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STRUCTURAL BEHAVIOR OF REINCORCED CONCRETE BUILDINGS WITH ASYMMETRIC MASONRY SHEAR WALLS LOCATED IN SOUTHERN MEXICAN CITIES

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SUMMARY

Use of masonry structural walls is a popular and an extended construction technique for improving earthquake building proficiency. In this paper, masonry wall-framed buildings located in Chilpancingo City, Mexico are numerically modeled. In the proposed models, asymmetric distribution of masonry walls, as well as three-dimensional and torsion characteristics are considered. Maximum building displacement under seismic loading are also presented. Actual reinforced concrete element's strength and the resulting seismic loading -as specified by the local construction code- are compared.

INTRODUCTION

This study is part of a comprehensive research project titled "Seismic strength evaluation of RC buildings in Chilpancingo City, Mexico", conducted by the Universidad Autónoma de Guerrero and sponsored by the Regional Research System "Benito Juárez" (National Mexican Council of Science and Technology). The principal goal of this research project is to have a better understanding of the seismic performance of typical southern Mexican RC buildings in a massive way, taking into account representative structures, that intends to contribute to earthquake disaster prevention.

Chilpancingo City, located at less of 100 km from the subduction zone between both, the Cocos and the North-American plate, possess one of the highest seismic risks in the world due to its proximity to the Guerrero seismic gap. Furthermore, stratum layering and special geological characteristics of the Chilpancingo Valley could generate seismic ground motion amplification, as it has been observed on accelerations recorded during the last 14 years.

Through Chilpancingo history, the high seismic activity of this zone has produced extensive damage on historical, public and private buildings. Thus, a seismic performance study for all the constructions

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(including masonry single houses) must be performed in order to mitigate future building damage, due to the expected big earthquake at the Guerrero's gap, Guinto [1]. This project considers this situation and it aims to evaluate the seismic performance of different kind of structures subjected to earthquake loading. Five types of structures were considered: a) Masonry houses, up to two-stories, b) Private buildings and Masonry multi-familiar houses, up to six levels, c) Hotels, d) Public or government buildings and e) Scholar buildings.

STRUCTURAL ANALYSIS OF REINFORCED CONCRETE BUILDINGS WITH ASYMMETRIC MASONRY SHEAR WALLS

In Chilpancingo City, an important part of framed reinforced concrete buildings with masonry shear walls are private buildings and masonry multi-familiar houses, up to six levels. Thus, in this work, private buildings including masonry walls or reinforced concrete shear walls were considered. In order to detect damage or structural deficiencies, buildings holding similar structural and architectural representative characteristics, were chosen.

Three-dimensional detailed models and spectral dynamic analysis were performed using program SAP2000. In order to perform an accurate building evaluation, drifts, dynamic structural characteristics, element shear stresses and forces as well as other important results were carefully considered. Then, reinforced concrete and masonry elements of the structure were checked.

The kind of structures analyzed in this work, are classified by the local construction code as structures type B that are commonly used for houses, office buildings, hotels and commercial or industrial buildings. Required information to identify and to classify the structural characteristics of the chosen buildings was obtained directly in situ.

The record for each building included: localization, dimensional general building data and dimensional characteristics for each structural element. Buildings with masonry walls confined by reinforced concrete frames and possessing a rigid floor system (RC slabs) are located principally in the central part of the city. In this study, 104 buildings were considered. A photographic record was also elaborated. An important part of the urbanized city was covered in this study. Figure 1 shows building's localization.

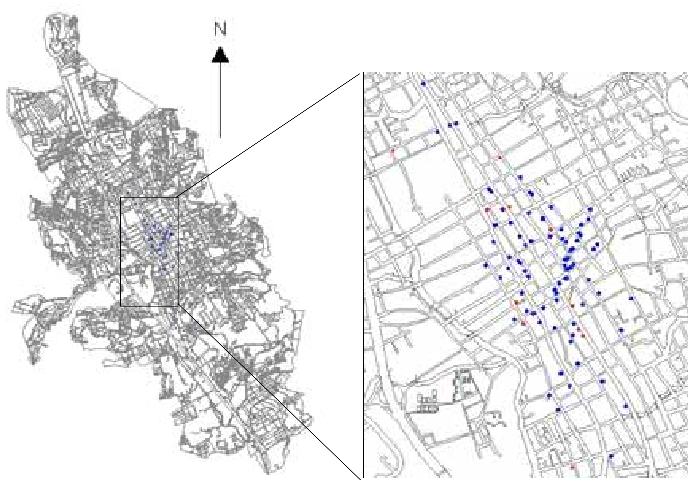


Figure 1: Localization of the studied structures in Chilpancingo City, Mexico.

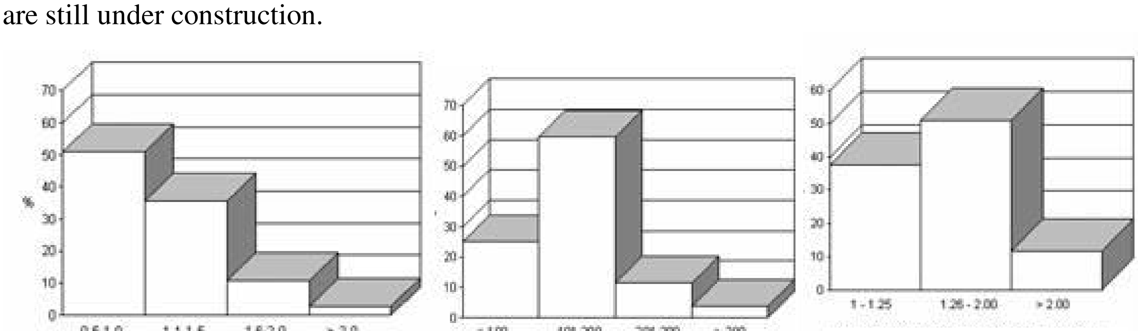
As soon as both, the visual inspection and record activities were concluded, data was extensively analyzed in order to establish similarities between these constructions. Geometrical and structural key parameters for buildings were determined and used to classify them. In order to perform the structural building checking, representative buildings with similar characteristics were chosen.

In order to perform the structural classification of buildings, basic concepts were taken in consideration, such as the size and shape of buildings. In seismic design, these characteristics are even more important than absolute size; for tall buildings, slenderness is more important than the building's height only.

Furthermore, in order to study torsional effects, floor's symmetry was considered. Buildings were classified accordingly to height's regularity: a very important part of the studied buildings possess height's regularity. Anyway, it would be worth to consider that an important part of these buildings are still under construction.

The second part of the study was to determine, in an approximated way, some of the configuration characteristics related with the form in which a building responds to seismic motions and then, to compare them with the regularity relations proposed by the local construction code.

Figure 2 (a) shows the slenderness for the 104 buildings considered in this study. Approximately 86% of these structures accomplish regularity conditions, 11% were classified as irregular structures with slenderness ratios between 1.5 and 2. Only 3% exceeds the limit proposed by the local code. Most of the analyzed structures are four-storied buildings. Anyway, an important percentage of the studied structures are still under construction.



a) Height/length ratios   b) Construction area (m^2)   c) Regularity

Figure 2: Building distribution according to: a) Slenderness, b) Construction area and c) Regularity

Figure 2 shows also, an in-plant length/wide ratio for the 104 structures. 89% hold a regular in-plant shape and 11% present an in-plant shape with a length/wide ratio greater than 2.00. 25% of the structures have a first floor construction area not higher than 100 m^2, and approximately 60% have a construction area between 100 y 200 m^2. 11.5% have a construction area between 201 and 300 m^2 and only 3.5% have a first floor construction area greater than 300 m^2.

An adequate structural system based on RC framed buildings is able to bear the inertial loads during a seismic motion and transmit them correctly to the foundation and then to the ground. Three-dimensional framed RC buildings are formed by a system of columns and beams, allowing the right transmission of forces in the elements and contributing to the horizontal building stiffness.

In addition, some of the analyzed structures possess shear walls. From the studied buildings, 46 were RC framed structures (EM) and 58 were RC framed structures combined with masonry shear walls (EMM). The building aspect ratio must be determined taking into account the building's shape. Anyway, an important part of the buildings possess 4 or 5 stories. Thus, its aspect ratio classification was simplified considering principally the long length/short length (a/b) ratio and the in-plant construction area. In this way, representative buildings were defined, considering both, plant regularity and construction area.

Among the studied structures, those possessing similar structural and architectural representative characteristics were chosen. In order to perform modeling and analyses, 13 structures were chosen as representative of southern Mexican buildings. These buildings include different geometrical configurations, and they aim to study different building representative shapes.

An extended very common practice in RC building's construction is to place masonry walls to fill reinforced concrete frames. In many cases, these walls are not considered in the structural analysis and walls are assumed to be isolated from the framed structure. Anyway, adequate separation between the wall and the reinforced concrete is always omitted. For this reason, masonry walls have a structural participation under seismic loading. Thus, a design of the structural elements where this interaction is not considered could be inadequate and dangerous.

Masonry walls present a linear behavior until the first cracking, after this, masonry walls undergo drastic inelastic behavior, characterized by a both, a strength and stiffness reduction. Wall structural strength disappears quickly even under low seismic cyclic loading. Modeling of this structural behavior can't be represented by linear elastic models. For modeling masonry shear walls, equivalent diagonal method was used as a reference method, Meli and Bazán, [2] and a finite element model was proposed in order to reproduce the results obtained with the reference method.

This model considers the separation between masonry walls and the framed structure. A special very short connection element joint was placed between the wall corners - subjected to compressional or tensional forces - and the frame's beam-column joint. This element was moment-released at one of its ends: the one connected to the frame.

Furthermore, stiffness of the connecting element was supposed to be extremely high. In order to get a better stress distribution schemes in masonry walls, a calibration of the model's finite element size was conducted and time of analysis was also optimized.

Results on meshes formed by quadrilateral four nodes finite elements of approximately 50x50 cm were found to be more accurate. A comparison between results obtained with the finite element model and those obtained with the equivalent diagonal method under static horizontal loading shown errors up 2%. The described calibrated model was employed in the RC buildings to be analyzed. SAP2000 program was applied to generate building numerical models.

Framed models were first considered in order to compare the forces employed by original designers and then, structural masonry walls were also modeled to know actual building behaviors. Figure 3 shows some of the modeled buildings in this work. In order to avoid using finite elements in slabs, a special diaphragm constriction was imposed to all the joints at the slabs of each building's floor.

Drawings, schemes and direct visual inspection of buildings allowed to generating representative numerical models. Static and dynamic analysis methods were used to establish maximum drifts, shapes and natural periods of vibration, dynamic eccentricities and shear basal forces for each building. Geometrical and material model characteristics are fully described in references, Magallanes and Jaimes, [3].

Structural analysis of buildings

In order to include the torsional effect into the dynamic analysis, a determination of the mass position center for each floor was performed. Slab's openings, such as those dedicated to lighting, as well as stairs, balcony or extended non-supported zones of slabs, were all considered. Rotational inertia due to the masonry walls was also considered and calculated for each wall and then placed on the slab, just directly on the vertical projection of the middle part of the masonry wall.

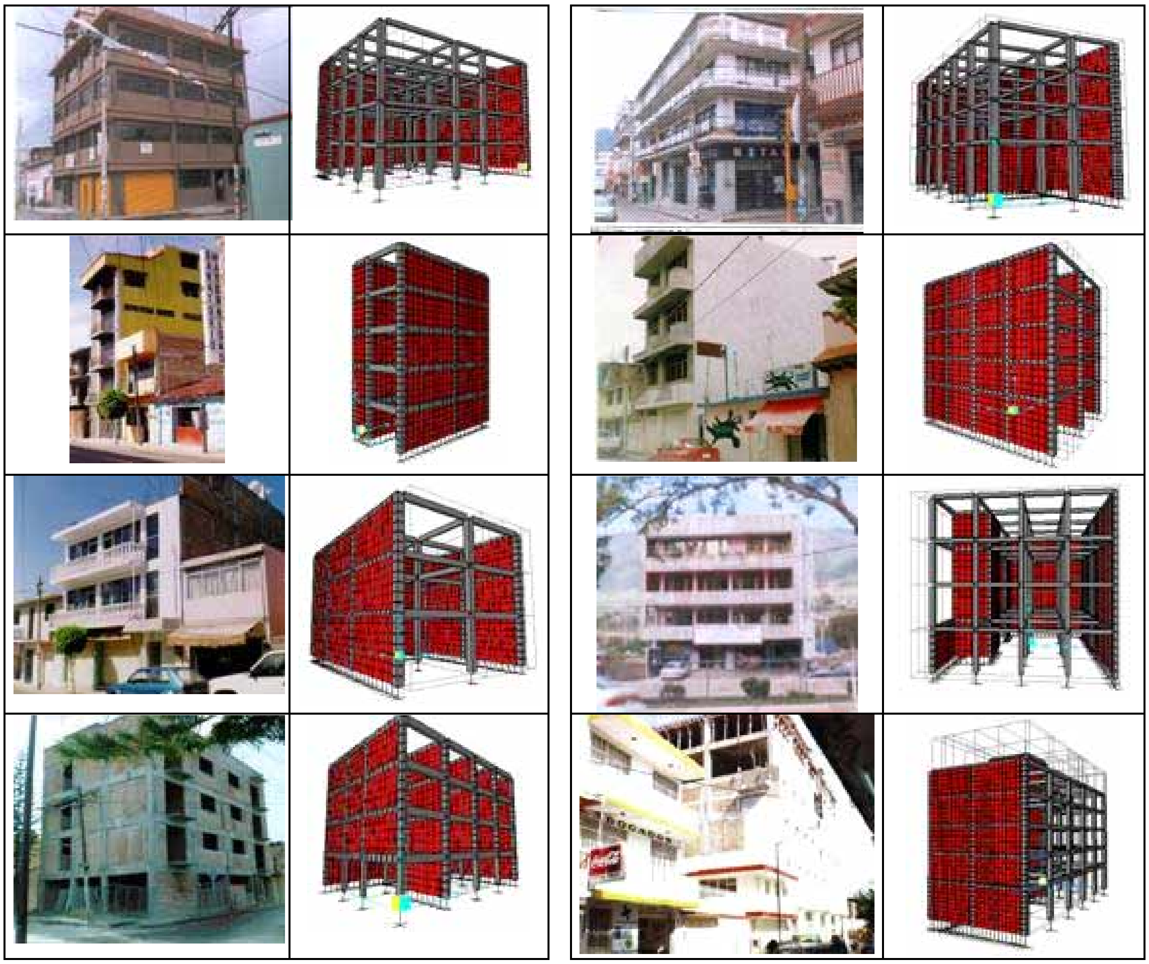


Figure 3: Type of analyzed buildings

Concentrated masses were also considered in a similar way. These values were automatically assigned to the floor master joint by the SAP200 program. As it was stated before, RC slabs were first modeled with quadrilateral four node finite elements with distributed mass including both, self-weight and live loads. In order to estimate the floor's mass position center, a diaphragm constriction was applied to each building floor. Then, a master floor joint was generated manually in these points.

Both, the floor translational mass and the rotational mass inertia of the created diaphragm were concentrated at the master floor joint. In this way, models were analyzed and an extensive group of results were obtained. Load combinations prescribed in the local code were also considered.

Gravitational loads were applied on the beams as uniform gravity loading. Self-weight of columns was considered applying an equivalent vertical point load at the superior column's end. In dynamic analyses, a local design spectrum depending on building's localization was also considered.

According to the seismic design criteria, lateral forces and seismic coefficients, must be divided by the seismic behavior factor, Q. Analyzed buildings possessing masonry walls were designed to have only an architectural function and in a lot of cases, this kind of wall was commonly used at the building's perimeter. Anyway, separations between frames and walls are often omitted. Then, masonry walls are subjected to high horizontal loading that transforms these non structural walls into improvised and non-adequate structural elements. This interaction can initiate extensive damage on other structural elements. In most of the studied buildings, original designers hadn't considered masonry filling structural walls in their structural analyses.

RESULTS

Principal periods of the 13 analyzed buildings for each of the four mass position centers prescribed by the Mexican code, are presented in table 1. As the local design spectrum hasn't an initial ascendant branch and building periods of vibration are lower than 2 seconds, the seismic spectral coefficient for designing is the same for all the analyzed buildings.

Principal periods lay approximately from 0.17n to 0.25n for three-storied buildings; from 0.13n to 0.21n for four-storied buildings, and from 0.14n to 0.16n for five-storied buildings; n is the number of building stories.

Table 1: Principal building periods at the four mass position centers

Bldg	Stories	Principal periods (design spectrum with C = 0.8)			
		Position p	Position s	Position t	Position c
7-EM	3	0.5332	0.5019	0.5043	0.5014
10-EM	6	0.8698	0.8699	0.9321	0.9363
16- EMM	4	0.5310	0.5349	0.5577	0.5373
18-EMM	4	0.7266	0.7140	0.6771	0.6766
23-EM	5	0.8036	0.7710	0.7945	0.8019
27-EM	5	0.7708	0.7870	0.7257	0.7250
31-EM	5	0.7065	0.7064	0.7141	0.7113
37-EM	5	0.7961	0.7684	0.7537	0.7510
39-EM	3	0.7180	0.7464	0.7009	0.6988
44-EM	4	0.8398	0.8188	0.8077	0.8561
46-EM	4	0.6278	0.6481	0.6258	0.6293
47-EM	4	0.7450	0.6884	0.7113	0.7186
49-EMM	3	0.7588	0.6930	0.6800	0.6906

Principal building periods lay from 0.5 to 0.9 sec. In the spectral building response, the total mass participation was always considered. This means that the modes of vibration were all considered, even those involving higher frequencies.

Table 2 shows a comparison between the shear basal forces obtained with both, the static and dynamic seismic methods. Seismic behavior factor Q, was always the same for each of the orthogonal directions of analysis for all the buildings. Table 2 shows that the dynamic shear basal force lays between 80% and 90% of the static shear basal force.

Table 2: Dynamic and static basal shear forces comparison

Building	Static shear force V_e (Ton)	Dynamic shear force V_{dx} (Ton)	Dynamic shear force V_{dy} (Ton)	V_{dx}/V_e %	V_{dy}/V_e %
7 – EM	137.49	108.82	119.07	79.15	86.60
10 – EM	793.25	666.00	650.32	83.96	81.98
16 - EMM	261.22	207.91	225.30	79.59	86.25
18 - EMM	409.44	341.90	334.33	83.50	81.66
23 – EM	260.77	208.58	224.21	79.98	85.98
27 – EM	428.57	349.48	362.74	81.55	84.64
31 – EM	219.46	193.11	203.95	87.99	92.93
37 – EM	307.30	269.37	244.87	87.66	79.69
39 – EM	188.83	167.19	154.75	88.54	81.95
44 – EM	324.48	249.57	266.62	76.91	82.17
46 – EM	256.79	236.36	200.32	92.05	78.01
47 – EM	196.96	156.50	172.98	79.46	87.83
49 - EMM	187.63	152.45	163.12	81.25	86.93

Dynamic amplification of building's eccentricities

Dynamic modal analysis allowed to estimating lateral forces (F) and torsional moments (M) at the mass position center (CM), at each building's story. When a mass position center doesn't coincide with the floor torsional center, the applied moment at the mass position center and the one produce by the horizontal force can be expressed with the same force, but acting at an amplified distance respect to both, the mass and the torsional center. Results are shown in figure 4. In this graphic, amplification factor decreases quickly as the calculated and accidental (0.1b) ratio eccentricities, increases.

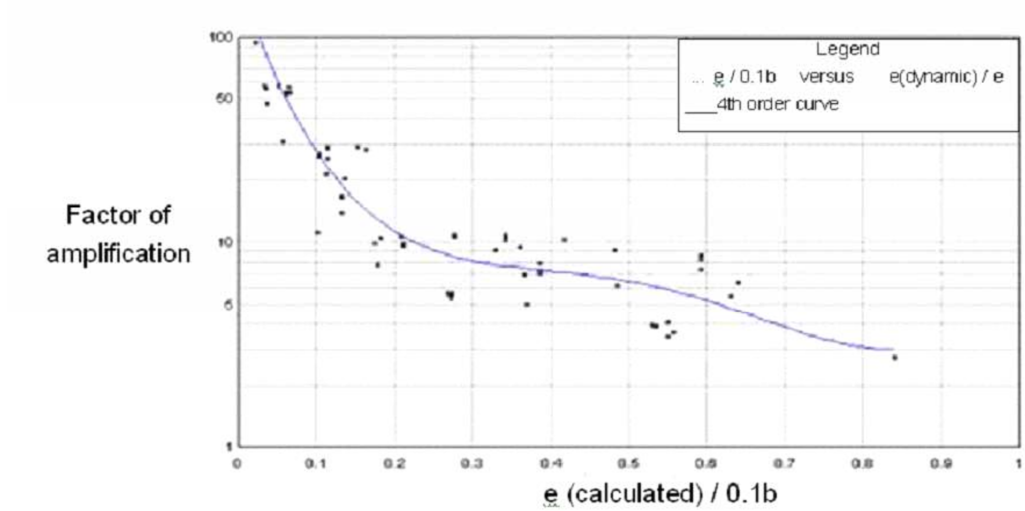


Figure 4: Direct eccentricity amplification factor

Where the correlation coefficients are: C(00) = 2.271309, C(01) = -11.390553, C(02) = 35.030353, C(03) = -47.336750, and C(04) = 22.331754. This graphic has a high utility as it allows knowing eccentricity amplification factors in three-dimensional dynamic analysis of structures with similar geometrical and material characteristics to the analyzed buildings in this paper. In this case, a design spectrum for zone II (intermediate soft clayed soil) and a seismic behavior factor of 2 were considered. Thus, if a dynamic analysis is performed without considering the torsional effect, it can be found with the factor given by figure 4, a dynamic eccentricity value even if a three-dimensional dynamic analysis is not performed. In this case, the calculated and accidental eccentricity ratio ec/(0.1b) must be known. The corresponding value of the factor is found intersecting the fitted curve. Ordinate value at this point is the factor that can be used to factorize the calculated eccentricity.

Maximum displacements

Table 3 shows a comparison between maximum roof displacements of the analyzed buildings and those obtained with the dynamic modal analysis. All the displacements are factorized by the corresponding ductility factor, Q. Considering the fact that the analyzed buildings have similar general characteristics and that the total absolute roof displacements and the principal period of vibration are the most relevant earthquake response parameters of a RC building, then most of the analyzed structures of Chilpancingo City have a low seismic performance than that prescribed by the local construction code.

Structural elements checking

Horizontal displacements depend strongly on columns horizontal stiffness. Furthermore, as the reinforced column elements are fundamental in the general structure stability, checking of structural elements was focused principally on this type of element. Design strength of reinforced concrete columns in all the buildings was fully checked. For these revisions, geometrical and material characteristics as well as the available reinforcing data on drawings were considered. Loading combinations including gravitational and seismic effects were also considered and prescriptions included in the local code were applied. The full seismic loading in a specified direction, was combined with 30% of the seismic orthogonal force acting in the perpendicular direction. In order to obtain the more unfavorable load combination and as design eccentricities can produce favorable effects on some of the structural elements, while they can produce unfavorable effects in some others, different acting directions of the shear load were considered in each analysis

Table 3: Maximum building drifts

Bldg	Ductility Factor Q	Displacement at p (cm)		Displacement at s (cm)		Displacement at t (cm)		Displacement at c (cm)		Maximum allowed Δ (cm)	$\Delta \leq \Delta_{max.}$
		X	Y	X	Y	X	Y	X	Y	$\Delta_{max.} = 0.012\ h$	
7–EM	2	8.32	2.96	7.74	2.18	3.12	6.62	2.98	7.00	10.8	✓
10-EM	1.6	17.31	10.67	17.33	11.86	6.11	29.36	6.35	29.50	24.74	×
16-EMM	2	7.20	2.86	7.12	2.54	3.22	8.78	1.82	8.18	12.00	✓
18-EMM	1.6	16.85	4.00	16.42	3.89	5.33	9.81	6.99	10.14	13.20	×
23-EM	2	18.22	7.76	17.38	6.04	6.34	18.08	6.54	17.30	18.24	✓
27-EM	1.6	19.86	4.80	20.61	4.61	3.94	11.87	8.45	12.69	18.00	×
31-EM	1.6	13.38	3.14	12.54	2.42	2.96	14.50	2.24	14.14	17.28	✓
37-EM	2	18.56	5.16	17.38	3.62	6.86	12.24	6.70	12.48	16.20	×
39-EM	2	15.64	3.52	15.88	5.36	6.62	12.88	4.32	12.32	11.40	×
44-EM	2	20.40	8.00	18.78	8.08	5.44	18.32	9.92	19.66	14.40	×
46-EM	2	10.34	1.00	11.40	3.14	2.98	7.72	3.70	7.64	12.48	✓
47-EM	2	15.80	6.52	13.12	2.54	5.58	13.16	5.68	14.76	12.60	×
49-EMM	1.6	10.08	1.65	8.50	0.82	2.90	4.67	3.07	4.67	9.72	×

RC column checking

For reinforced concrete columns checking, the most unfavorable load combination was considered. Column's biaxial flexocompressional strength was checked and ultimate shearing in the shear strength of the column's section according to local code prescriptions was considered. Furthermore, interaction column's curves generated by the SAP2000 program were also checked. In this program, ACI criteria is employed to develop the interaction columns curves. Anyway, there are not big differences between these curves and those obtained with both, the Mexican and local codes. Figure 5 shows biaxial interaction column curves obtained with the SAP2000 program. Yellow curves correspond to the applied eccentricities ratios and the red point shows the axial and flexural force levels produced on the column by an unfavorable load combination. From the first and second floor columns checking, it was observed that generally load combinations that such kind of structural elements will be bearing during a major earthquake, could be greater than their actual resistant capacity. Furthermore, most of the revised column sections are not suitable to bear the high expected shear forces.

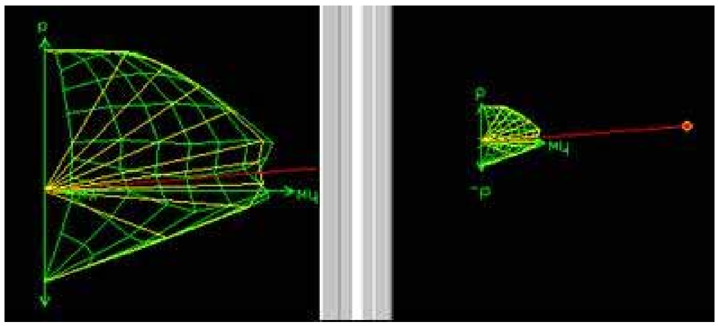


Figure 5: Biaxial interaction columns curves, first floor RC column in building 10-EM

Flexural and Shear strength checking of reinforced concrete beams

The specified reinforcing bars depicted in structural building drawings were carefully considered and the flexural beam strength was determined principally in the first two stories of buildings in both, the long and the short building directions. As the amount of reinforcing steel bars was high in the cross section of beams, they were considered as double-reinforced beams. Strength was calculated considering both, positive and negative flexural moments. Figure 6 shows the beam moment envelopes. Horizontal dark lines show the flexural strength of the critical cross RC section for negative and positive moments. This flexural moment envelope shows that in most of the analyzed buildings, flexural strength for RC beams is not satisfactory to bear the expected high seismic moments. Actual flexural strength in some of the spans has only 50% of effectiveness. If the high shear forces that could be developed during a big earthquake are considered, actual shear reinforcing on beams - as specified in drawings - is not satisfactory. Consequently, shear reinforcing have a non-adequate spacing of stirrups.

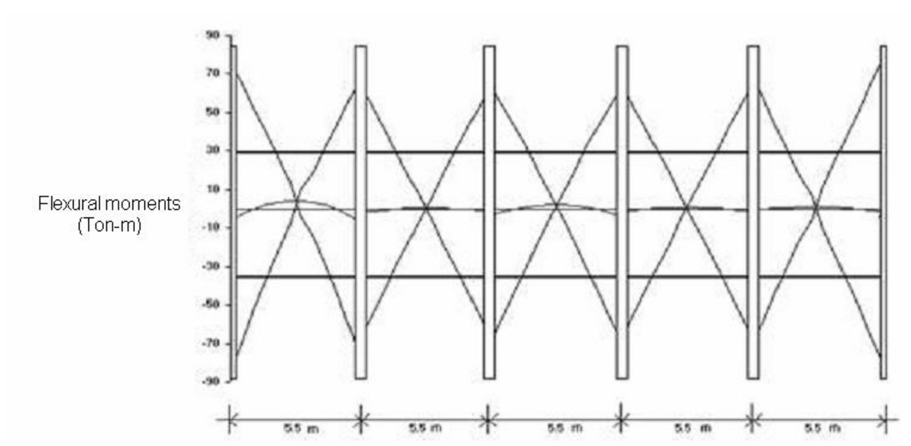


Figure 6: Flexural moment envelope for a critic RC beam

Shear stresses on masonry walls

The principal revised parameters were both, the shear stresses and the maximum corresponding drift at each story. The fact that shear stresses on walls are greater than the maximum shear strength and that roof horizontal displacement is greater than the maximum allowed one shows that the structural design in most of the buildings doesn't accomplish the seismic prescriptions criteria of the local construction code. Buildings were modeled including the structural participation of the masonry walls. The following figures (7 and 8) show a general three-dimensional view of some of the analyzed models. For example, masonry walls distribution for building 31-EM is asymmetric at the first floor since this building is located on a corner. Therefore, masonry walls are located principally in two adjacent sides of this building. Upper stories have a more regular and symmetric distribution of the masonry walls. Thus, there's an important seismic risk in this construction since it possess a "soft" first story. Results in table 4 show that compared to those obtained in the same structure without masonry walls, this building hasn't an important reduction of its period of vibration.

Table 4: Periods of vibration for models with and without masonry walls

Building	Framed model First period (sec)	Walled Model First period (sec)
10-EM	0.8698	0.7739
31-EM	0.6441	0.5516
46-EM	0.6278	0.4961

The most important parameter in masonry walls is the shear strength. Figure 7 shows the masonry walls shear stresses distribution in the corresponding analysis direction x. A higher shear stress concentration on the shear wall's principal diagonal is observed. Stress level data is shown at the bottom of figure 7. It's important to note that some of the higher developed shear stresses are greater than the maximum shear masonry strength (v* = 0.34 MPa) prescribed by the local construction code.

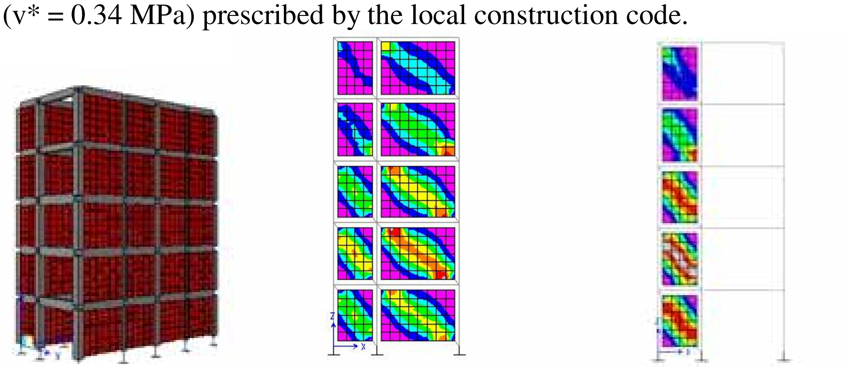


Figure 7: Shear stresses on perimeter masonry walls of building 31-EM (x direction)

As it was shown in the short-length analysis direction, long-length analysis direction shows walls shear stresses that are greater than the maximum masonry shear strength. Figure 8 shows shear stresses distribution developed on masonry walls of building 46-EM. In this case, a higher stress concentration is located on the principal wall's diagonal. Shear stresses are higher than the masonry shear strength proposed by the local construction code. Shear stress distribution on external walls shows greater shear stresses than the maximum allowed ones.

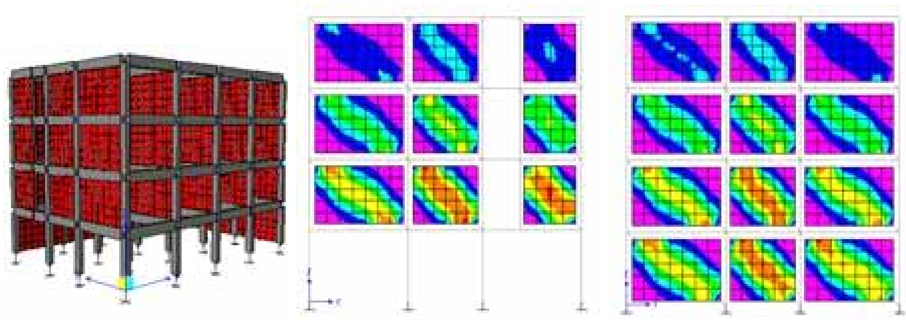


Figure 8: Shear stresses on masonry walls of building 46-EM

Drifting at some of the building stories are presented in the following table. In this table is discussed if story absolute displacements are lower than the allowed prescribed ones. Structural model with masonry shear walls don't accomplish the maximum allowed displacements at the first and second stories. A similar situation can be found in the case of the framed building. Anyway, in masonry walled structures, drifts exceeds 62% of the allowed drifts, while the framed structure (the same structure but without masonry walls), drift exceeds the allowed drift in 5%.

Table 5: Displacement comparison for models with and without masonry walls

Building	Displacements (cm) on Walled model		Maximum displacement allowed (cm) $\Delta_{max.} = 0.006$ h *	$\Delta \leq \Delta_{max.}$	Displacements (cm) on a framed structure		Maximum displacement allowed (cm) $\Delta_{max.} = 0.012$ h *	$\Delta \leq \Delta_{max.}$
	X	Y			X	Y		
10-EM	2.40	0.91	1.89	×	3.42	2.08	3.78	×
31-EM	2.02	0.40	1.73	×	2.72	0.61	3.46	✓
46-EM	2.04	0.66	1.56	×	3.28	0.33	3.12	×

* h is the story's height.

CONCLUSIONS

Future earthquake damage can be greatly reduced by identifying and improving or removing most vulnerable and dangerous structures. Results of the structural analysis of representative buildings located in Chilpancingo City, Mexico, show that their structural strength doesn't accomplish actual prescriptions of the local construction code. Shear stresses developed on masonry walls are greater than the masonry shear strength and roof displacements are higher than the allowed ones prescribed by the local code in most of the analyzed buildings. Therefore, rehabilitation and reinforcing tasks have an extremely importance in order to obtain an adequate seismic performance in these buildings. Structural behavior of buildings having an asymmetric distribution of masonry walls present unfavorable effects. Furthermore, using structural shear walls requires applying reduced drifts to almost a half of those obtained in a framed structure. This reduction of the maximum displacements is not compensated by the strength that masonry walls confer to the general structural stiffness.

In the dynamic spectral analysis performed for each analyzed building, most of possible modal shapes and the modal mass participation ratio were considered. In order to obtain results with 90% of effectiveness, it was found out that it's necessary to consider at least the first five modes of vibration.

The high seismic coefficient for Chilpancingo City assigned to the seismic zone II (intermediated soft clayed soil) has direct implications on the structural design, in such a way that if a structural design optimization is not performed, an anti-economic structural design could be obtained. Optimized design must employ a high seismic behavior factor. It means that buildings should be designed with sharp design methods such as ductile designing. In order to obtain a more solid technical basis to propose building's reinforcing, a more detailed analysis of this kind of buildings, considering both, nonlinear geometrical and material characteristics must be considered. Anyway, the proposed models give a more reliable method to determine building's seismic behavior.

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