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IMPORTANCE OF DUCTILE CONFINEMENT IN REINFORCED CONCRETE FRAMES WITH STRUCTURAL FUSES

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Abstract

In this paper the authors summarize the results obtained, from static nonlinear analyses (pushover), for reinforced concrete intermediate moment-resisting frames (RC-IMRFs), designed with minimum ductile confinement requirements, and with hysteretic energy dissipation devices (HEDDs) mounted in chevron steel bracing which work as structural fuses.

Moment-resisting frames, ranged from 5 to 25 stories, were designed using diverse stiffness ratios between the different systems (frames, bracings and hysteretic devices). The first one is the stiffness ratio related between the frame system and the whole structure (α). The second stiffness ratio is between the HEDD and the supporting brace (β). A post to pre yielding stiffness ratio (k_2/k_{EL}) of 5% for the hysteretic devices was also considered, based on experimental results. Furthermore, the angle of inclination of the chevron braces with respect to the horizontal axis was taken into account from typical story heights and bay widths used into the Mexican construction practice.

The structural behavior of RC-IMRFs with these extra requirements is much improved with respect of the typical detailing established in Mexican codes for RC-IMRFs, favoring satisfactory design mechanisms where the energy dissipation devices develop most of the nonlinearity of the system (structural fuse), while the RC frames remain elastic or develop only incipient yielding exclusively for beams. Also, updated seismic global design parameters are proposed related with the ductility and overstrength that such structures could develop.

Keywords: Energy dissipation; metallic fuses; seismic control response; reinforced concrete frames; seismic parameters

1. Introduction

After the September 19, 1985 Michoacán Earthquake, one alternative that was attractive for Mexican engineers was the seismic response control, because it offers a better chance to assure life safety and full occupancy performance objectives of any structure after a major seismic event. There are several research studies related to "metallic fuses" applications, and much more about seismic response control, which their use could be dated since 100 years ago [1]. During the last decade, researchers have involved into diverse analytical and experimental projects. Some have demonstrated the efficiency of new geometrical configurations of passive energy dissipation devices [2]. Others investigators have assessed the use of new materials in order to create such devices [3], or explored new structural configurations for the dissipation device's implantation at supposedly lower costs than those already thoroughly studied. In recent years, new energy dissipation systems no necessarily located in concentric braced frames have been proposed based upon experimental research [4, 5]. Of course, there are studies available about the optimal placement to implement energy dissipation devices capable to mitigate the torsional and translational seismic effects in structures [6].

Probably, the original idea to study the elastic stiffness ratio between the bracing-hysteretic device system and the moment frame system for the design of structures with HEDDs was proposed for Ciampi *et al.* [7] and, in Mexico, the earlier work was reported by Esteva and Veras [8], who have emphasized that structural elements of the original structure cannot be described exclusively with their effective damping for systems with HEDDs. Some research studies have focused on assessing elastic stiffness ratios (supporting bracing system and main frame structure) for structures with HEDDs [9-13]. A wide but sparse range of parameters have been evaluated, so it is difficult to compare results directly. However, the reported global structural behaviors in those studies are similar, even though the models vary from single degree of freedom to multiple degrees of freedom systems. The reported results are encouraging to recommend engineers to design structures with HEDDs.

With the purpose to promote the construction of structures with HEDDs, the authors present a parametric study for RC-IMRFs with HEDDs mounted in chevron steel bracing devoted to assess global seismic design parameters to later adapt to design procedures commonly used for typical concrete structures. The addition of supplementary minimum ductile confinement in the design is proposed in order to improve the global structural efficiency. These requirements are specified in the reinforced concrete guidelines of Mexican codes for reinforced concrete special moment-resisting frames (SMRFs). The research is based on stiffness ratios between the frame system and the bracing-energy dissipation device system designated as α; and the parameter β, for the stiffness ratio between the bracing system and the HEDDs. This study takes advantage of findings reported in previous works [12, 13] and recommendations done by practicing engineers, in order to propose different structural geometric parameters, such as model's height and angles of inclination of the braces with respect to the horizontal plane.

2. Analytical models under study

Reinforced concrete plane frames with HEDDs mounted in chevron bracing have been analyzed. The geometric structural configuration corresponds to regular buildings, formed with bays of 8 meters and story heights of 4 meters. The analytical models ranged from 5 to 25 stories with changes in cross sections as the model increases in height (typical usage in Mexican structural engineering). Those section changes (beams, columns, braces, HEDDs) were avoided at the same story in order to minimize the possible formation of soft stories at intermediate levels. Also, the floor system was designed as a strong reinforced concrete slab for lateral loading and it was modeled as a rigid diaphragm in order to distribute the seismic actions.

The assessed methodology in this paper is based on previous research where different stiffness ratios α and β were evaluated [12, 13]. Three different cases for the stiffness ratio α between the frame and bracing-HEDD systems were evaluated: α=0.25, the frame system provides a smaller lateral stiffness to the complete structure than the bracing-HEDD system; α=0.50, the frame and the bracing-HEDD system have the same lateral stiffness; and α=0.75, the bracing-HEDD system is more flexible than the frame system. Besides, two different elastic stiffness ratios β were proposed for the HEDDs with respect to the braces; β=1.0, the bracing system has the same stiffness as the HEDD and; β=0.50, the HEDD has half the stiffness of the brace. For all the evaluated models, the seismic design base shear was defined as the 10% of the total structure weight (V/W=0.10).

The predesign methodology (based on stiffness ratios) has been reported in detail in previous works [12, 13]. In table 1 lateral stiffness values are presented for the different structural systems involved for the 10-story model. The changes in color for the columns of the table indicate the variations of cross sections as the model increases in height. The use of typical cross sections and their variations along the height do not permit a perfect match for target values of parameter α. Moreover, a wider variation of α is appreciated at stories where changes in cross sections are set. For practical purposes, variations of α larger than 10% were not allowed. Also, at least 70% of stories had to accomplish with the former premise in order to take the stiffness ratio as valid.

Table 1 – Lateral stiffness for the 10-story model, α=0.75, β=0.50 (units: ton/m)

Story	K_{Frame}	K_{Brace}	K_{HEDD}	$K_{Brace-HEDD}$	α
1	35,784.90	32,265.82	16,132.91	6,146.03	0.85
2	22,487.16	32,265.82	16,132.91	6,146.03	**0.79**
3	21,201.07	32,265.82	16,132.91	6,146.03	**0.78**
4	21,201.07	21,437.87	10,718.94	4,083.57	0.84
5	13,355.60	21,437.87	10,718.94	4,083.57	**0.77**
6	10,797.24	21,437.87	10,718.94	4,083.57	**0.73**
7	10,797.24	1,868.84	934.42	1,868.84	0.85
8	6,152.25	1,868.84	934.42	1,868.84	**0.77**
9	4,718.45	1,868.84	934.42	1,868.84	**0.72**
10	4,718.45	1,868.84	934.42	1,868.84	**0.72**

The lateral stiffness ratios were not the exclusive parameter in the design procedure; strength requirements were also taken into account for all the structural elements in accordance to the material guidelines of Mexican codes [14, 15]. For structures designed with the structural fuse concept, the nonlinear behavior has to be concentrated in the HEDDs; for this reason, the braces must remain elastic in the whole seismic event. Therefore, no damage should be experienced by the braces before the target displacement ductility demand is reached by the HEDDs. In order to warrant such premise, the slenderness ratios for the braces were reviewed and a safety factor of 1.5 was used for their design, following the recommendations by Enrique Martínez-Romero [16].

3. The importance of ductile confinement in reinforced concrete elements

The proposed stress-strain curve by Kent and Park was used in this paper. Based on experimental evidence, this constitutive model assumes that spacing of stirrups increases the concrete strength and its deformation capacity [17]. The additional ductility is given by the ε_{50h} variable; and this is related to the ratio between the volume of confined concrete core and the stirrups used. Besides, a bilinear curve with strain hardening was used for longitudinal bars.

Moment-curvature diagrams are compared in fig. 1 for a reinforced concrete column with different spacing of stirrups. Based on theoretical assumptions described by Kent and Park, the dashed line represents a non-ductile confinement with stirrups spacing of 15 cm, whereas the continuous line represents the spacing of stirrups with ductile confinement requirements in accordance to the Mexican reinforced concrete guidelines [14]. Providing concrete elements with the minimum requirements for ductile confinement increases slightly the strength, comparing it with a non-ductile confinement. Moreover, the limiting curvature is increased considerably. In consequence, the rotation capacity will be larger, as it is shown in inelastic demands mappings in following sections.

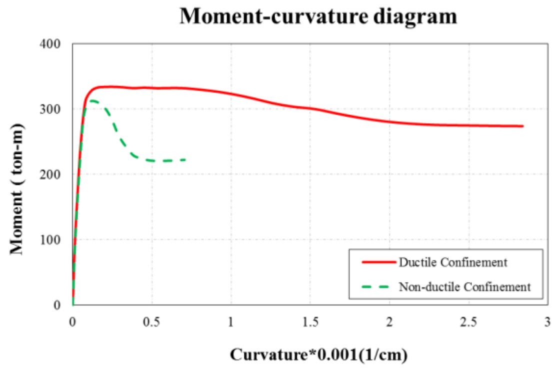
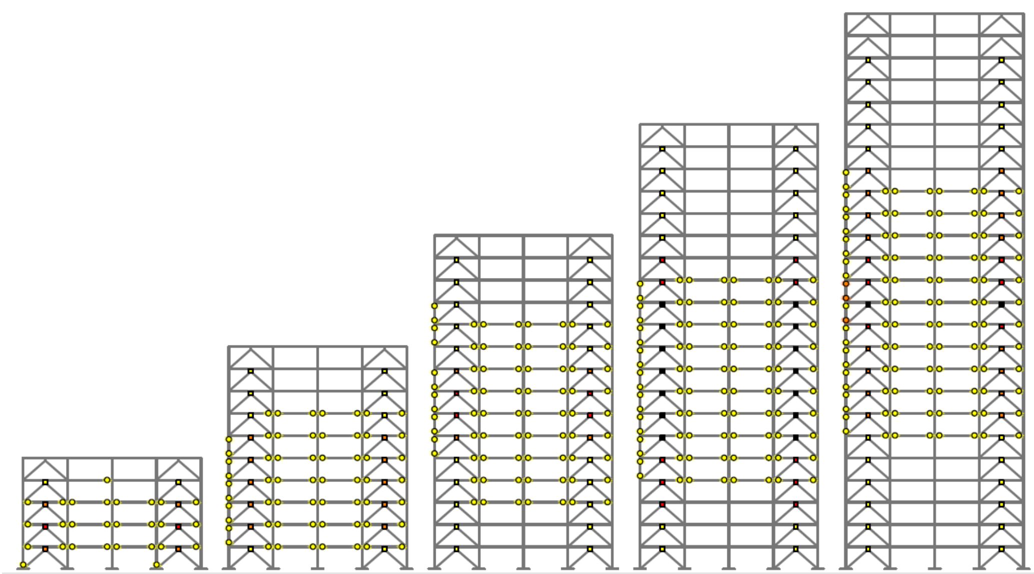


Fig. 1 – Moment-curvature diagram for different spacing of stirrups

The main objective for the addition of ductile confinement requirements in the design procedure for RC-IMRFs with HEDDs is to reduce the inelastic demands presented in the frame system (fig. 2), even for those optimal stiffness ratios among the systems involved. Moderate damage is associated to inelastic demands in beams and columns; therefore, the complete system does not fulfill the design philosophy of structures designed with the structural fuse concept. It is a prerogative that for structures with seismic response control, additional benefits from the seismic performance viewpoint should be achieved with respect to conventional structural systems. Therefore, the goal must be that all the damage should be concentrated in the HEDDs, and the frame elements must remain elastic or with an incipient nonlinear behavior during an earthquake.



a) α=0.25, β=0.50 b) α =0.25, β=0.50 c) α =0.25, β=0.50 d) α =0.50, β=0.75 e) α =0.50, β=0.75

Fig. 2 – Inelastic demands mapping for optimal stiffness ratios when Q≤4 [13]

4. Differences in the design procedure for confined concrete

Differences between non-ductile and ductile confinement requirements are evaluated, in order to compare the variations resulted from the design procedure. For this purpose, longitudinal reinforcement is the same for both kinds of confinements, such that the implemented additional confinement requirements are related to the diameter, spacing and number of overlapping stirrups used.

The diameter of the stirrups used is one of the most important differences in the confinement requirements. In NTCC-04 [14] as per section 2.5.2.2, it is established that for RC-IMRFs (non-ductile confinement), the minimum diameter to use in beams and columns must be number 2.5 (diameter 5/16 inch or 7.9 mm), inasmuch the shear forces are distributed in accordance to the stiffness ratios for designing the frame elements. Shear demands were satisfied in most cases with the use of stirrups number 2.5 (fig. 3), except in beams for the tallest models and stiffness ratio α=0.75.

There is a significant difference related to the smallest diameter to use for ductile confinement requirements in beams and columns. For beams, stirrups of number 2.5 could be used as the minimum possible diameter (sections 7.2.3 and 7.2.4.2). For columns, as per section 7.3.4, the minimum possible diameter for stirrups must be number 3 (diameter 3/8 inch or 9.5 mm). Schematically, different colors are used to identify each diameter of used stirrups in figs. 3 and 4; red for stirrups number 2.5, blue for stirrups number 3 and green for stirrups number 4 (diameter ½ inch or 12.7 mm).

In the case for columns, stirrups of number 2.5 were used for all the evaluated models with non-ductile confinement. In most cases, ductile confinement requirements lead to the use of stirrups number 3. Stirrups of number 4 were used in some stories in order to satisfy the minimum reinforcement ratio condition (section 7.3.4c). However, there are no differences in the number of overlapped stirrups, with or without the ductile confinement requirements (table 2). This similarity is established in section 6.2.3.3, specifying that stirrups shall be arranged in a way that each longitudinal reinforcement bar of the corner and one of each two consecutives from the periphery must have a lateral support provided by a closed rectangular stirrup or tie [14]. In the case for beams with non-ductile confinement, the use of the aforementioned described section is not specified. Because of this, beams with non-ductile confinement have less number of overlapped stirrups or ties (fig. 3) than beams with ductile confinement (section 7.2.3d), as it is shown in fig. 4.

Table 2 – Differences in confinement requirements for the 15-story model, α=0.25, β=0.50 (units: cm)

	Stories	**b**	**h**	**Confinement at end sections**		**Confinement at midspan**	
				Non ductile	**Ductile**	**Non ductile**	**Ductile**
Beams	B 1-5	60	90	4 No. 2.5 @ 30	6 No. 2.5 @ 15	4 No. 2.5 @ 35	6 No. 2.5 @ 20
	B 6-10	50	75	3 No. 2.5 @ 25	6 No. 2.5 @ 15	3 No. 2.5 @ 30	6 No. 2.5 @ 20
	B 11-15	40	65	3 No. 2.5 @ 30	4 No. 2.5 @ 15	3 No. 2.5 @ 35	4 No. 2.5 @ 20
Columns	C 1-5	90	90	6 No. 2.5 @ 15	6 No. 3 @ 8	6 No. 2.5 @ 30	6 No. 3 @ 15
	C 6-10	80	80	6 No. 2.5 @ 15	6 No. 3 @ 10	6 No. 2.5 @ 30	6 No. 3 @ 15
	C 11-15	70	70	4 No. 2.5 @ 12.5	4 No. 4 @ 10	4 No. 2.5 @ 25	4 No. 4 @ 15

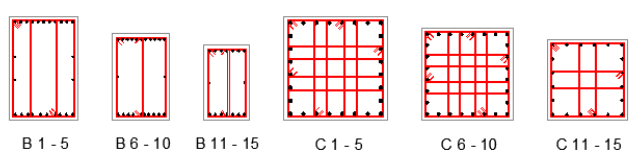


Fig. 3 –Reinforcement detailing for 15-story model, α=0.25, β=0.50, NON-ductile confinement

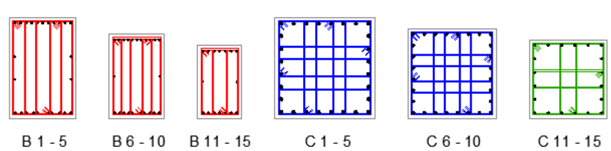


Fig. 4 – Reinforcement detailing for 15-story model, α=0.25, β=0.50, ductile confinement

Spacing of stirrups is the biggest difference between the use of ductile or non-ductile confinement. For beams with non-ductile confinement, this spacing is established in sections 2.5.2.2 and 2.5.2.3 [14]. Combined with the shear distribution condition between the frame system and the bracing-HEDD system, the spacing of stirrups for non-ductile frames could be wide enough in order to satisfy shear force demands (fig. 5). The section 7.2.3b defines the maximum spacing of stirrups at end zones for beams with ductile confinement; which in most cases is delimited by 24 diameters of the used stirrup bar. Therefore, for some stiffness ratios, the stirrups spacing could be reduced to the half, if ductile confinement requirements are used (fig. 6).

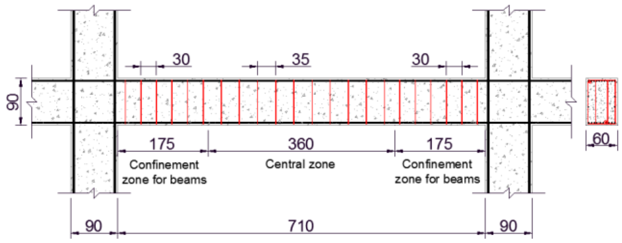


Fig. 5 – Layout of non-ductile confinement for beams, 15-story model, α=0.25, β=0.50, units: cm

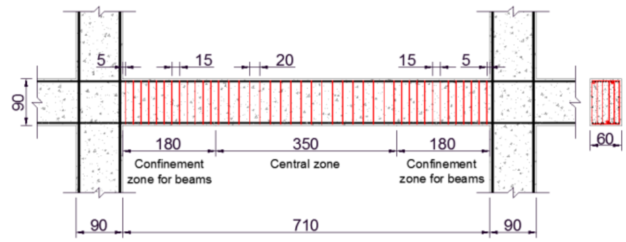


Fig. 6 – Layout of ductile confinement for beams, 15-story model, α=0.25, β=0.50, units: cm

The spacing of stirrups in columns with non-ductile confinement was defined by the requirement of maximum allowable spacing (fig. 7), which it is indicated in section 6.2.3.2 [14]. This was a consequence that their longitudinal reinforcement and large cross sections were enough to cover the shear force demand, as specified in section 2.5.1.3. In the case of the columns with ductile confinement (fig. 8), the spacing of stirrups was defined by section 7.3.4 [14], which establishes that it cannot exceeded one-fourth of the its minimum cross section dimension, or six diameters of the thickest longitudinal rebar, or 10 cm. In the particular case for this research, such spacing did not exceed from 10 cm for the condition of ductile confinement. Moreover, there were few cases where the spacing of stirrups was smaller than 10 cm (table 2), especially when reducing the spacing of stirrups was more efficient than increasing the diameter of the used transversal rebar or placing an extra overlapped stirrup.

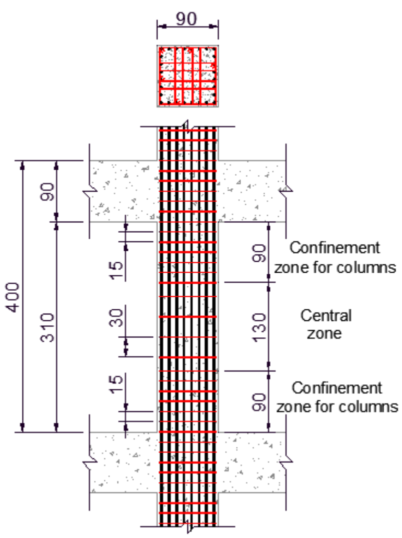


Fig. 7 – Layout of non-ductile confinement for columns, 15-story model, α=0.25, β=0.50, units: cm

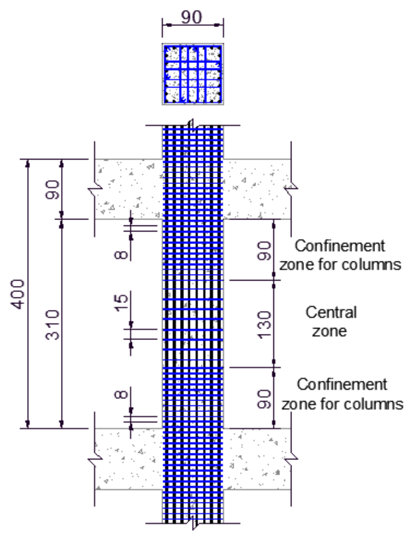


Fig. 8 – Layout of ductile confinement for columns, 15-story model, α=0.25, β=0.50, units: cm

One may suppose that the use of minimum ductile confinement requirements may increment significantly construction costs because of the spacing reduction of stirrups, increment of the minimum diameter for stirrups in columns or the number of required overlapping stirrups. For this reason, a quantification of construction costs was done, involving all elements of the frame system for all the models evaluated in this research. Weight differences were compared as first instance, caused for the using of ductile confinement. Also, a comparative of costs was done with all the frame system elements, including secondary beams and floor system.

A comparative of weights is summarized in table 3, for transversal and longitudinal rebars, with both confinement requirements. On average, the ductile confinement requirements lead to an increment of weight of 25% for the different evaluated models, although the weight variation ranged between 15% and 33%. The smaller difference was for the 15-story model with α=0.75, in spite of the fact that the number of overlapping stirrups increased for the beams and the diameter used for the stirrups in columns increased too.

A comparative of costs is also presented in table 3 for the different used confinement requirements. Current commercial prices were managed in the first semester of 2016, in the metropolitan zone of Mexico City. The cost for concrete with compressive strength f'_c=250 kg/cm^2 was \$105.51 (USD) per m^3; and for reinforcement bars with diameters from number 2.5 to 12, was \$703.67 (USD) per ton. For the different evaluated models, the costs variations increase as the ratio α also increases; this tendency is appreciated in table 3. For the 10-story models, the cost variation is barely perceptible, comparing the same confinement requirements for the different stiffness ratios. Moreover, using non-ductile confinement or ductile confinement has a cost variation of 10% (on average), being the tallest models where the biggest difference was observed.

Table 3 – Comparative of costs between non-ductile confinement and ductile confinement

Stories	Stiffness Ratio	Concrete (m^3)	Transversal + Longitudinal Reinf.(ton)		(%)Weight	Concrete + Reinforcement Bar ($ USD)		(%) Price
			Non-ductile	Ductile		Non-ductile	Ductile	
5	α=0.25	809.4	70.4	84.5	120.1%	$138,429.90	$149,088.32	107.70%
	α=0.50	858.8	79.0	96.9	122.6%	$150,179.39	$163,621.12	108.95%
	α=0.75	930.4	92.8	116.3	125.3%	$168,098.30	$185,813.53	110.54%
10	α=0.25	2275.5	189.6	251.8	**132.8%**	$383,013.31	$429,862.31	112.23%
	α=0.50	2180.8	201.5	254.4	126.3%	$381,943.74	$421,805.78	110.44%
	α=0.75	2275.5	190.6	224.2	117.7%	$383,709.50	$409,062.82	**106.61%**
15	α=0.25	3535.7	329.7	414.1	125.6%	$621,574.18	$685,141.84	110.23%
	α=0.50	3450.7	377.2	476.2	126.3%	$648,362.89	$723,007.71	111.51%
	α=0.75	3450.7	428.2	492.1	**114.9%**	$686,827.62	$734,953.79	107.01%
20	α=0.25	5552.7	793.9	948.1	119.4%	$1,184,237.98	$1,300,431.79	109.81%
	α=0.50	6286.4	859.9	1036.8	120.6%	$1,311,396.20	$1,444,711.68	110.17%
	α=0.75	6286.4	898.8	1110.1	123.5%	$1,340,732.53	$1,499,944.70	111.88%
25	α=0.25	7931.9	1307.7	1619.6	123.9%	$1,822,500.40	$2,057,608.44	**112.90%**
	α=0.50	8049.4	1347.2	1647.4	122.3%	$1,864,644.05	$2,090,910.51	112.13%
	α=0.75	8960.5	1496.3	1844.0	123.2%	$2,073,194.22	$2,335,236.46	112.64%

5. Nonlinear static analysis

Based on results obtained from nonlinear static analysis (pushover), comparatives among local ductilities developed by the HEDDs and capacity curves and inelastic demands mapping are presented. This was done in order to analyze if ductile confinement requirements improve the global structural efficiency for the different stiffness ratios of the models under study.

Displacement ductility demands for the HEDDs in frames with ductile confinement are presented in fig. 9. The vertical dotted lines represent the range where the dissipation devices could be considered efficient ($8 \leq \mu_d \leq 12$). As the stiffness of the HEDD is smaller than the bracing system (i.e., β=0.50), local ductility demand increases. As the evaluated models become taller, the stiffness ratio α must increase in order to develop an appropriate displacement ductility for the HEDDs (α=0.50 and α=0.75). Given that in the design procedure large cross sections were obtained for the bracing system when α=0.25, the complete structure tends to be stiffer. As a consequence, HEDDs cannot develop ductility demands greater than $\mu_d \geq 8.0$, regardless of the used β ratio.

Normalized global lateral shear vs drift curves are plotted in fig. 10. Overstrength can be assessed at the y-axis, as it is defined as the ratio greater than one between the developed shear and the design shear. The drift angle (percentage) is depicted at the x-axis. Three different curves are plotted in order to compare the variation of the stiffness ratio α. The structural deformation capacity increases as α decreases. Differences in elastic stiffness are appreciated as well; when α increases, the elastic stiffness is reduced for the complete structure (fig.

10). Otherwise, when α decreases, the overstrength increases because larger cross sections for columns were obtained in the design process as a consequence that higher axial loads were transmitted to the columns by the bracing system when α=0.25.

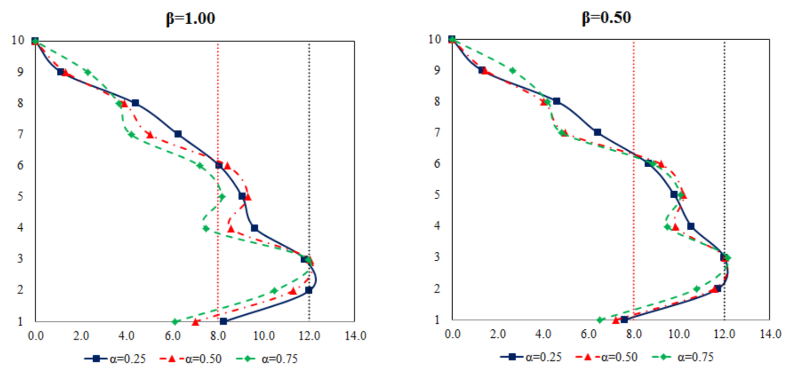


Fig. 9 – Ductility demands (μ) for the HEDDs for the 10-story model with ductile confinement

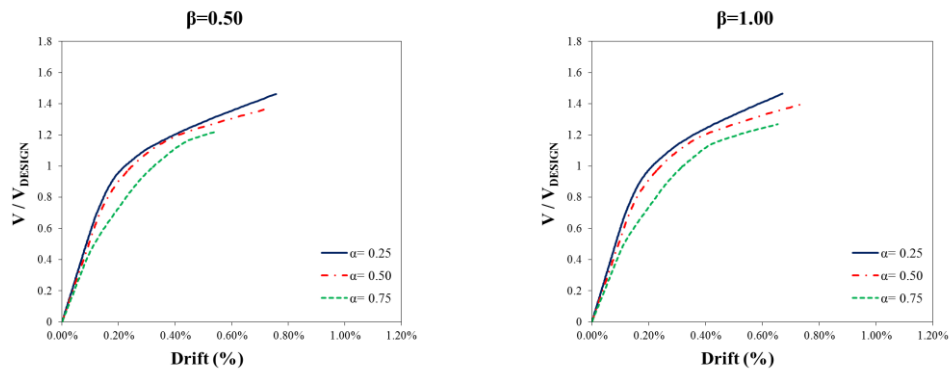


Fig. 10 – Normalized global lateral shear vs drift curves for the 15-story model with ductile confinement

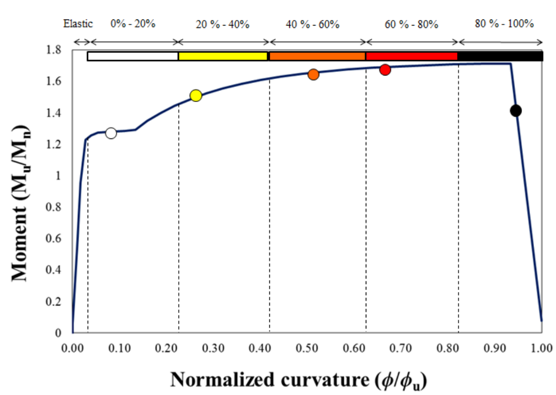
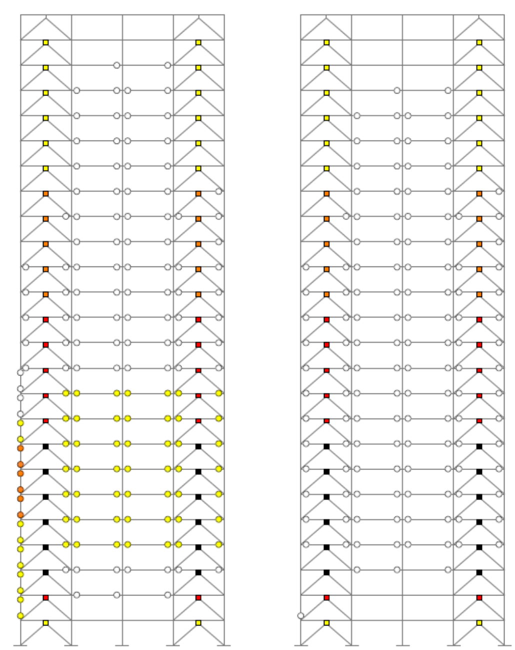


Fig. 11 – Color intensity scale for the inelastic responses for beams and columns



a) Non-ductile confinement b) Ductile confinement

Fig. 12 – Comparative of inelastic demands mapping for the 25-story model, α=0.50, β=0.50

Inelastic demands mapping is the best option in order to compare the confinement parameter. A schematic color intensity scale is represented in fig. 11 for the nonlinear behavior in concrete elements. The white dot represents an incipient yielding; the black dot represents the ultimate state (plastic hinge); the yellow, orange and red are intermediate points between the incipient yielding and the failure of the element. Comparatives of inelastic demands mapping about confinement requirements are presented in fig. 12. Notorious differences are observed in the inelastic response for frame elements. An almost perfect elastic response for beams and columns (incipient yielding at some beams only) is obtained using ductile confinement requirements (fig 12b), whereas intermediate yielding (damage) in beams and columns is obtained for non-ductile confinement (fig 12a).

It is worth noting that as the proposed color intensity scale is normalized with respect to the ultimate curvature of each element (fig. 11), the actual color range for the curvatures of ductile and non-ductile confinement are different (fig. 13). Although the yielding rotation of the columns increases slightly, the strength of elements increases by about 10%, depending on changes at spacing of the stirrups, the diameter and number of overlappings used. For most columns with ductile confinement, the white shade covers up the failure of the element with a non-ductile confinement (fig. 13). In fact, for columns, inelastic responses were faded away using ductile confinement and they remain elastic, even for the tallest model (fig. 12b), as consequence of the increase in strength and the remarkable increase in rotation capacity of the elements.

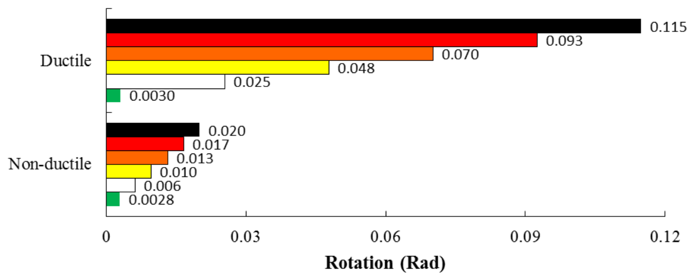


Fig. 13 – Rotation capacity for a column with different confinement requirements

6. Recommended stiffness ratios

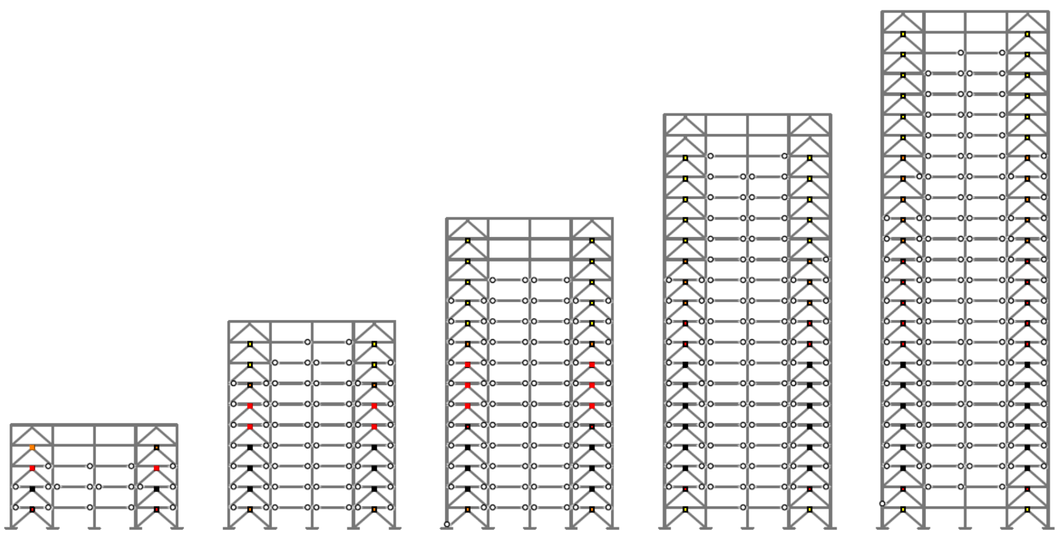
Recommended stiffness ratios for the different structural systems are reported in table 4. Two different color shades were used in order to identify the damage level in the frame system while maximum displacement ductility capacities were developed by the HEDDs. No color shade represents an elastic behavior for columns and incipient yielding at some beams. Yellow shade identifies tolerable nonlinear behavior in some beams and columns. Using ductile confinement requirements increase the number of possible combinations to use among the designed models with the structural fuse concept.

For the tallest models with α=0.25 ratio, nonlinear behavior was observed in some external columns. Using the stiffness ratio when the frame system is more flexible could lead to larger structural deformation capacities compared to the others evaluated ratios; but the damage level related to the rotation capacity at beams and columns could be much larger also. Thereby, for structures with slenderness ratio $H/L \geq 2.0$, the use of this stiffness ratio it is not recommended in order to fulfill the design philosophy for structures with seismic response control.

Inelastic demands mapping are presented in fig. 14 for the models with apparently the “best” structural performances. In order to set the recommendation for stiffness ratios, the local ductility developed by the HEDDs and the global structural deformation capacity and inelastic responses were taken into account. Columns remained elastic and incipient or barely perceptible yielding was observed for beams. Comparing the mappings presented in fig. 14 (ductile confinement) with those presented in fig. 2 (non-ductile confinement), the importance of minimum ductile confinement requirements are appreciated in the structural efficiency of RC-IMRFs with HEDDs. This additional detailing is a viable option currently used by some Mexican structural design firms [18], in order to avoid the rigorous procedure specified for RC-SMRFs contained in Mexican reinforced concrete guidelines [14].

Table 4. Recommended values for the structural parameters for RC-IMRFs with HEDDs and ductile confinements for beams and columns

Stories	*α*	*K₂*	*β*	*μ*
5	0.25	0.05	0.5 - 1.0	10-8
	0.50		0.5 - 1.0	10-6
	0.75		0.5 - 1.0	12-8
10	0.25	0.05	0.5 - 1.0	12-8
	0.50		0.5 - 1.0	10-8
	0.75		0.5 - 1.0	10-8
15	0.25	0.05	0.5 - 1.0	12-8
	0.50		0.5 - 1.0	12-8
	0.75		0.5 - 1.0	12-8
20	0.25	0.05	0.5 - 1.0	12-8
	0.50		0.5 - 1.0	12-8
	0.75		0.5 - 1.0	12-8
25	0.25	0.05	0.5 - 1.0	12-6
	0.50		0.5 - 1.0	12-8
	0.75		0.5 - 1.0	12-10



a) α=0.25 *b) α=0.25* *c) α=0.25* *d) α=0.50* *e) α=0.50*

Fig. 14 – Inelastic demands mapping for the "optimal" stiffness ratios with ductile confinement, β=0.50

7. Updated seismic parameters

Based on results obtained from all the evaluated models, some global seismic design parameters are proposed in accordance to Mexican codes [19]. As mentioned in the previous section, the recommended stiffness ratio for models with slenderness ratio H/L<2.0 were α=0.25; for models with H/L≥2.0 (tallest evaluated models), the recommended stiffness ratio were α=0.50. In figures 15 and 16, recommended stiffness ratios were marked with red circles, for practical and fast visualization.

Overstrength factors are reduced as the slenderness of the models are increased (fig.15). This parameter tends to 1.5 as the assessed models were taller, regardless the stiffness ratio α. Also, the overstrength factor is reduced as the stiffness ratio α increases (except for the 5-story models). An overstrength factor Ω=1.5 is a plausible average value to use for reinforced concrete structures with HEDDs.

Assessed seismic response modification factors *Q* (global deformation capacity) are shown in fig. 16. This parameter tends to reduce as the models increase in height. Besides, a smaller *Q* value was obtained using a stiffness ratio α=0.75 (*Q*≈3.0), regarding the other proposed stiffness ratios. A factor *Q*=4.0 could be a reasonable value in order to design reinforced concrete frames with HEDDs. These proposed seismic factors shall be used in structures that satisfy the regularity conditions established in seismic codes (i.e., H/L≤2.5).

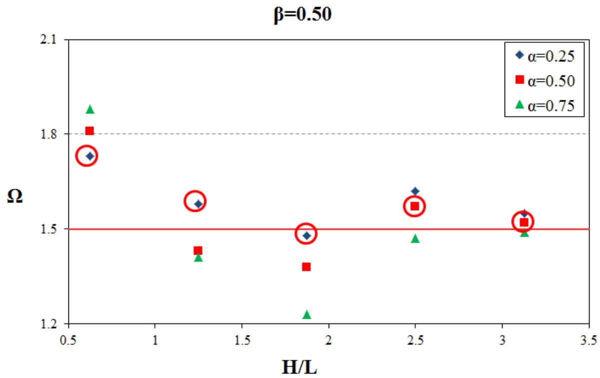


Fig. 15 – Overstrength factor Ω

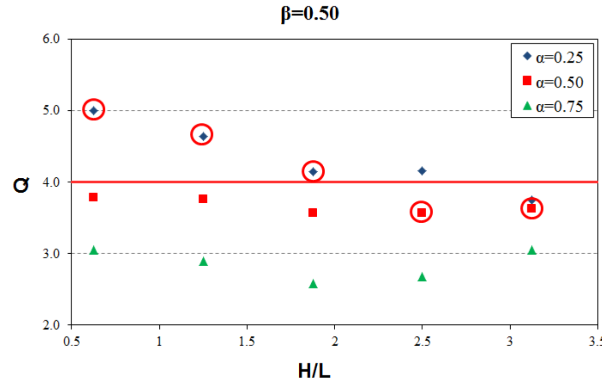


Fig. 16 – Seismic response modification factor *Q*

8. Concluding remarks

The importance of ductile confinement requirements for RC-IMRFs with HEDDs were studied in this paper. Improvement in structural behavior was observed with the implementation of this additional detailing procedure in agreement with Mexican reinforced concrete guidelines [14]. Differences between non-ductile confinement and ductile confinement are focused on spacing of the stirrups, the number of overlappings and the minimum diameter to use for stirrups. Reducing the spacing of the stirrup is the most relevant parameter in order to increase the rotation capacity for concrete elements. Besides, increasing the number of overlappings and the stirrup diameter contributed as well to reduce inelastic demands at beams and columns while maximum displacement ductility capacities are developed by the HEDDs.

An almost perfect elastic response for beams and columns were obtained using ductile confinement requirements. Incipient yielding was developed by some elements of the frame system, mainly beams. The columns did not develop inelastic demands with the recommendation of stiffness ratios proposed for the "best" structural performances. The range for "optimal" stiffness ratios to use for RC-IMRFs with HEDDs was increased with the additional detailing in concrete elements, and the concern for a possible structural damage after an earthquake is almost completely dissipated. As the models increase in height, the stiffness ratio α provided by the frame system shall be larger, in order to avoid possible moderate damage in the reinforced concrete elements. The importance of β parameter (stiffness ratio between HEDDs and braces) is primarily related to the maximum displacement ductility demands developed by the HEDDs. Overall, as the β ratio is reduced, the local ductility demands developed by the HEDDs tend to increase and, as consequence, larger inelastic demands at some columns and beams are observed.

Global seismic design parameters have been proposed to Mexican seismic codes, in order to encourage the design of such structures using procedures commonly known and used by Mexican practicing engineers. Overstrength factors are reduced as the slenderness of the models is increased. An overstrength factor Ω=1.5 is a plausible average value to use for reinforced concrete structures with HEDDs. Seismic response modification factors *Q* tends to reduce as the number of stories increase. From a practical viewpoint, a factor *Q*=4.0 is reasonable to use in RC-IMRFs with HEDDs.

Finally, using ductile confinement requirements did not seriously impact constructions costs. A comparative of costs was presented for the different stiffness ratios under study. For all the models under study, the ductile detailing increased the costs between 6% to 13% (an average of 10%); the biggest differences were obtained for the tallest models. Therefore, the real economic impact could be smaller since the costs of braces and dissipation devices were not included (same cross sections for both kind of confinement); the costs for the foundation would also be the same, given that the weight of the frame system would not increase in a transcendental manner and seismic forces are essentially the same for both confinement options.

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