SEISMIC BEHAVIOR OF EXISTING MID-RISE MOMENT RESISTING FRAMES AT MEXICO CITY SOFT-SOIL SITES

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Abstract

Mexico City, due to its location and soil conditions, is one of the areas with the greatest seismic hazard in the country, which implies a significant risk in construction. Illustrative examples of this are the severe damages caused by the earthquakes occurred on September 19th, in 1985 and 2017, where a large number of structures suffered damage, and even collapse in many low-to-medium-rise buildings.

With the objective of mitigating future damages caused by earthquakes, this study evaluates the behavior and determine the seismic structural vulnerability of buildings similar to those damaged by the earthquake of September 19th, in 2017, so that the corresponding measures may be taken before the occurrence of a future seismic event.

Based on the above, in this work the structural performance of reinforced concrete moment resisting frame structures are studied considering a soft-storey and half-height masonry walls of 5 and 7 levels with 2 and 3 bays, respectively. The studied structures are assumed to be dwellings located in soft-soil sites (*i.e.*, Zone III of Mexico City according to the Mexico City Building Code published in 2017). The structures under study are designed using previous seismic design regulations of the Mexico City Building Code (*i.e.*, 1977, 1987 and 2004) given that a large majority of existing buildings were designed based on these previous codes. Two case studies are evaluated for each of the structures: the first considers walls linked to the structure, while the second takes into account walls separated from it, in order to reveal their contribution to the structure, in addition to analyze their response capacity to lateral forces. This response capacity is defined in terms of their dynamic capacity curves, obtained from Incremental Dynamic Analysis, IDA, using actual seismic records representative of the study area.

Likewise, this study will give the opportunity to estimate the expected structural damage, and thereby propose appropriate mitigation measures to reduce the negative effects caused by a future seismic event for existing structures. For this reason, the occurrence of an earthquake of magnitude Mw8.2 originating in the Guerrero Gap is assumed to occur. The obtained results will contribute to generating actions aimed to mitigate those possible damage caused by seismic events, which will be reflected in an increase in the resilience of society and a better use of the scarce economic available resources.

Keywords: seismic risk, existing structures, weak-first storey structure, building codes, Mexico City

1. Introduction

Buildings in general may suffer diverse type of damage when are subjected to seismic excitations; however, such damage is not necessarily the same for all buildings, due to several factors such as: structural system, quality of construction, maintenance, irregular characteristics from a structural standpoint, etc. From the experience of past earthquakes, such as the 1985 and 2017 in September 19th, it was observed that most of the generated damage was due to the architectural and structural configuration on plant [1]. From these experiences, Mexican engineering has considerably understood the structural behavior oriented to define the seismic demands, material properties, safety factors and coefficients employed in the design, with the objective of establishing more rigorous safety standards.

The 1985 earthquake occurred at 7:17 am with a magnitude Mw8.1, in front of the Michoacan coast in the subduction zone, 400 km away from Mexico City, with several aftershocks, being the most significant that one of the September 20th same year with a Mw7.6. The balance of both events was about 20 thousand victims [2], 210 buildings collapsed totally or partially, some of them with more than 15 levels (*e.g.* Pino Suarez building, Nuevo León building, La Superleche coffee building, Hotel Regis, etc.) [3]. Furthermore, approximately 5,025 buildings suffered several type of damage, this without considering one or two storey dwellings. These buildings were classified into two groups for administrative reasons: i) buildings with more than four levels (1,205), supervised by the General Ministry of Works and ii) buildings with four levels or less (3,820), which represent the majority of structures based on masonry walls, supervised by the local Government Delegations[4].

Thirty-two years after the large Mw8.1 earthquake that shocked the capital of the country, at 1:14 pm, Mexico City was shaken again by a seismic event of Mw7.1 with epicenter located between the states of Puebla and Morelos, 120 km away from the capital of the country. This event produced strong ground shaking causing the death of 228 people and the collapse of 38 buildings, besides severe damage in other 1,500 buildings [5].

In order to minimize the negative consequences that an earthquake might produce in urban or rural settlements, such that produces by the Mw7.1 earthquake in September 19th of 2017 in Mexico City, in this paper is studied the behavior of mid-rise reinforced concrete moment resisting frames of 5 and 7 levels, as those type of building collapsed during the aforementioned seismic event. To achieve this goal, the geometrical and mechanical characteristics were inferred through their year of construction and later on, step-by-step non-linear analyses were carried out to construct their vulnerability functions so that a probabilistic seismic risk analysis was performed. With the obtained results it will be possible to understand and, therefore, mitigate the negative effects that a large earthquake might generate such as those presented during the occurrence of the Mw8.1 in 1985 and Mw7.1 in 2017 earthquakes in Mexico.

2. Methodology

To estimate the structural behavior and be able to assess the seismic risk in the thousands of building that are constructed in any region with high seismicity, the following procedure is proposed:

2.1 Seismic hazard

In this study, hazard is defined as a stochastic set of events, collectively exhaustive and mutually exclusive, that describes the spatial distribution, the annual frequency, and the randomness of the hazard intensity at the site of interest. Seismic hazard intensity is quantified in terms of the relevant seismic intensity related to the performance of the structures. In this case, the selected intensity measure is the spectral pseudoacceleration corresponding to the first-mode period of vibration. The hazard intensity of each estimated seismic event is represented as a random variable by, at least, its first two probabilistic moments: (1) the expected value, and (2) the variance. The uncertainties considered for the variance estimation must be those related to the used input data and the simplifications of the models. There are no formal approaches to evaluate these uncertainties; however, the analyst should make an effort to estimate and include them in risk assessment. This process is completed by reviewing historic events and previous scientific studies performed on the severity and frequencies of earthquakes in the region of interest.

2.2 Structural response under seismic actions

Historically, earthquakes have caused a large amount of damaged buildings, even if the design code criteria were meet for the 1977, 1987 and 2004 Distrito Federal Construction Codes (RCDF from Spanish) [6, 7, 8], so it is important to estimate the structural response of buildings.

The main goal of structural seismic engineering is to maintain structural integrity while keeping the people safe during the occurrence of an earthquake. When such events occur, the ground motions cause displacements on the structure which, in turn cause internal forces and deformations that could lead to failure if the design actions are exceed. Thus, it is important to study the structural response as it represents the behavior of a structure exposed to an external excitation (e.g. earthquakes) in terms of the forces, stresses, strains, accelerations, velocities and displacements. To obtain the structural response, it is recommended the use of nonlinear step-by-step analysis such as an incremental dynamic analysis (IDA) [9].

2.2.1 Definition of structural characteristics

To estimate the structural response, it is required to define the structural and geometrical characteristics, such as: inter-storey height, number of stories and spans, use and location of the structure, as well as the material properties. However, this information is not always available, therefore, when the engineering draws or the structural design are missing, the gross structural characteristics may be obtained through a structural design using the construction codes relevant to the location and construction year of the analyzed structure. With this information, it is possible to define the hysteretic behavior model that represents the best the response of each structural element, in the nonlinear range, such as the dissipated energy due to the cumulative damage in terms of displacements and mechanical elements (e.g. bending moment of shear force).

2.2.2 Non-linear behavior model

Current design codes allow the structures to enter the inelastic range in some degree, thus it is important to define the non-linear behavior of the elements when exposed to external excitations such as earthquakes. A hysteretic behavior model of each structural element must be defined to this end.

2.3 Seismic vulnerability functions

The vulnerability function quantifies the expected damage caused to each asset class by the intensity of a given seismic hazard. The classification of the assets is based on a combination of construction material, structural system, building use, number of levels, employed building code, among other characteristics that will define the structural behavior. To have a better approximation of the actual structural behavior, there should be a vulnerability function corresponding to each building typology.

Structural vulnerability means damage that a specific property will incur if a hazardous event occurs. It is generally measured as an average percentage of damage or the economic value required to repair the impacted property and restore it to a state equivalent to its state prior to the occurrence of the event. Vulnerability is expressed in terms of "vulnerability functions". These express the distribution of loss values as a function of the intensity produced during a specific event. They are expressed as curves associating the value of expected damage and the standard deviation of the damage with the intensity of the hazardous phenomenon. Vulnerability functions for structures are expressed as curves associating mean damage ratio, also expressed as β, with a measure of event intensity. The standard deviation of the latter parameter, as well as the intensity function for the event, should also be taken into account [10].

2.4 Probabilistic risk assessment

The risk due to natural hazards is commonly expressed in terms of the expected annual loss, *EAL*, which specifies the frequency, usually annual, of the occurrence of the losses [11, 12] and the loss exceedance rate,

$\nu(\beta)$. For the computation of this indicator, two uncertainties are considered: that one around the occurrence or non-occurrence of unknown seismic intensities and a second one, which is the uncertainty of the size of losses given that specific event has occurred. This second uncertainty is employed to take into account that identical events can cause different amount of losses, resulting in a range of possible values with different probabilities. This uncertainty also reflects that structures with the same characteristics affected by the same event could have different loss level. In order to take into account this uncertainty, a standard deviation associated to the expected damage should be defined.

2.4.1 EAL

Accepting that each hazard type is defined by *EN* events that are collectively exhaustive and mutually exclusive, the *EAL* for any hazard can be estimated with the following expression:

$$EAL = \sum_{i=1}^{EN} E\left(Loss_i\right) P_A\left(i\right) \tag{1}$$

where $E(Loss_i)$ is the expected loss that an event i causes to the exposed asset, and $P_A(i)$ is the annual occurrence probability of event i.

To compute the *EAL*, any of the many open access software tools to compute losses due to natural hazards that have been developed and distributed around the globe can be used, an example of this platform is CAPRA [13] that computes losses based on a probabilistic approach.

3. Case Study

3.1 Seismic hazard in the Valley of Mexico

In this study seismic hazard for Mexico City (CDMX from Spanish) is defined from a set of seismic records obtained near the studied site and corresponding to the 19 September 1985 and 2017 earthquakes. Such seismic events were selected due to the high socioeconomic impact in terms of damaged buildings and life losses (Table 1).

Table 1 – Earthquakes and accelerometric stations used

Earthquake event	Station	Station code
19 September 1985	Sismex Viveros	SXVI
	Tláhuac Bombas	TLHB
	Tláhuac Deportivo	TLHD
19 September 2017	Cibeles	CI05
	Culhuacán	CH84
	Jardín de Niños Luz García Campillo	GC38
	Escuela Secundaria Técnica No.95	IB22
	Parque Jardines de Coyoacán	JC54
	Liconsa	LI33
	Lindavista	LV17
	Miramontes	MI15
	San Simón	SI53
	Unidad Colonia IMSS	UC44

3.2 Studied buildings

3.2.1 Structural characteristics

An important percentage of the collapsed buildings during the 1985 and 2017 September earthquakes (about 91.5% and 100% respectively) were reinforced concrete moment resisting frames with medium height [14]. Several of them had irregularities on elevation such as soft storey or short column effect due to medium height masonry walls that were not considered during the design process.

Thus, this study focuses on housing buildings of 5 and 7 storeys, constructed with reinforced concrete moment resisting frames, with an inter-storey height of 3 m in the low level and 2.4 m in the following floors and a span length of 5 m. From the second level on, medium height masonry walls were considered, thus a soft storey is generated in the first floor (Fig. 1). It is assumed that the buildings were designed according to RCDF 1977 [6], 1987 [7] and 2004 [8] using the seismic actions related to the zone with the higher damaged buildings during the 2017 earthquake, the zone IIIa [8].

Three study cases were defined to account for the potential effects of the medium height masonry walls: 1) infill masonry walls, IMW, 2) reinforced concrete frame, RCF, and 3) linked columns-masonry, LCM. The LCM case considers the short column effect, caused by the constrain of the elements due to the masonry, hence the increase in the shear forces could lead to a sudden failure of the element.

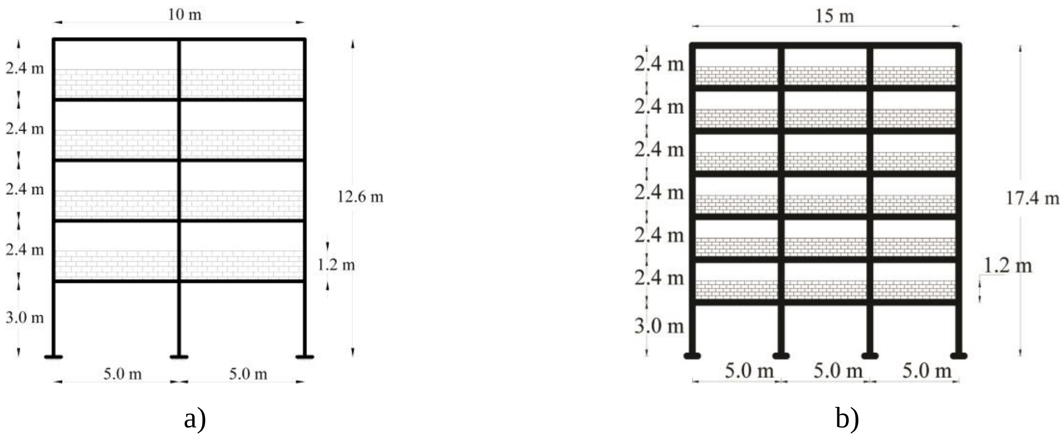


Fig. 1 – Geometric configuration of the buildings: a) 5 storey and b) 7 storey

Concentrated plasticity was considered in both ends of beams and columns, assuming a plastic hinge that follows a hysteretic behavior represented for the modified Ibarra-Medina-Krawinkler model [15]. A modal damping of 5% respect to the critical was assumed in the modeling. Considering that the structures are low-rise, the soil-structure interaction was neglected. The building codes were selected by carrying out an analysis considering all building code versions in Mexico City, since 1942 until the 2004 version. All modifications between codes were identified such as changes in classification of structures, interstorey drift limits, seismic behavior factors, and seismic design demand, among others. From this review, three building codes were identified with the most important differences: 1977 [6], 1987 [7] and 2004 [8] Building Codes. For instance, in those building codes, interstorey drift limits changed from 0.008 to 0.006, the number of seismic behavior factors (Q) increased (the 1977 building code defined only Q values of 2 and 3, while 1987 and 2004 building codes considered 2, 3, and 4) and the expressions to define the seismic design spectra were also modified. Taking into account these differences, considering Q = 2 and a damping ratio respect to the critical equal to five, the seismic design demand employed for the structures are those showed in Fig. 2.

Figure 2 shows the inelastic pseudoacceleration spectra obtained from the Mexican Design codes RCDF 1977, 1987 and 2004 [6, 7, 8]. There is an important difference between the spectral ordinates values associated with the 1977 code and the 1987 and 2004 codes [6, 7, 8], as the older design criteria shows pseudoaccelerations of approximately half of the other codes. One of the main reasons is the addition of an irregularity factor in the 1987 and 2004 codes [7, 8].

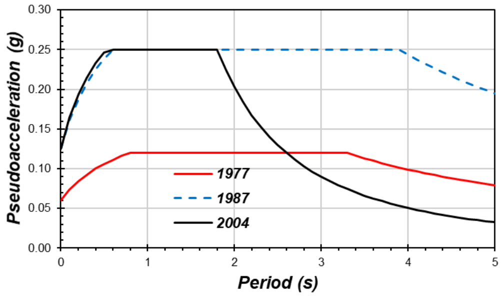


Fig. 2 – Pseudoacceleration design spectra

The specified compressive strength of all concrete elements is 250 kg/cm^2. Table 2 shows the section geometry of the structural elements associated with each building. It is noted that the recent design codes show an increased transverse section geometry of about 14% in columns and 11% in beams (the latter only for the 7 storey buildings).

Table 2 – Section geometry for each building

Code version (RCDF)	Floor number	Columns [cm]	Beams [cm]
1977	5	35×35	30×45
	7	35×35	30×45
1987 and 2004	5	40×40	30×45
	7	40×40	30×50

After performing an eigen-value analysis, the vibration period of each structure were obtained (Table 3). It was found that the older buildings (i.e. 1977 code) are 26% more flexible than those designed with recent construction codes (e.g. 19987 or 2004 codes). On the other hand, five storey buildings were found to have a 29% increased stiffness compared to seven storey structures. Finally, it should be noted that LCM and IMW structures presented only a 15% increase in stiffness compared to the RCF structure.

Table 3 Fundamental vibration period, structures built with RCDF 1977, 1987 and 2004 [6, 7, 8]

Structure	Structural period 1977 T [s]	Structural period 1987 T[s]	Structural period 2004 T[s]
5 IMW	0.68	0.52	0.52
5 RCF	0.79	0.60	0.61
5 LCM	0.68	0.52	0.53
7 IMW	0.91	0.70	0.70
7 RCF	1.10	0.75	0.75
7 LCM	0.90	0.63	0.63

3.2.1.1 Non-linear behavior model

As previously mentioned in the methodology section, the nonlinear behavior of the structural elements (beams and columns) must be defined. For this purpose, a concentrated plasticity model was used, where the non-linearity is lumped in zero-length spring at member ends. Also the modified bilinear Ibarra-Medina-Krawinkler [15] was used to represent the non-linear behavior. A section analysis (moment-rotation analysis) was performed in order to find the parameters needed to develop the hysteresis model (i.e., elastic stiffness, yielding strength, post-yielding stiffness, capping strength, ductility). The masonry walls were modeled as equivalent section columns.

3.3 Structural Response

This section shows the results of the non-linear analysis, as a plot of base shear versus roof displacement. Fig. 3 shows the seismic behavior of the five storey buildings according to the records corresponding to the CH84 station. The maximum displacement was observed in the E-W direction for all structures (Fig. 3e and 3f). While the building design with the 1977 code [6] presented a 12 cm maximum displacement, in the "newer" buildings (design with the 1987 and 2004 codes) [7, 8] it was only 4 cm. In the same way as the previous case, the older structures exposed inelastic behavior while the newer stayed in the elastic range.

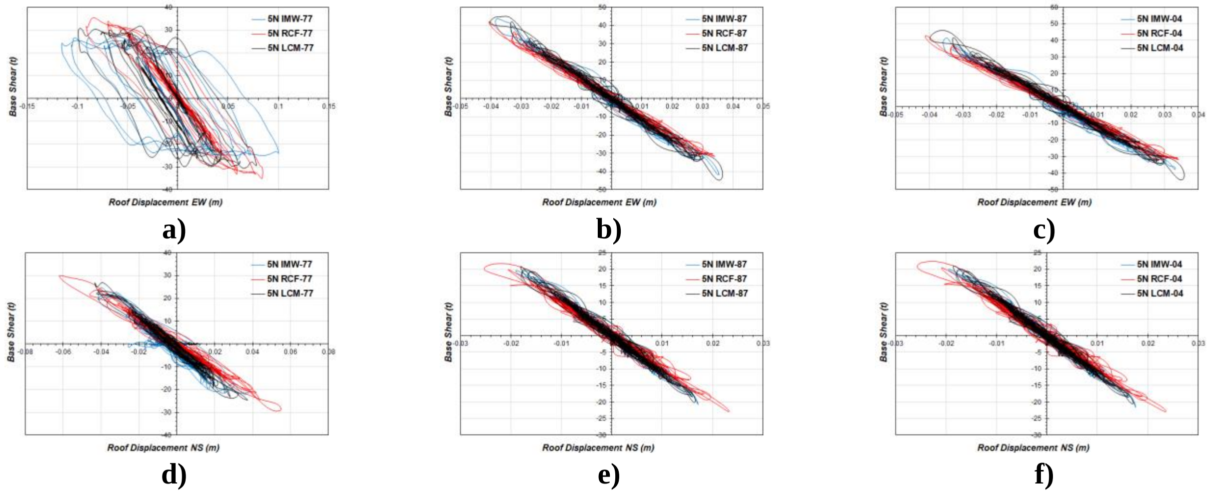


Fig. 3 – Seismic behavior of five storey structures analyzed with the seismic records of station Culhuacán, designed with the following codes: E-W direction a) RCDF-1977, b) RCDF-1987 and c) RCDF-2004 and N-S direction d) RCDF-1977, e) RCDF-1977 and f) RCDF-2004

After performing the non-linear analysis of the seven-storey structures using seismic records of the CH84 station, it was found that the structures built with the 1977 code [6] showed larger damage. They fall into the inelastic range in both directions (Fig. 4a and 4d), where the maximum displacements for the E-W and N-S directions were 20 and 12 cm, respectively. While the buildings designed with the 1987 and 2004 codes [7, 8] presented a 6 cm displacement in both directions (Figs. 4b, 4c, 4e, 4f).

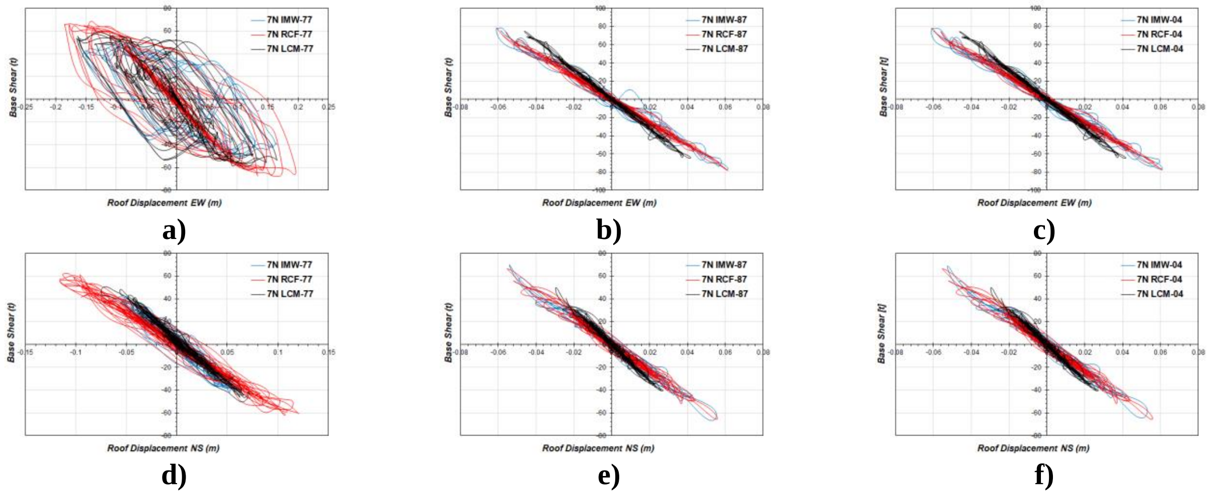


Fig. 4 – Seismic behavior of seven storey structures analyzed with the seismic records of station Culhuacán, designed with the following codes: E-W direction a) RCDF-1977, b) RCDF-1987 and c) RCDF-2004 (DDF, 1977a, DDF, 1987a and GDF, 2004a) and N-S direction d) RCDF-1977, e) RCDF-1977 and f) RCDF-2004

3.4 Seismic vulnerability

To define seismic vulnerability, the methodology presented by [16] was used, consisting on the following steps: 1) define seismic hazard, 2) define structural characteristics and typology along with the nonlinear properties, 3) perform IDAs, 4) define a damage model and obtain a relation between the structural response and the expected damage, and 5) associate the structural response to a seismic intensity parameter. IDAs were performed with OpenSees [17] considering a concentrated plasticity model and a Modified IMK Peak-oriented model [15]. The seismic hazard was represented with 50 seismic records with both horizontal components (N-S and E-W), obtained from the accelerometric stations presented in Table 1. The authors of this study propose to employ a damage index defined by Teran-Gilmore and Jirsa [18]. As many other damage indexes, Teran-Gilmore and Jirsa damage index, IDTJ, does not consider soil-structure interaction; however, it does employ hysteretic energy dissipated by a SDOF system and its associated displacement in terms of ductility (Eq. 2).

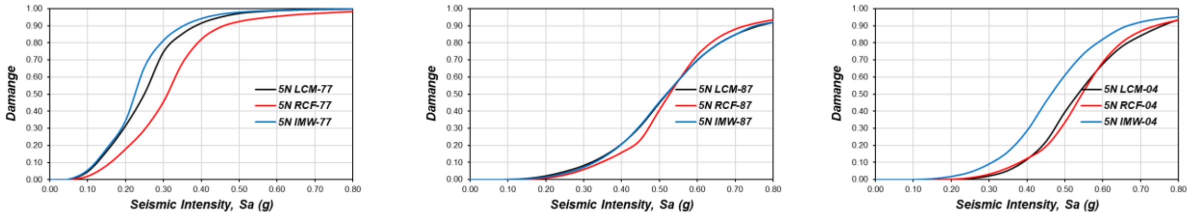
$$ID_{TJ} = \frac{NE_{H\mu}(2-c)}{r(2\mu_u - 1)} \tag{2}$$

In Eq. (2), μ_u is the capacity in terms of ductility due to a monotonic force action, c and r are structural parameters that measure the structural stability of the hysteretic cycle, and $NE_{H\mu}$ is the normalized hysteretic energy demand due to a given seismic excitation and it is defined as:

$$NE_{H\mu} = \frac{E_{H\mu}}{F_y \delta_y} \tag{3}$$

where $E_{H\mu}$ is the hysteretic energy demand during a hysteretic cycle, F_y is the yielding lateral strength and δ_y is the yielding displacement.

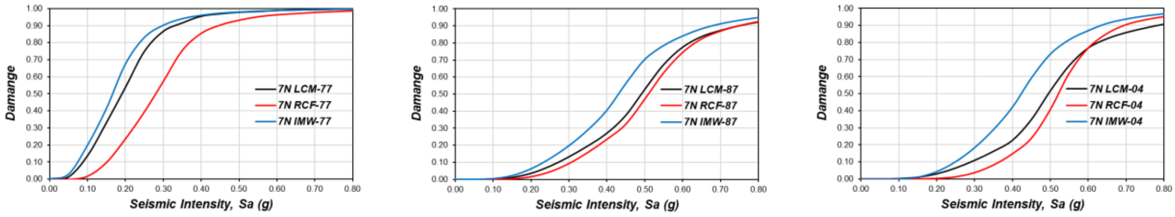
Vulnerability functions for the five storey structure are depicted in Fig. 5. It can be observed that for a given pseudoacceleration value, Sa, the expected damage is higher in the older structures, which is consistent with all the discussion exposed in previous sections. In addition, the slope of the vulnerability functions of the recent structures is softer (e.g., 2004 code) which indicates a larger ductility.



a) b) c)

Fig. 5 – Vulnerability curves for 3-5 storey buildings built with the different building codes considered

The same situation is presented in seven-storey structures designed with the RCDF-1977 [6] where damage occurs due to low intensities (0.12g) while, for those structures designed with more recent building codes (1987 and 2004) [7, 8] the same level of damage is presented under 0.2g intensity. It is important to point out that the obtained vulnerability curves for the 1977 structures have a steeper slope than those built in more recent years; this is due to the fact that the 1977 structures have a brittle failure or less ductile behavior while the others exhibit a more ductile behavior.



a) b) c)

Fig. 6 – Vulnerability curves for 6-10 storey buildings built with the different building codes considered

3.5 Seismic risk

The *EAL* computed with Eq. (1) for earthquake is shown in Fig. 7. The spatial distribution shows the risk for four value intervals of the *EAL*: lesser than 1%, 1 to 2%, 2 to 4, 4 to 7, and larger than 7%. These *EAL*s are the amounts that should be saved annually by the decision maker to cover all future disasters due to seismic hazard effects on dwellings. The seismic risk map (Fig. 7) is a useful tool for developing appropriate strategies on environment protection, hazard assessment and city planning. It can be observed in Fig. 7, regarding to the 3-6 storey structures, that the most affected would be those built with unlinked walls (RCF) and those which would have the better behavior are those designed with unlinked walls but built linked (LCM). This situation seems to be contradictory to the common sense, as it has been observed that such structures have been more damaged mainly because of the short column effect produced for the mid-rise walls. This situation is explained because RCF and LCM have the same design vibration period, therefore, both type of buildings shares the same design lateral strength (fig. 2); however, when they are built, the LCM building has a shorter vibration period, taking it away from the spectral range where the larger demands occur.

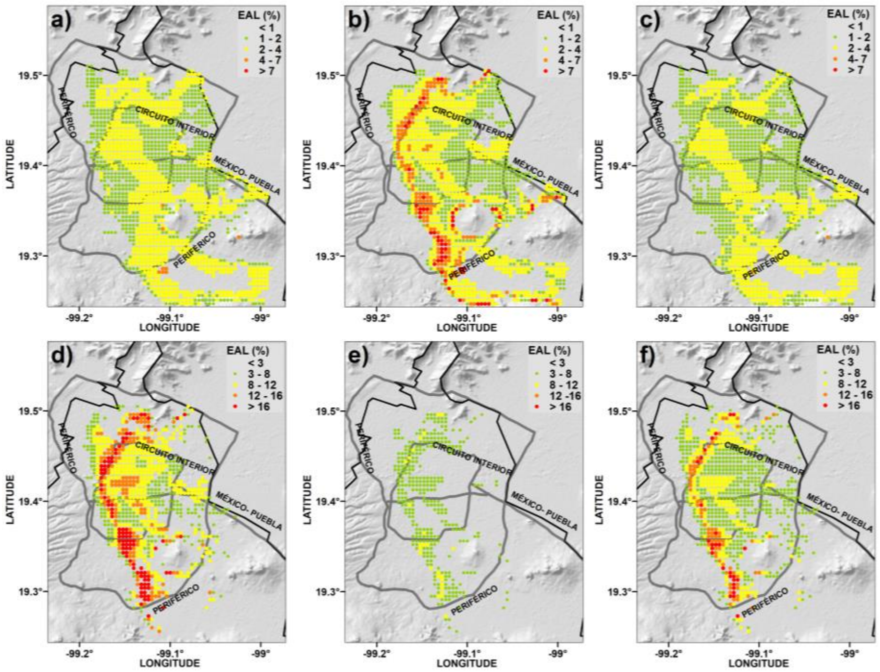


Fig. 7 – Distribution of the *EAL* for the studied dwellings designed with the RCDF-1977 due to earthquake in Mexico City. a) 5 IMW, b) 5 RCF, c) 5 LCM, d) 7 IMW, e) 7 RCF, and f) 7 LCM

For those structures designed with the 2004 building code, the EAL is considerable shorter than that obtained for the 1977 structures, situation that is in agreement with their corresponding vulnerability functions. However, it is observed that for 6-10 storey structures (Figs. 8d, 8e and 8f), even though is obtained the same *EAL* value, the less favorable condition is for the RCF structures (Fig. 8e) as an important number of them reach the maximum value; this condition is mainly due to the fact that such type of structure is more flexible than the other two types, putting it in the vibration period range where the seismic demand is larger.

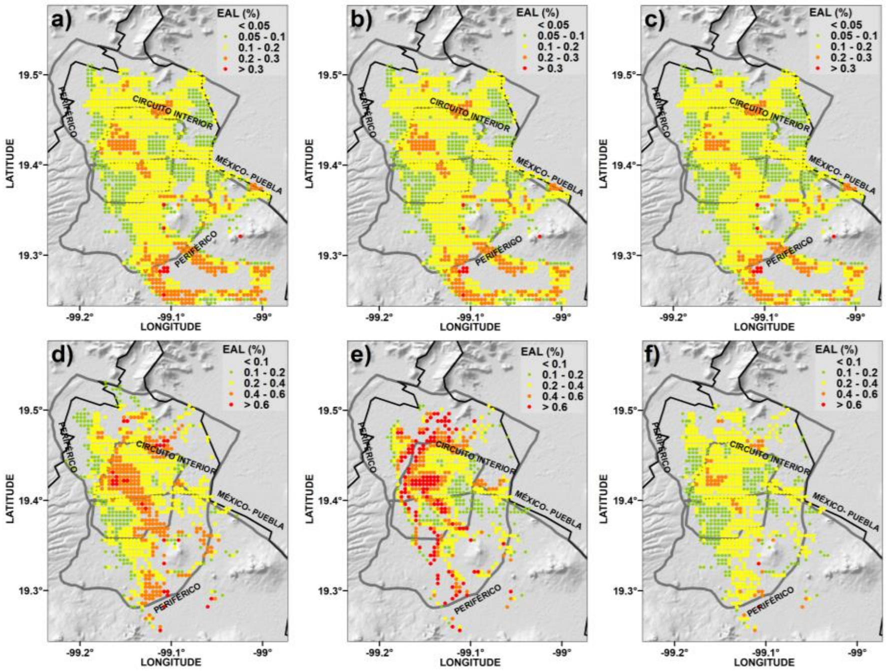


Fig. 8 – Distribution of the EAL for the studied dwellings designed with the RCDF-2004 due to earthquake in Mexico City. a) 5 IMW, b) 5 RCF, c) 5 LCM, d) 7 IMW, e) 7 RCF, and f) 7 LCM

The above information is important to define preventive actions oriented to mitigate the harmful effects generated by the earthquake actions. However, some other times it is important to be prepared for emergency situations creating civil protection plans or orientate the available sources to those areas that are potentially more susceptible to suffer large damage; for this reason, the *EAL* may not be appropriate and the expected loss associated to a specific seismic events is required. In this sense, Fig. 9 shows the expected loss associated to the occurrence of a M8.2 earthquake generated in the Guerrero gap for the hypothetical structures built with the 2004 building code. This results are in agreement with the EAL presented in Fig. 7; however, as may be observed, the obtained losses are much larger than the presented above as they are not weighted by the occurrence frequency as the *EAL*. As a first sight all the results presented in this paper are considerable large; however, it is important to point out that the associated over-strength is not considered in these computations.

4. Conclusions

The structural behavior and the seismic vulnerability are presented for buildings similar to those damaged by the earthquakes that occurred on September 19, 1985 and 2017, designed with the RCDF in its versions of 1977, 1987 and 2004. From these analyzes the following is taken: (1) Consider the participation of medium-height walls in the design of buildings of five and seven levels with a weak floor, significantly reduces the ability to resist shear forces resulting from an earthquake compared to those where it was not considered, since the wall provides resistance and rigidity to the structure. Its influence is reflected in the obtaining of mechanical elements, where in terms of design shear in both structures is almost the same. However, in terms of flexion there are greater demands on structures with RCF, this modifies the assembly of the sections as well as their ability to resist the demand. (2) The short column effect is expected from the design in this way the buildings are prepared in such a way that they are able to resist it and thereby reduce the damages that could occur. (3) For the simulation of an M8.2 event it was observed that the structural response is better in structures designed with recent construction regulations, which define the design demands considering how regular or irregular the structures are, as well as the properties of the materials, factors and design coefficients. Given the possible occurrence of an event of this magnitude, this study suggests that the structures considered here should not be affected, however, it is also a wake-up call to reinforce those buildings designed with older regulations to comply with the limits allowable established in the most recent regulations. (4) The effect of weak floor, is appreciated in an important way before intensities of 0.05g to 0.30g, this causes the demands to be greater than expected, in this way the buildings try to dissipate as much energy as possible through the deformations. (5) Based on the vulnerability curves it can be inferred that the structures most susceptible to damage first are those of seven levels, however, of the three case studies the most unfavorable corresponds to the condition of LCM in the design for construction regulations more recent, since being more rigid structures the demands to which it is subjected are greater than those of design.

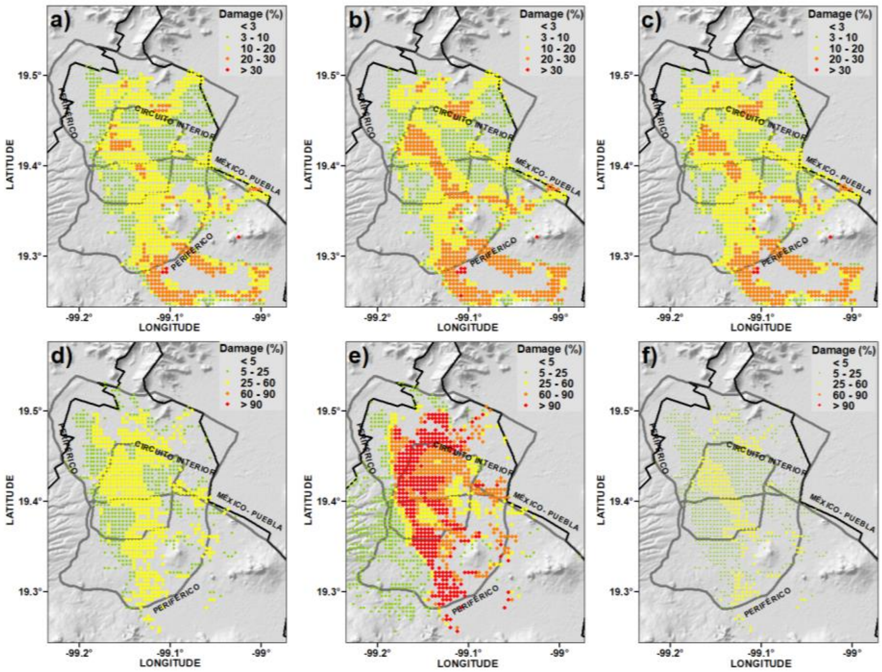


Fig. 9 – Expected loss for the studied dwellings designed with the RCDF-2004 due a hypothetical M8.2 earthquake in Mexico City. a) 5 IMW, b) 5 RCF, c) 5 LCM, d) 7 IMW, e) 7 RCF, and f) 7 LCM

In structures designed with the RCDF-77 or in previous versions they do not consider the irregularity of the structures, so the supposed design demands are lower in comparison to the real demands, which is why when considering structures designed with IMW these turn out to be more vulnerable.

5. Acknowledgements

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