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BEHAVIOR OF ECCENTRICALLY BRACED STEEL FRAMES OF REGULAR BUILDINGS IN SOFT SOILS

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Abstract

Results of nonlinear analyses of 6- and 9-stories regular buildings structured with moment-resisting eccentrically braced steel frames are discussed in this paper. Models were designed following the capacity design principle for a seismic modification factor Q= 4.0, the maximum allowed for these structures, according to the Mexico City Building Code. Nonlinear pushover and dynamic analyses were performed in 3D in *OpenSees* and were supposed located in soft soil site condition (Mexico City's lake-bed zone). Dynamic analyses were performed under ten historical records related to the considered design spectra. For this purposes, a detailed model in *OpenSees* with material and geometrical non-linearities was developed. Nonlinear beam-column elements, with plasticity spread along the element length, were considered for the link beam elements, beam segments outside of the shear links, braces and columns; and a detailed model of the connection at the ends of link elements with springs was considered. A quadratic perturbation shape was used to define the initial camber in the bracing system and an out-of-plumbness of columns was also considered as imperfections in the locations of points of intersection of elements. Through 1,756 certified laboratory coupons test of steel samples a better reference of the actual yielding steel stress was included in order to evaluate the material overstrength capacity at the local market.

According to the results, the overall response of the structure was governed by the maximum link rotations by interstory. Top stories reported practically an elastic behavior, whereas peak story drifts were developed at the middle stories, driving potential weak stories under static and dynamic inelastic analyses. Median link rotations went beyond the code limit (0.08 rad). A magnitude of the rotation link capacity is proposed in order to better predict the seismic response, which is based on a statistical study of results of experimental tests with respect to the beam link cross-section. A slight relationship among ductility and building height was noticed.

Keywords: Eccentrically braced frames; drift; ductility; material overstrength; soft-story.

1. Introduction

The successful performance of Eccentrically Braced Frames (EBFs) under seismic loading depends on stable inelastic rotation of active links, which are designed to act as structural fuses. In order to dissipate seismic forces, links must sustain large inelastic deformations. When links are properly designed, the columns, braces and beam regions outside the links should remain essentially elastic.

Frames with eccentric bracings shall be designed so that seismic links are able to dissipate energy by the formation of plastic bending and/or plastic shear mechanisms. For shear links, while the link web provides the majority of the shear resistance, the link flanges can also contribute [1, 2]. In a study on link overstrength, Okasaki *et al*. [3] found that built-up short links with heavy flanges typically had larger than anticipated overstrength factors. According to Richards [4], the overstrength is increased by a flange shear contributions, which act as slender beams once the web has fully yielded.

Recent researches have underlined [1, 5, 6] that medium or high-rise eccentrically braced frames designed in compliance with capacity design principles can develop undesired soft-story collapse mechanisms with a non-uniform inelastic response along the height, which is characterized by negligible plastic rotation of most links. In fact, design procedures generally suggest by seismic codes [7, 8] for eccentrically braced structures do not ensure wide spread of the plastic behavior of links among all stories prior to link failure. Some studies [1, 6] have shown that in moderate and high-rise buildings, the design of eccentrically braced frames according to capacity design principles, leads to low and non-uniform values of the damage distribution. This design deficiency may negatively affect the assessment of the seismic response of eccentrically braced frames by the designers in terms of their dissipative capacity: lateral deformation, ductility and overstrength capacities.

This research aims to evaluating the seismic response of typical ductile eccentrically braced steel frames located in soft soil condition. Pursuant to this goal, a detailed model was developed in order to perform nonlinear pushover and dynamic analyses. The study pretends to improve the acquired knowledge and to propose some additional provision to control the damage distribution by moving the inelastic mechanism. Specifically, the attention of this paper is focused on explore a more realistic assessment of the inelastic link capacity, actual material properties available in the local market, the ductility and overstrength capacities of such structural systems and, finally, recommendations are given for safe and economical design of ductile eccentrically braced frames.

2. Models description

Two medium rise buildings were studied. Buildings were 6- and 9-stories in height and were located in soft soil site conditions having a site period equal to T_g= 2.0 sec and T_a= 1.175 sec and T_b= 2.40 sec (periods that defines the plateau of the design spectrum). A seismic response modification factor equal to Q= 4.0 was considered, the maximum allowed for these structures, according to the Mexico City Building Code [8]. The buildings were representative of typical office buildings with special moment resisting eccentrically braced frames in two different bracing configurations for the internal and the external frames as it is shown in Fig.1. It is worth noting that in the local design practice, all frames (exterior and interior) are designed to resist earthquake loading. The design gravity loads for the studied models are also given in Fig. 1. Buildings were designed using three-dimensional models using response spectrum analysis according a capacity design procedure [9].

2.1 Design considerations

The resisting elements were designed using the results obtained with the 3D buildings models and standard capacity concepts for ductile systems through an iterative process. The link overstrength factor was used to estimate the maximum forces that can be generated by a fully yielded and strain hardened link in all stories, which in turn was the used to design the diagonal brace, the beam segment outside of the link, the columns and, finally the panel zone connection.

Some studies [10] have recommended a link overstrength factor of 1.5. However, the actual specified factors are less than 1.5 for a number of reasons, including the use of the R_{mat} factor to account for material overstrength, the use of resistance factors when computing the strength of the brace and other members outside of the link, the ability to sustain limited yielding in members outside of the link among other factors [3]. Currently, AISC 341-10 [9] specifies a link overstrength factor of 1.25 for I-shaped links and 1.4 for box links. Where the capacity-design methodology is employed, it is reasonable to use the expected material strength in the determination of the member capacity. For limit states based on yield, the factor R_{mat} applies equally to the designed yielding member capacity used to compute the required strength and to the strength with respect to the limit states to be precluded. Further detail and information of designed buildings can be found elsewhere in [11].

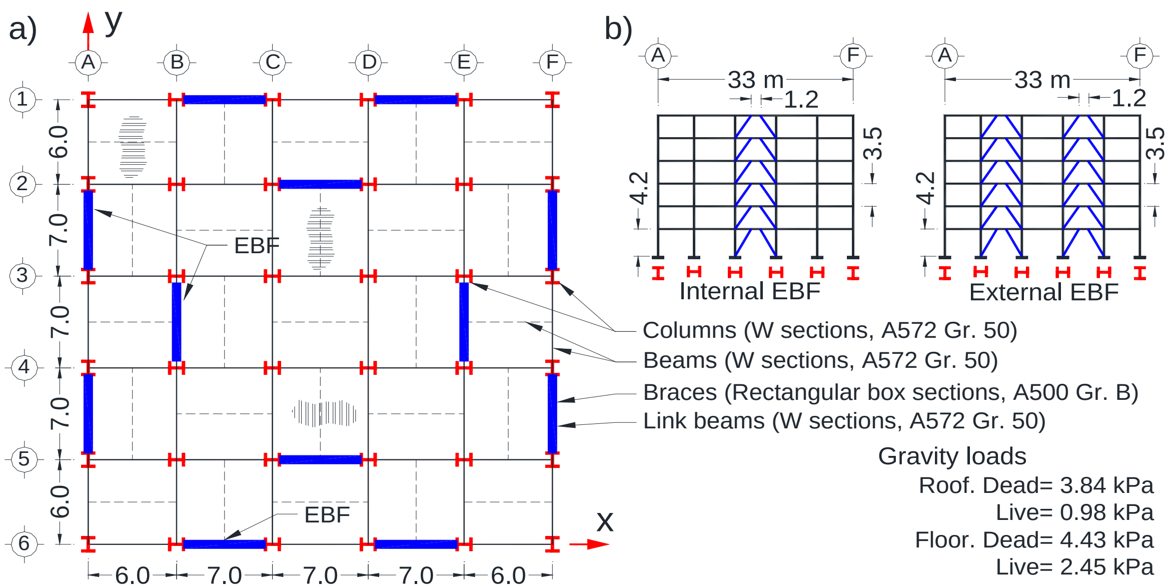
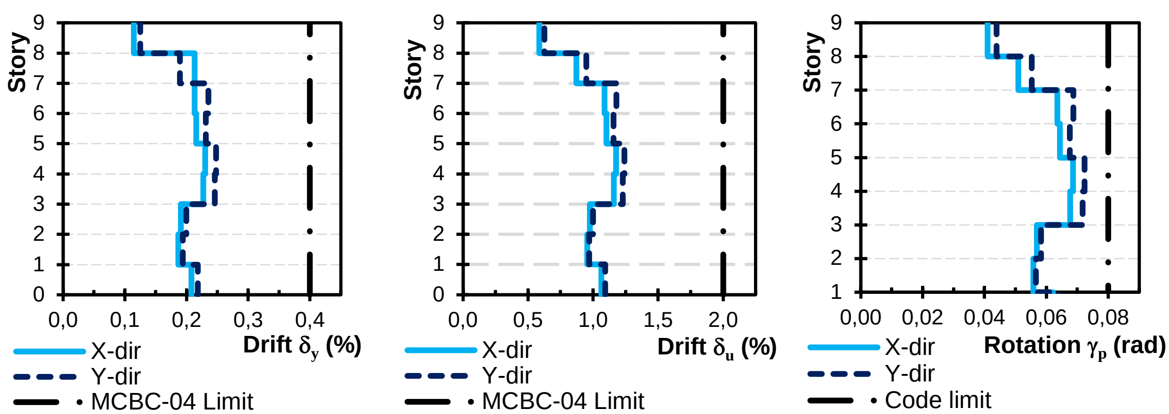


Fig. 1 - Studied buildings: a) Typical floor plan view; b) EBF elevations

Buildings were designed according to the seismic provisions for a service drift limit equal to 0.4% and a maximum story drift ratio 2.0% for the ultimate drift limit [8]. Because shorter links that rotate due to web shear yielding are more common that longer links which develop flexural hinges at each end [1], here shear links were considered. All beam links are e= 120 cm (e/L= 0.17). The maximum rotation of the links was subjected to meet the Code's target plastic rotation levels for the shear links. Therefore, the link rotation angles were limited to 0.08 rad for shear yielding links based on the current codes [8, 9]. In fact, as it is depicted in Fig. 2 for the 9-story building, the overall design is governed by the maximum link rotation by interstory, instead by the lateral deformation demand.

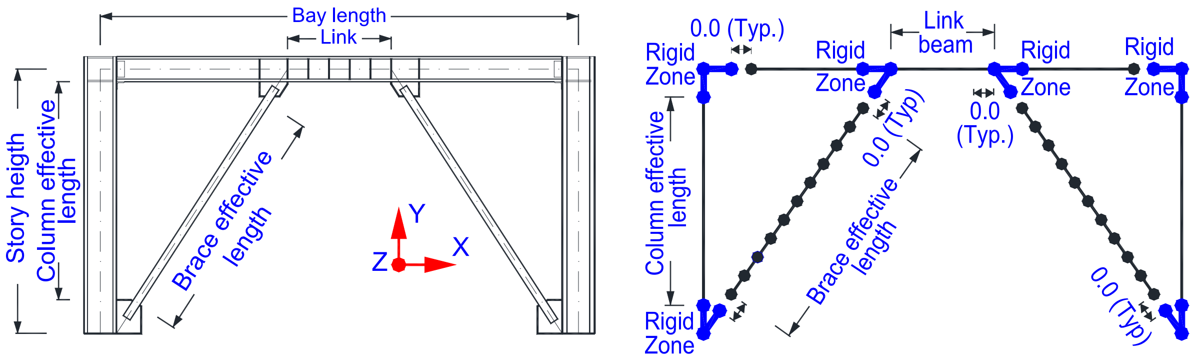


a) Service drift (%) b) Final drift (%) c) Ultimate rotation (rad)

Fig. 2 - Deformation demands for the 9-story model

3. Inelastic model

Three-dimensional inelastic analyses were performed by using the open access software *OpenSees* [12]. Beam and column centerlines were used to define model geometry. Beam segments outside of the shear links, braces and columns were all modeled as beam-column elements. Rigid zones were included at the ends of columns, beams and brace (Fig. 3).



a) EBF elevation b) Analytical model

Fig. 3 - Typical elevation

Nonlinear beam-column elements, with plasticity spread along the element length, were considered. Three rectangular patches were used to generate cross-section of wide flange beams: one for the web and two for each flange; whereas, four rectangular patches were used for the hollow structural sections. Patches were discretized into fibers with quadrilateral shapes and four integration points per element.

In the model, torsional restraint of the element was also included when effects out-of-plane are studied. Element torsional properties have been added to the fiber nonlinear beam-column element by using the corresponding aggregation tool in the software [12]. In addition, brace elements were modeled with a set of ten nonlinear beam-column elements to reproduce the response of an axially loaded element including large translational displacement and P-delta effects (Fig. 3b).

A quadratic perturbation shape was used to define the initial camber in the bracing system; without this initial camber, this pin-ended brace will behave as an ideal, perfectly straight uniaxial element, with no global buckling possible [13, 14]. The initial out-of-straightness assigned to the brace model was $L/500$, which is in agreement with that recommended by the local code [8]. This small initial camber introduces a perturbation that triggers buckling. To simulate the behavior of the gusset plate connection at each brace end, rotational springs were defined in the zero-Length element that connects each end of the brace member to a rigid link.

A steel A572 Gr. 50 with nominal properties f_y= 345 MPa, E= 200,000 MPa and G= 77 GPa was used for all members; except for the bracing system, where a steel A500 Gr. B f_y= 320 MPa was considered. The uniaxial Giuffre-Menegotto-Pinto (GMP) material is used for steel fibers with extensions included for kinematic and isotropic hardening. The inelastic model used herein accounts the response along the element by integration of the uniaxial hysteric steel material model over the cross section.

3.1 Link model

Shear links were modeled using a technique similar to that proposed by Ramadan and Ghobarah [15], but with some modification. Two nodes at each end of the link, referred to as the external and internal nodes, are defined to have the same coordinates through a zero-length element. The beam connects the internal nodes on either end of the link. Shear hinging in the link is modeled by translational springs that couple the vertical translational degrees of freedom of the external and internal nodes [1]. Three translational springs operate in parallel at each end in order to achieve multi-linear force deformation relationships using bilinear spring element. The horizontal displacement of each internal node is constrained to equal that of the corresponding external node. Finally, link rotation is calculated as the vertical distance between the external nodes divided by the length of the link.

Thus, model was found to give realistic predictions of the inelastic response of a member. Specially, similar correlation was observed for analytical and experimental results of the links [15, 16]. The correlation study indicated that the link behavior can be reasonably modeled with only cinematic hardening in the translational springs (rather than combined isotropic and kinematic), although some accuracy is lost in small initial cycles [1]. Further details and validation of this modeling technique may be found elsewhere in [11].

3.2 Material overstrength

The overstrength factor for eccentrically braced frames is usually obtained from $R = R_{size}R_{\phi}R_{mat}R_{sh}$, where R_{size} accounts for the difference between actual member sizes against required member size, R_{ϕ} reflects the difference between nominal and factored member resistances, R_{mat} is the ratio between expected and nominal steel yield strength and R_{sh} is the overstrength due to link strain hardening upon yielding [17].

According to the AISC 341 [9], the specified values of R_{mat} for rolled shapes are somewhat lower than those that can be calculated using the mean values reported in a survey conducted by the Structural Shape Producers Council (Table 1). Those values were skewed somewhat by the inclusion of a large number of smaller members common in seismic design. The given values are considered to be reasonable averages of what can be found in the United States, although it is worth noting that this trend may not be true in other countries, where the values of R_{mat} might be unconservative.

Although this higher strength translates into safer structures for non-seismic design, unexpectedly higher yield strength can be disadvantageous for seismic design. In particular, in eccentrically braced frames, link element is designed to yield, absorb energy and prevent adjacent elements from being loaded above a pre-determinate level during a strong earthquake. Then, yield strength much higher than expected could prevent that link from yielding and overload the adjacent structural components, with drastic consequences on the ultimate behavior of the eccentrically braced frames. For this reason, the realistic assessment of the link capacities is extremely important in order to evaluate the structure overall behavior.

Table 1 - Values for steel materials

Application	AISC 341-10		This study			
			Non-sampling error		98% confidence level	
	R_{mat}	R_t	R_{mat}	R_t	R_{mat}	R_t
Hot-rolled structural shapes						
ASTM A36	1.50	1.20	1.32	1.15	1.33	1.17
ASTM A529 Gr. 50	1.20	1.20	1.08	1.16	1.09	1.17
ASTM A500 Gr. B	1.40	1.30	1.12	1.11	1.12	1.11
ASTM A572 Gr. 50	1.10	1.10	1.01	1.08	1.12	1.12
Plates and sheets						
ASTM A36	1.30	1.20	1.10	1.13	1.11	1.14
ASTM A572 Gr. 50	1.10	1.20	1.08	1.11	1.08	1.12

Based on the above, through 1,756 certified laboratory coupon tests of steel samples available in the Mexican market, an improved assessment of the actual yielding steel stress was obtained. The available samples were angles, HSS, I-shapes, channels and plates with thickness between t= 3.2 mm (1/8”) to t= 62.5 mm (2.5”). The 68 percent were elaborated by Mexican steel producers and the other 32 percent were imported from the steel industry of Germany, Korea, Spain, China, Ukraine and the United States. No dependency between the material overstrength and the place of origin was noticed. Further information can be found in [11, 14].

Yielding steel stresses were studied in two scenarios: (i) non-sampling error and (ii) a 98 percent confidence level. Non-sampling error is a catch-all term for the deviation from the true value that is not a function of the sample chosen; it is related with a level of confidence of 100%. A confidence interval for a parameter is an interval of numbers within the true value of the population parameter is expected to be contained.

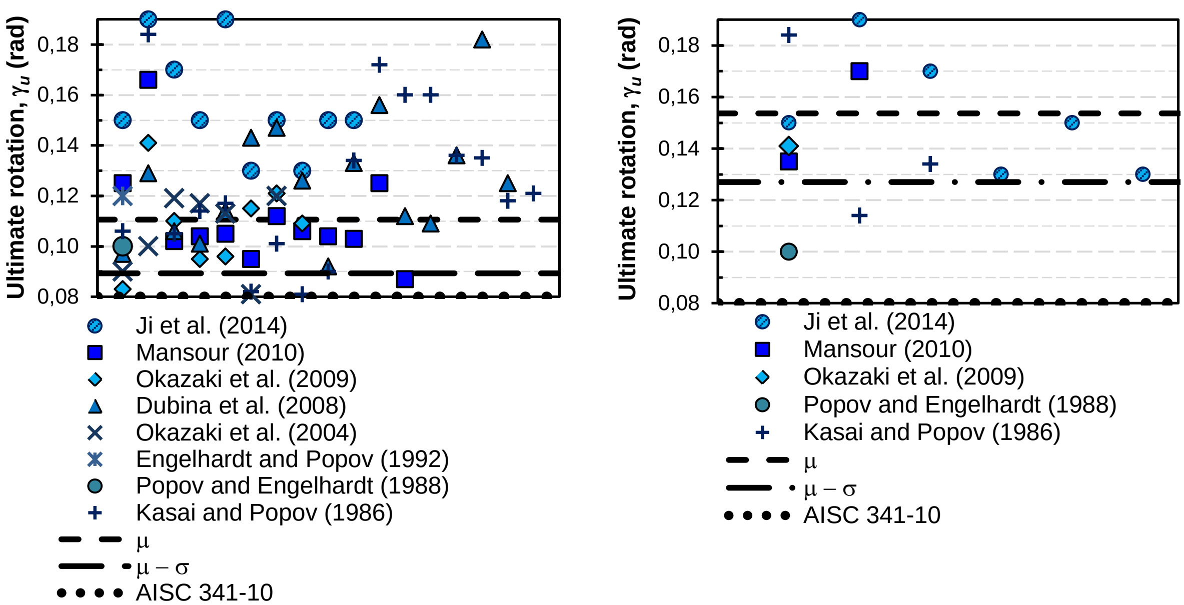
Obtained results showed average values of material overstrength around 1.12 for A500 Gr. B and A572 Gr. 50 steel with a 98% confidence level (Table 1), these corresponding to actual values of yielding stress available in the local market. Based on these results, the theoretical nominal strength of the link beam was modified considering the actual material overstrength. It is worth noting that the values of material overstrength cannot be extrapolated, but they have a strong dependency of the local market condition of each region.

4. Pushover analyses

Eccentrically braced frames have good ductility if links can accommodate the inelastic rotations imposed by sever seismic loading; the successful performance depends on stable inelastic rotation of active links while other frame components remain elastic. Current deign provisions are based primarily on a series of experimental studies in the 1980’s [16]. Most of the experimental testing to determine link inelastic rotation capacity, which has led to current codes rules, has been addressed almost on wide-flange shapes and shear-yielding links located at beam mid-spans [1, 3].

In this study, the incipient collapse was defined by the point where the link rotation exceeds the ultimate rotation capacity of the link elements based on the result of experimental researches, although the analyses showed a numerical convergence for a larger lateral load. Therefore, incipient collapse herein was not defined in terms of the point where there is no-convergence (in the iterative algebraic process) for an increased lateral load but instead of that, by the theoretical limits for the actual link element capacities.

Experimental testing has shown that links with column connections have less inelastic rotation capacity than mid-span links, because they tend to fracture in the flange connection [1]. In the past during the experimental testing, most of the short links (eV_p/M_p < 1.6) failed to reach 0.08 rad inelastic rotation, which is the design value permitted in current codes [7, 8, 9]. However, recent link experiments [3, 10, 18, 19, 20] indicate that shear-yielding links located at beam mid-spans should be able to achieve inelastic rotations beyond 0.08 rad (Fig. 4). Based on research tests, link rotations from experimental testing might be expected to be greater than the rotation achieved by current codes.



a) All cross-section b) I-shaped links

Fig. 4 - Ultimate rotations reported in experimental researches

Two scenarios of the ultimate rotations reported in experimental researches were studied: (i) the inelastic rotations regardless the cross-section of link beam (Fig. 4a) and (ii) rotations for I-shaped links with $e/L \approx 0.17$ as the beam link considered in this study, for sake of consistency (Fig. 4b). According to the results, the mean values were 0.111 rad. and 0.154 rad. respectively, which are larger than the one proposed in current codes. In order to account for the deviation, a conservative magnitude of the ultimate rotation relation to experimental capacities was developed from the mean minus one standard deviation in a Gumbel distribution.

Following this procedure (μ–σ), the ultimate rotations were equal to 0.089 rad and 0.127 rad, respectively, which were also included in Figs. 4. Thus, according to the procedure suggested, the available inelastic capacity is computed by taking into account an ultimate maximum rotation equal to 0.127 rad for I-shaped links.

4.1 Deformation capacities

Inelastic response for the studied models agreed reasonably well with the initial design assumptions with a good distribution of the inelastic demand along the height. Top stories reported practically an elastic behavior, whereas peak story drifts were developed at the middle stories (Fig. 5).

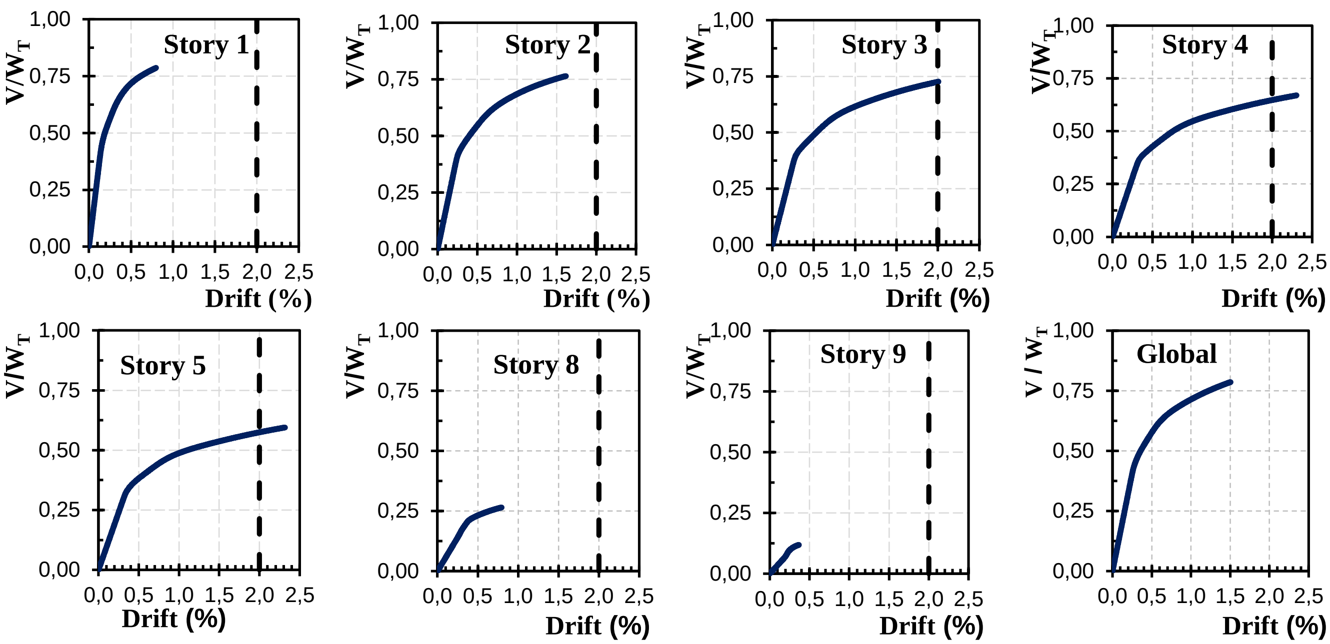
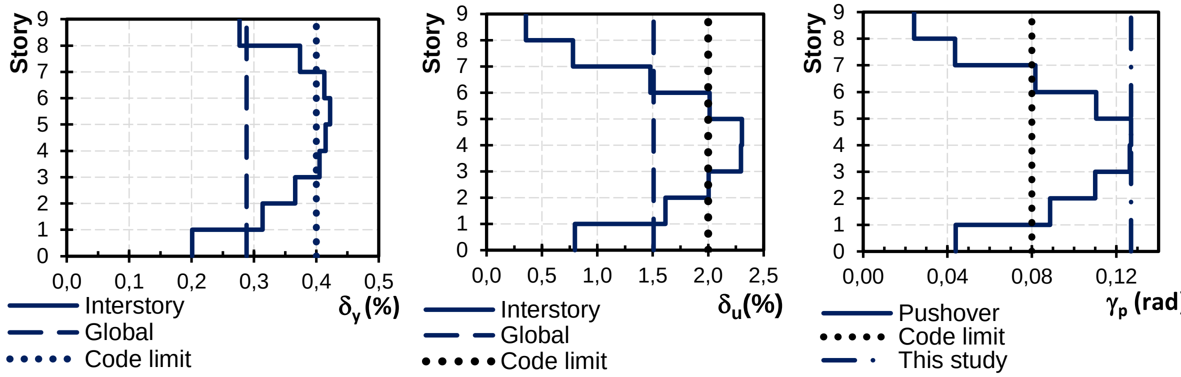


Fig. 5 - Pushover curves of 9-story model

The obtained average of the drift at first yielding δ_y and ultimate drifts δ_u are close to the code limit (MCBC-04) equal to 0.4% and 2.0%, respectively as it is shown in Figs. 6a and 6b, for the 9-story model. Therefore, these deformation limits seems adequate for practical purposes; the same trendy was noticed in the 6-story model. Following the procedure discussed above, the largest link rotation was limited to a maximum rotation equal to 0.127 rad. (Fig 6c).

Global ductility obtained from the curve that relates the base shear with top displacement and the ductility by interstory were calculated from pushover curves (Table 2, Fig. 7). The average story ductility was also obtained; it excludes the first story results due to the assumed fixed boundary condition and the results of stories with an elastic behavior. The final drifts considered are the maximum deformations related to the actual theoretical capacity for the members following the link rotation limit proposed above, instead the numerical solution obtained from the program output.

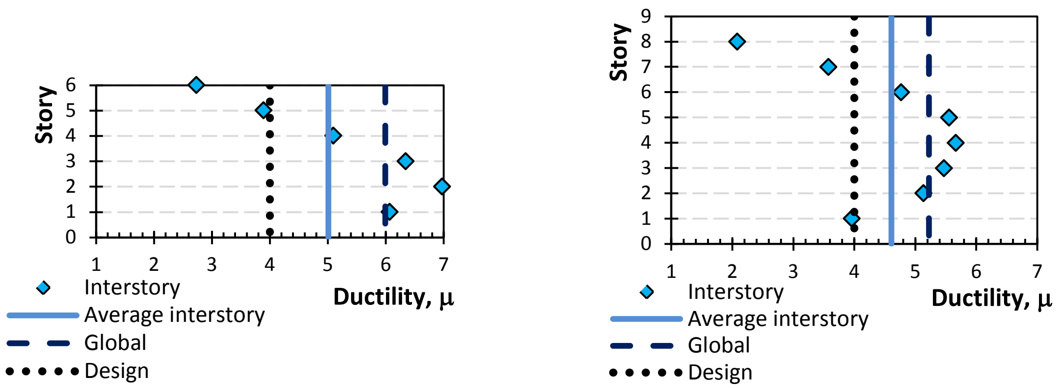
A slight relationship among ductility and building height was noticed (Table 2), something that it is not currently considered in building codes. It is observed that the assessed ductility (deformation capacity) decreases as the number of stories increases, which is in agreement with the results of some other studies of ductile steel braced frames [21].



a) Drifts at yielding, δ_y (%) b) Final drifts, δ_u (%) c) Final rotation, γ_p (rad)

Fig. 6 - Deformation demands for the 9-story model

The assessed ductility capacities are larger than the value considered in the design stage (Q= 4.0). Thus, spite of that fact that the magnitude of the reduction factors varies deeply with respect to the building configuration and the seismic design criteria, the ductility reduction factor is not completely representative of the capacity that might be developed by buildings designed following the MCBC-04 criteria [8].



a) 6-story model b) 9-story model

Fig. 7. Ductility with respect to the design ductility

Table 2. Characteristics of capacity curves

Model	6-story building	9-story building
Drift at yielding, δ_y (%)	0.277	0.288
Final drift, δ_u (%)	1.659	1.506
Shear at yielding, V_y/W_T	0.452	0.450
Final Shear, V_u/W_T	0.799	0.786
Global ductility, $\mu = \delta_u/\delta_u$	5.992	5.224
Overstrength, $\Omega=(V_u/W_T)/(V_y/W_T)$	1.766	1.748

In contrast, system overstrength at final drift was between 1.7 and 1.8 (Table 2), reasonably similar to the value of 2.0 assumed in design according to the MCBC-04 [8]. According to ATC-63 [22], the overstrength reduction factor of ASCE 7-05 [9] is not consistent with recent research results and varies between 1.5 (in the worst case) to 6.0. The Canadian Code establishes 4.0 for ductile eccentrically braced frames [23]. The EC8-05 [7] recommends only one value 1.25 for steel frames; nevertheless, different values for use in a given European Country may be found in its National Annex.

In MCBC-04 [8], an equation is proposed to determine the reduction factor as a function of the characteristic period T_a (the initial period that defines the plateau of the design spectrum), which is dependent of the ground period T_g. This criterion is shown in Fig. 8, using T_g= 2 s and T_a=1.175 s. In the plot, overstrength capacities obtained for the models under study are also included. Any dependency between the overstrength factor and the buildings height was found.

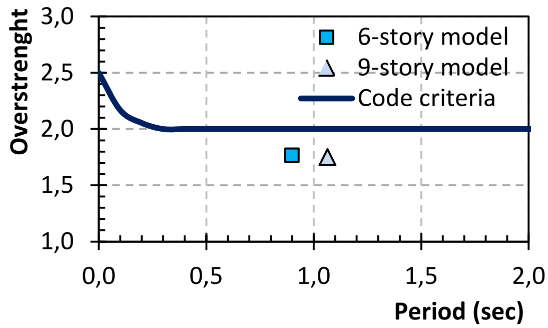


Fig. 8 – Assessed overstrength and the one obtained according to the MCBC-04

5. Dynamic analyses

Nonlinear time-history analyses using acceleration records related to the considered design spectra were also used to evaluate the seismic response in the OpenSees Software [12]. Dynamic analyses were performed through ten historical records, which are associated with the largest intensities recorded in Mexico as reported in the Mexican Database of Ground Motion Records between 1960 and 1936. The main characteristic of the records are summarized in Table 3.

Table 3 - Characteristics of the selected historical records

No.	Location	Long.	Lat.	Soil	Record
1	Alberca olímpica	99.154	19.358	Transition zone	AO249509
2	Jardínes de Coyoacán	99.127	19.313	Clay, Lake zone	JC549906
3	Cibeles	99.165	19.419	Soft clay	CI058904
4	Mariano Escobedo	99.182	19.438	Transition zone	ME529509
5	Granjas	99.180	19.475	Sand, Clay	GR278904
6	Ángel Urraza	99.168	19.383	Transition zone	AU468904
7	Córdoba	99.159	19.422	Clay, Lake zone	CO568904
8	Plutarco Elías Calles	99.132	19.39	Clay, Lake zone	PE108904
9	Colegio Madrid	99.134	19.287	Soft clay	DFCM9005
10	Lindavista	99.128	19.493	Soft clay	LV178904

Elastic response spectra for the selected records are compared with the design spectrum as shown in Fig. 9. It can be observed that records compare reasonably well with the seismic hazard established in the Code [8]. Fundamental periods of the studied models were also included in Fig. 9.

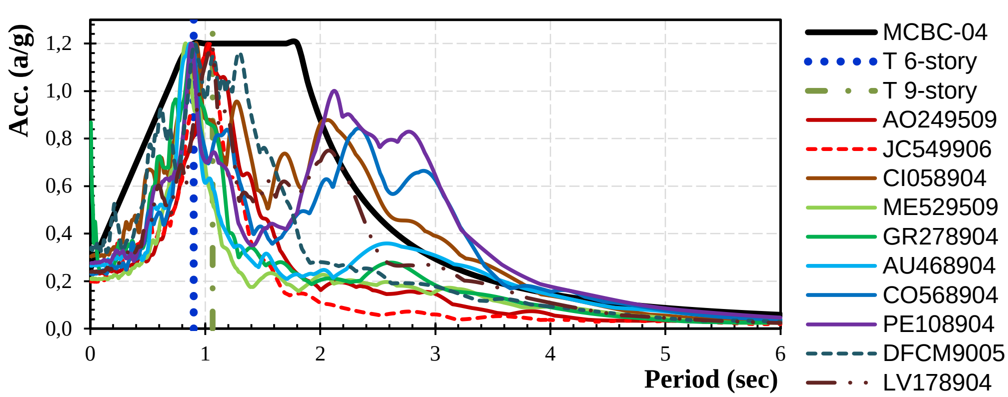
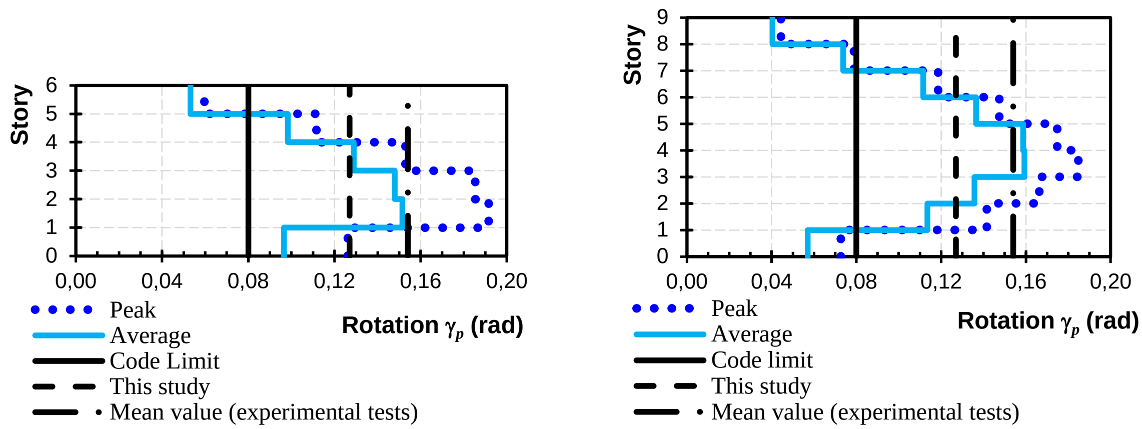


Fig. 9 - Elastic response spectra for the historical records and the elastic design spectrum

5.1 Inelastic angle rotation

The inelastic link rotation was monitored along the dynamic analyses in order to assess the maximum demand. The results of maximum link rotation by interstory and its median value between ten earthquakes are presented in Fig. 10. Obtained values are still within the range of experimental results, where the link rotations exceed the limit of 0.08 rad specified in current codes [7, 8, 9]. The average values of link rotations are about 0.113 and 0.110 rad for the 6-story and 9-story models, respectively; which are enveloped by the proposed ultimate rotation limit for I-shaped links (0.127 rad). So, for the studied models, the rotation limit seems adequate for practical purposes and might be a conservative approach of the actual capacity of the studied models.



a) 6-story model b) 9-story model

Fig. 10 - Developed ductility by the studied models

In the studied models, the largest rotations occurred in the middle height and despite of the fact that models were carefully designed in compliance with capacity design principles. Models show a near soft-story tendency with no uniform distribution of yielding within the height. Thus, final collapse mechanisms do not necessarily agree in many instances with the assumptions related to the code's design as it is reported in similar studies [6, 21]. No dependency was observed between the ultimate link rotation and the total height and/or the building aspect ratio.

6. Conclusions

In this paper, the results of nonlinear analyses of 6- and 9-stories regular buildings structured with ductile eccentrically braced frames in *OpenSees* were presented. Nonlinear pushover and dynamic analyses were performed in 3D in *OpenSees* and were supposed located in soft soil site condition (Mexico City's lake-bed zone). Nonlinear beam-column elements, with plasticity spread along the element length, were considered for the link beam elements, beam segments outside of the shear links, braces and columns; and a detailed model of the connection at the ends of link elements with springs was considered. A quadratic perturbation shape was used to define the initial camber in the bracing system and an out-of-plumbness of columns was also considered as imperfections in the locations of points of intersection of elements.

The study pretends to improve the acquired knowledge and the assessment of critical responses parameters. The main following observations were made from the analysis:

- It was observed that models behaved in compliance with the capacity design approach: links were yielded at middle and lower floors, whereas other members remained quasi-elastic. Nevertheless, a non-uniform inelastic response along the height was developed. In fact, top stories reported practically an elastic behavior, whereas peak story drifts were developed at the middle stories, driving potential weak stories under static and dynamic inelastic analyses. The design according to capacity design principles might not be enough to assess and predict the inelastic response of eccentrically braced frames and a substantial improvement on this matter might be still required.

- The obtained average of the drift at first yielding and ultimate drifts are close to the code limit and seems adequate for practical purposes. Nevertheless, the overall response of the structure was governed by the maximum link rotations by interstory.
- According to the results of the inelastic analysis, median link rotations went beyond the code limit (0.08 rad). A magnitude of the rotation link capacity is proposed in order to better predict the seismic response, which is based on a statistical study of results of experimental tests with respect to the beam link cross-section. Following the procedure, the available inelastic capacity in the pushover analysis was computed by taking into account an ultimate maximum rotation equal to 0.127 rad for I-shaped links. Based on the dynamic inelastic analyses, the rotation limit seems adequate for practical purposes under and might be a conservative approach of the actual capacity of the studied models.
- Through 1,756 certified laboratory coupons test of steel samples a better reference of the actual yielding steel stress was included in order to evaluate the material overstrength capacity at the local market. A strong dependency of the local market condition and values of material overstrength was underlined.
- A slight relationship among ductility and building height was noticed, something that it is not currently considered in building codes. The deformation capacity decreases as the height of the building increases. In contrast, overstrength factors were close than those proposed in the building code. Any dependency of the overstrength factor from the height of the building was found.

7. Acknowledgements

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