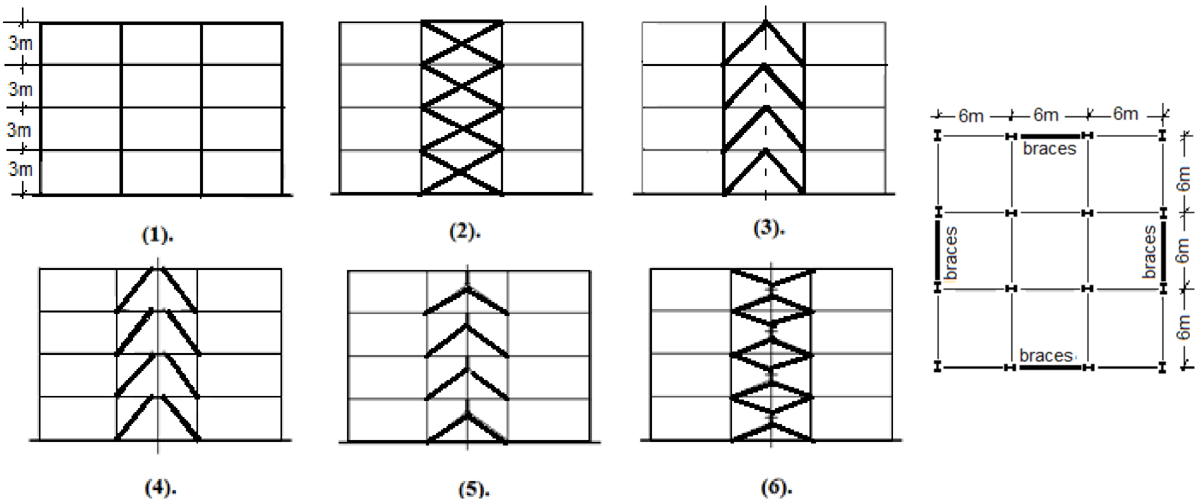
2. SELECTION OF THE MODELS

The shapes of models it show in Fig. 2(a) and the reasons to select it are the followings:

a). Fig. 1(1). Normal steel building frame without braces, to compare the cases.

b). Fig. 1(2). Concentrically braces (CBF) in X, the most stiffness condition.
c). Fig. 1(3). Concentrically braces (CBF) in inverse V to compare with Fig 1(4).
d). Fig. 1(4). Eccentrically K braces (EBF) with a link on beam.
e). Fig. 1(5). Eccentrically braces (EBF) in inverse V with a vertical link.
f). Fig. 1(6). Double eccentrically braces (EBF) in X with a vertical link.

The typical floor plan is the same in all the buildings and stories, that it is show in Fig. 2(b). Also on the plan are show the braces and the steel HEB sections of columns and your directions. Also HEB sections were used for columns and braces, and IPE on beams with ST-37 steel grade.



(a) (b)

Figure 2. (a). Bracing arrangements analized; (b). Typical floor plan of the buildings.

The structural analysis for dimension of elements (beams, columns, braces and links) was made with ETABS software, that which were calibrated after with ZEUS NL software. The values of the basic selected sections are shown in Fig. 3, for the types of buildings. For 6 types of braces and 4 different heighs it result a total of 24 buildings analyzed in modal elastic design and 24 cases in nonlinear condition. In Fig. 4 is shown some (not all) arrangements of links in reference to the sections, the shapes and the spacing of stiffener and steel sections (Table 2). Beside, In Table 1, it is shows the Steel sections used in braces and links

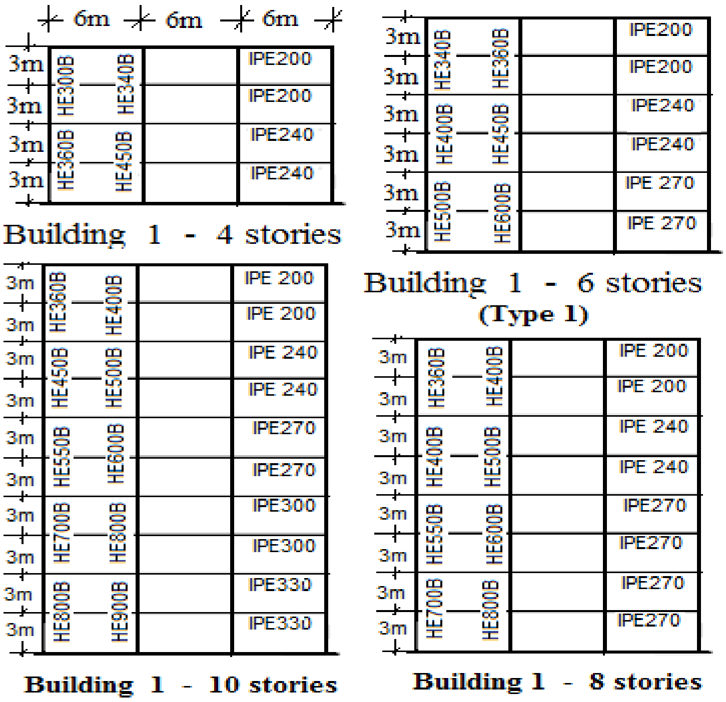
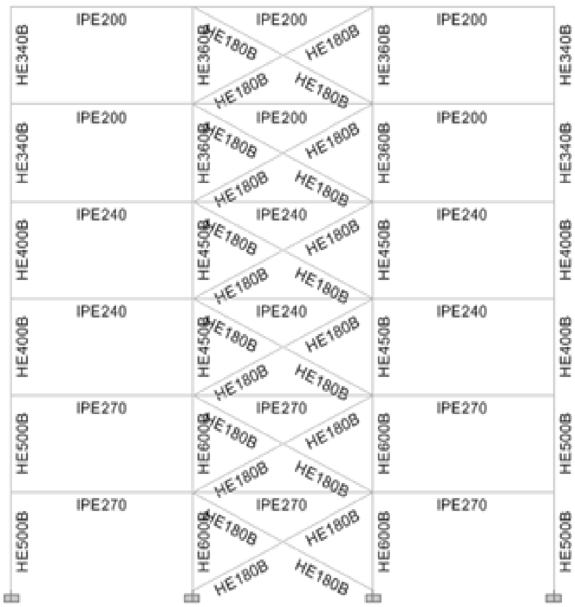


Fig. 3. Normal Steel Buildings frame without braces

Table 1. Steel sections used in braces and links

BUILDINGS	TYPE	STEEL SECTIONS
4,6,8,10 STORIES	2	HE180B
	3	HE160B
	4	HE160B
	5	HE160B/HE260B
	6	HE160B/HE200B



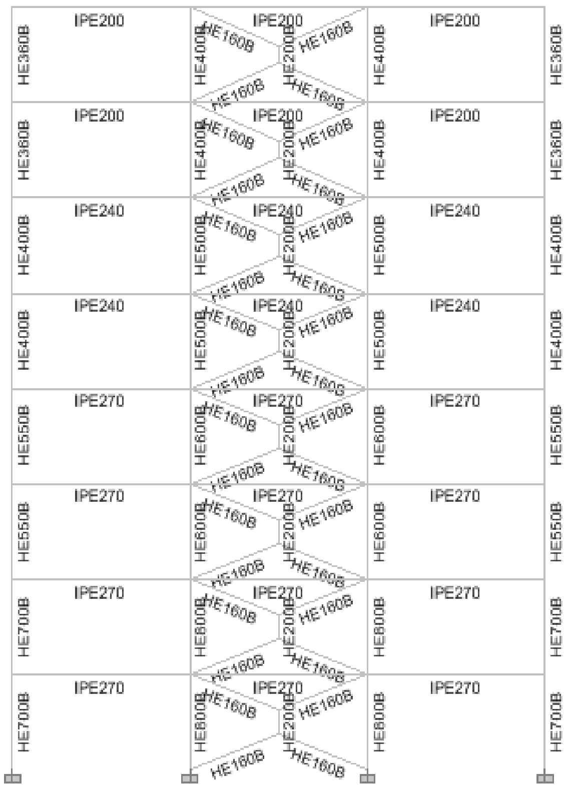


TABLE 2

LINK BEAM :

Covenin 1618-1998

BEAM IPE	e(cm)	Df (mm)	N°	S(cm)
200	55	100	5	12.00
240	70	120	6	12.67
270	75	135	6	13.30
300	85	150	7	13.14
330	85	160	6	15.17

Fig. 4. Examples of two arrangements of frames (2-2; 3-6), links, spacing of stiffener and steel sections.

2. RESULTS OF MODAL ELASTIC ANALYSIS

The inelastic lateral displacements Δi of the story was calculated according with the Covenin Code 1756-2001, by the following expression: Δi = 0.8R Δei, where R = 6 is the response factor and Δei is the elastic lateral displacement of level (i). These values were obtaining by the superposition of three modes applying SRSS rule. The results are expressed in drifts are shown in Fig 6, and controlled by the quotient of the interstory drift (Δ_i - Δ_{i-1}) / (hi – hi-1), where “hi” is the heigh of story (i). En this case this maximum value is 0.018 corresponding to Group B2 or susceptible structures to experiment damages by structural deformations. The drift curves obtained by modal analysis shown that the maximum values correspond to case (1) without braces and the minor values it refer to configuration (2) and (3) which are concentrically (CBF) in X braces and concentrically (CBF) in inverted V braces, the most rigid conditions. The other models make medium condition, but reducing the displacement by the braces, in degradation of effectiveness. These are the eccentrically K braces (EBF) with a link on the beam (4), the eccentrically (EBF) inverted V braces (5) with a vertical link and the double eccentrically (EBF) in X braces with a vertical link (6). These results will be calibrated by the nonlinear analysis.

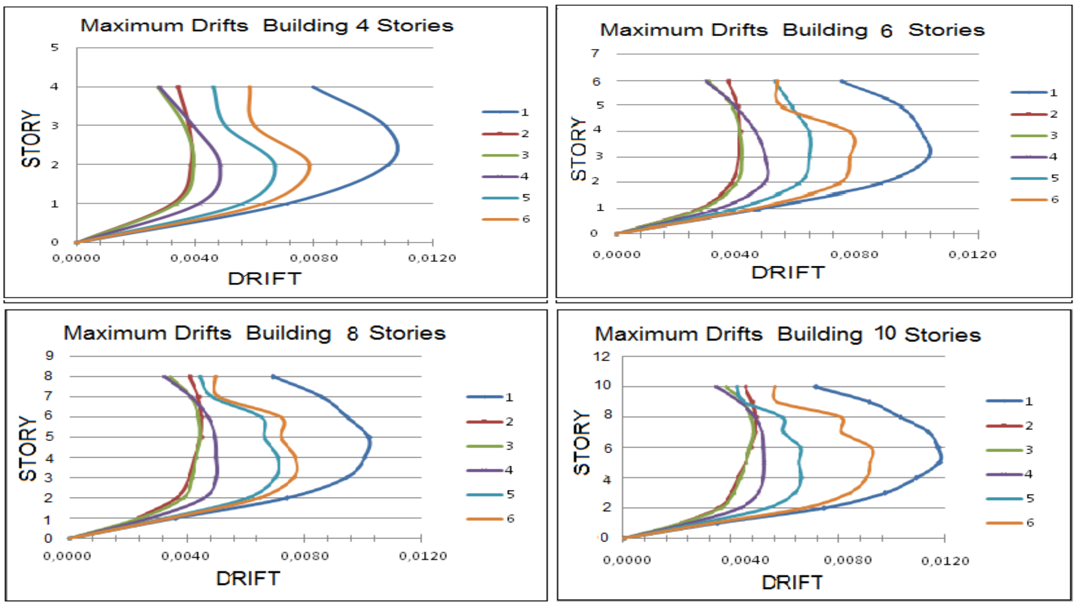


Figure 7. Results obtained on drifts in 24 cases studied

3. NON LINEAR CHARACTERISTICS

The three-dimensional nonlinear analyses was made by the software ZEUS NL, considered as "A System for Inelastic Analysis of Structures" (Elnashai et al, 2010), a way to solve nonlinear dynamic time-history, conventional and adaptive pushover and eigenvalue analysis. In this paper, it constructs the pushover curves in Non Linear Static Analysis to know the difference of behavior of the eccentrically and concentrically steel frames. Before, it apply the Capacity Spectrum Method "interpreted as an extension of response spectrum analysis for a linear structure, but using the secant period and damping ratio rather than the elastic period and 5% damping" (Powell, 2006). The pushover curve permit the computation of R, the Ksec, Tsec, ductility, the hysteretic area Ah or hysteretic energy, the elastic area Ae or the elastic energy, energy dissipation E_D and the factor of overstrength. The procedure is the following:

1. To construct the Pushover Curve and approximate bilinear curve, and to obtain the yielding point.
2. To establish the appropriate displacement which can be the normative value.
3. To define one performance point on the curve with the assumed displacement.
4. To take the normative elastic spectrum and to construct the reducing spectra increasing the damping factor β.
5. Over the performance point or other point to obtain Ksec, to calculate Tsec and ductility ratio μ.
6. With Tsec to intersect the reducing spectra βequiv, calculated with the area into the pushover curve in an iterative process, with the following expressions:

$$\beta = 0.05 + \beta o \quad ; \quad \beta o = E_D / 4\pi . ESo \; ; \; Ah = E_D \; ; \quad ESo = Ae \tag{4}$$

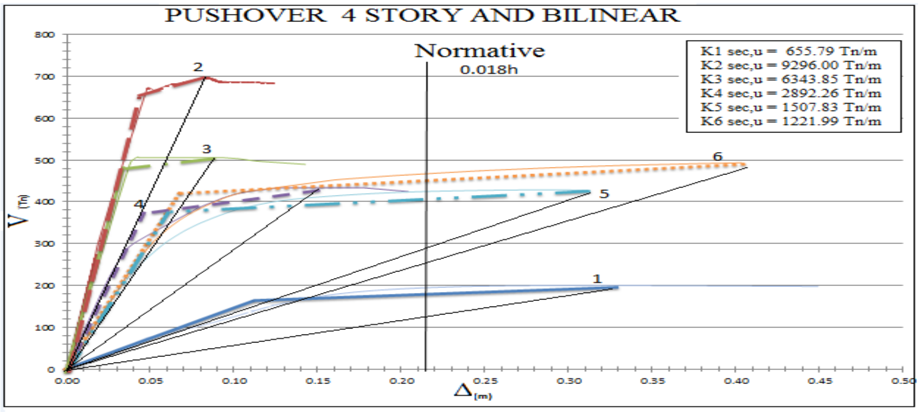
The pushover and bilinear curves are shown in Fig 8, for 4, 6, 8 and 10 stories. These curves will be the values which are taking into account in the response analysis.

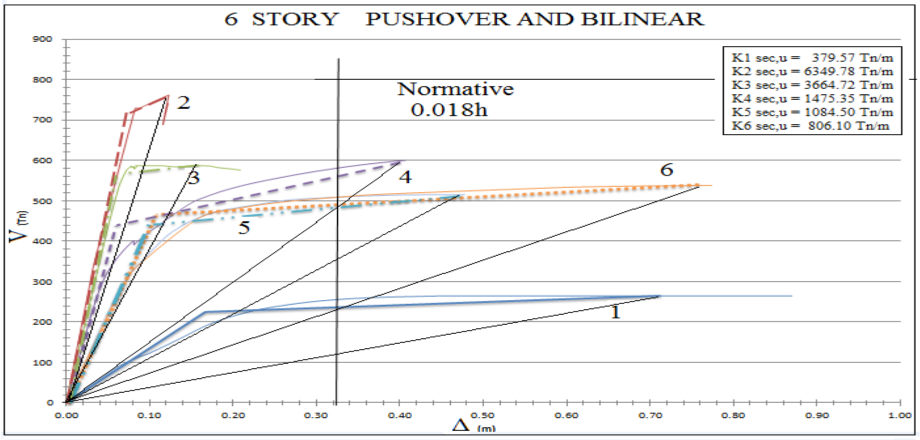
4. DISCUSSION OF RESULTS

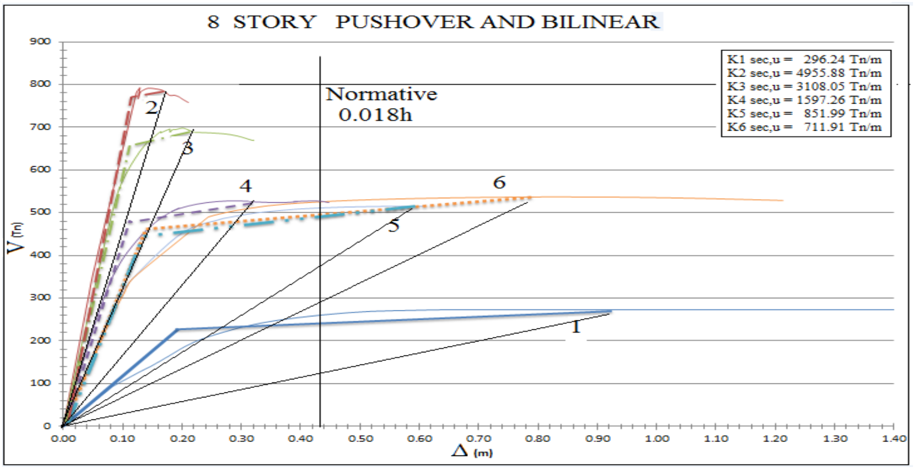
The behavior of frames in 24 studied cases by static nonlinear analysis (Fig. 8) is similar and shown that the capacity curve of frames without braces (Case 1) is the lower strength and lateral stiffness with some deformation capacity. The others extreme conditions are represented by the structures with concentrically braces in X (Type 2) or inverted V (Type 3), whose responses are of the high initial strength and stiffness but without deformation capacity. These structures have a little ductility and lower energy dissipation capacity under seismic actions. However, there are differences of strength and capacity deformation between the case (1) and the other cases (Fig.9). When the links are present the pushover curves given intermediate responses in the cases (4), (5) and (6), where the better behavior obtained corresponding to vertical links. The case (4) it refer to the eccentrically K braces (EBF) with a link on the beam. That result is very important because facilitate the change of the links after an earthquake. It is possible to improve the EBF condition by the buckling restrained brace (BRB), which are a better technology. This solution has been shown in other papers (Dasgupta, 2004). In this paper the elastic (Te) and the inelastic periods (Tsec) are shown in Table 3 for all cases. The K sec,u was taken in the point of initiation of progressive collapse in the pushover curve and correspond to T sec,u. The analysis of this values show a good interpretation about the increased periods in the nonlinear state in 24 frames.

Table 3. Elastic (Te) and inelastic periods (Tsec, u) of the buildings (sec.)

Type	4 story	6 story	8 story	10 story
1	1.2163 (1.67)	1.607 (3.03)	2.010 (3.96)	2.410 (4.14)
2	0.5231 (0.70)	0.779 (0.98)	1.064 (1.26)	1.374 (1.61)
3	0.5234 (0.77)	0.778 (1.24)	1.061 (1.46)	1.366 (1.78)
4	0.6052 (1.04)	0.886 (2.00)	1.181 (1.95)	1.501 (2.30)
5	0.7684 (1.60)	1.088 (2.19)	1.412 (2.79)	1.763 (3.15)
6	0.8869 (1.99)	1.234 (2.98)	1.594 (3.42)	2.009 (3.97)







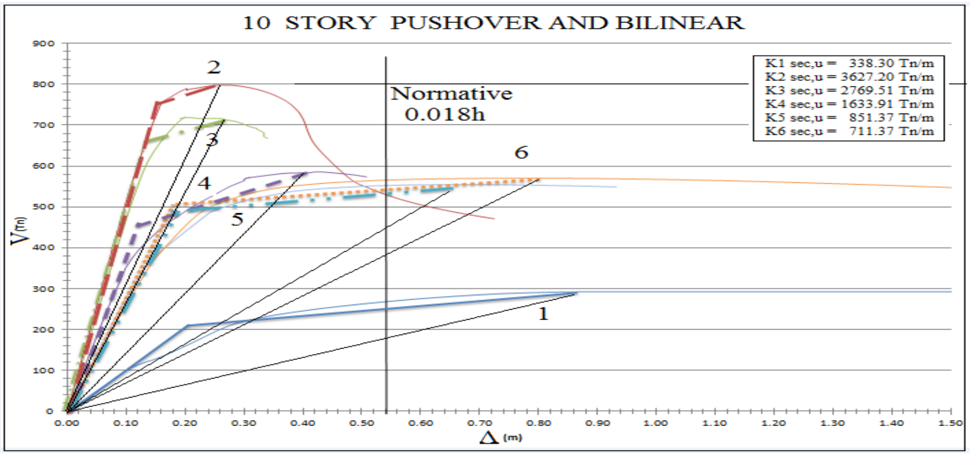


Figure 8. Pushover, bilinear curves and Ksec,u: (a). 4 stories; (b). 6 stories; (c). 8 stories; (d). 10 stories

In Fig. 9 show the different results of 24 cases, according to Overstrength factor, Maximum ductility and the Response factor R. In relation to the overstrenght, we can see that the building Type 1 have the lower value in all cases, the building Type 2 in concentrically braces increased around 50%, but the Type 6 have a value of 80% in 10 story building. That is the structure with major overstrength. The ductility factor

(μ_{max}) it refer to the displacement corresponding to the maximum strength of the capacity curve and the Type 2, result the frame with minor ductility in all heighs. The maximum ductility correspond to cases Types 5 and 6, and in never case the ductility was major of 4. The response factor R result increased en cases 5 y 6, overcoming the value of Covenin 1756-2001. All the values of R were calculated for a maximum deformation of 0.018 h of the Covenin Code.

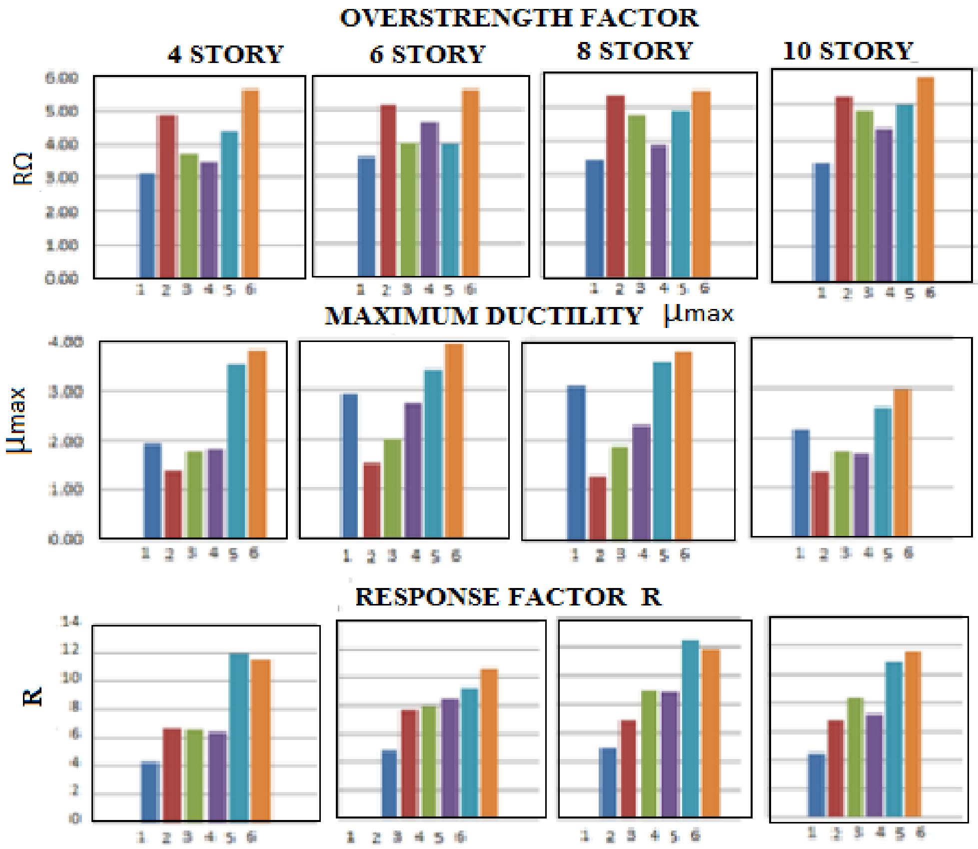


Figure 9. Values of overstrenght factor, maximum ductility and response factor R

Table 4(a) and 4(b) shows the results of the equations (4) and determine the equivalent damping factor βequiv in nonlinear condition to the maximum strength and the admissible displacement of

0.018h. The values of damping are aproximately from 20% to 50% , where the major values correspond to types (4), (5) and (6), in all heighs. The energy dissipation E_D, which is the principal objetive of this paper, determine values limited by the admisible displacemet in Type 1-1 result 39.60 Ton-m, that increase to Type 4-1 of 174.89 Ton-m. But, in eccentrically braces Type 1-6 have 215.52 Ton-m and Type 4-6 result 610.23 Ton-m.

The 4 Storey braced frame can to increase 5.4 times the energy dissipation according to the brace and the configuration of 10 story can to increase 3.48 times. A resume of the results can be seen in Tables 4 and 5.

			1	2	3	4	5	6
4 STOTY	ULTIMATE	ESO	31,31	26,15	20,30	32,54	60,91	99,61
		ED	92,39	86,44	82,55	152,91	340,55	556,32
		βO	0,23	0,26	0,32	0,37	0,44	0,44
		βequi	0,28	0,31	0,37	0,42	0,49	0,49
	NORMATIVE 0.018h	ESO	21,31	-	-	-	45,36	50,17
		ED	39,60	-	-	-	192,96	215,52
		βO	0,15	-	-	-	0,34	0,34
		βequi	0,20	-	-	-	0,39	0,39
6 STOTY	ULTIMATE	ESO	92,17	45,72	46,91	122,10	123,02	179,75
		ED	461,26	127,45	217,67	588,26	631,68	1040,16
		βO	0,40	0,22	0,37	0,38	0,41	0,46
		βequi	0,45	0,27	0,42	0,43	0,46	0,51
	NORMATIVE 0.018h	ESO	40,50	-	-	93,96	82,30	81,81
		ED	103,28	-	-	444,00	367,04	348,80
		βO	0,20	-	-	0,38	0,35	0,34
		βequi	0,25	-	-	0,43	0,40	0,39

Table 4(a). Values of Eso, E_D, βo and β_{equiv}, in 4 story and 6 story.

		TYPE					
		1	2	3	4	5	6
ULTIMATE	ESO	125,64	63,44	78,35	81,78	156,89	202,46
	ED	629,06	143,91	276,73	389,51	813,16	1087,02
	βO	0,40	0,18	0,28	0,38	0,41	0,43
	βequi	0,45	0,23	0,33	0,43	0,46	0,48
NORMATIVE 0.018h	ESO	56,25	-	-	108,00	108,00	113,75
	ED	133,95	-	-	589,44	507,60	441,39
	βO	0,19	-	-	0,43	0,37	0,31
	βequi	0,24	-	-	0,48	0,42	0,36
ULTIMATE	ESO	126,35	87,78	93,61	100,08	181,11	228,52
	ED	457,52	181,21	283,16	369,50	878,62	1192,55
	βO	0,29	0,16	0,24	0,29	0,39	0,42
	βequi	0,34	0,21	0,29	0,34	0,44	0,47
NORMATIVE 0.018h	ESO	72,12	-	-	-	148,92	152,62
	ED	174,89	-	-	-	553,28	610,23
	βO	0,19	-	-	-	0,30	0,32
	βequi	0,24	-	-	-	0,35	0,37

Table 4(b). Values of Eso, E_D, βo and β_{equiv}, in 8 story and 10 story.

CONCLUSIONS

1. The objetive of this paper is to know the behavior of normal steel building frames reinforced with simply eccentrically braces disposed over the perimeter of buildings to dissipate energy under seismically actions.
2. The design of steel building was made according with the Venezuelan Seismic Code Covenin (1756-2001) and the Venezuelan Steel Structures Code Covenin (1618-1998), for buildings with 4, 6, 8 and 10 stories to obtain the response initially without braces and thereafter including concentrically and eccentrically braces.
3. The concentrically-braced frames showed greater stiffness and strength to limit the lateral displacements, but decreasing displacement capacity and ductility.
4. The linear elastic analysis and non linear static analysis showed a similar behavior of the buildings; the elastic and inelastic periods, the ductility, the energy dissipation and the reduction factor of response R, resulting a better behavior in the cases of vertical links.
5. It is a moment to apply these technologies and improve the knowledge and the practice into the design and construction of eccentrically braces buildings in developing countries.

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