Simplified Method for the Seismic Design of Low-Rise, Shear Wall Base-Isolated Buildings

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Abstract: A simplified method for the seismic design of low-rise, base-isolated shear wall structures is proposed in MOC-2012. This simplified method is basically a hybrid method, where the design of the isolation system is a simpler version for the static method available in US guidelines for the design of base-isolated structures and the design of the superstructure essentially is an improved version of the simplified method for the seismic design of conventional low-rise shear wall structures of Mexican seismic codes. The application of this method and its effectiveness to obtain safe designs is illustrated with a practical example. It is anticipated this simple methodology would help promote the use of base isolation in low-rise shear wall structures and reduce their vulnerability when subjected to strong earthquakes.

INTRODUCTION

The Manual of Civil Structures (MOC), one of the model design codes in Mexico, was updated. This manual is frequently used in the entire nation in lieu of a specific code for a state or a city. The new version for this manual and all the chapters was published in 2008 [1, 2]. The bases and design philosophy of previous and current seismic design provisions of MOC code in English language can be found [2, 3].

There is one new chapter in MOC-2012 devoted to the seismic design of base-isolated structures [4], the philosophy of which is presented in detail in [5]. Among the innovations of these base-isolation guidelines is that a simplified method of seismic analysis is allowed for the design of low-rise, shear-wall base-isolated structures.

This method is hybrid in nature, as briefly mentioned before. The design of the isolation system is a simpler but more restrictive version for the static method for base-isolated structures, which is very similar to US guidelines [6-9]. The design of the superstructure essentially is the improved simplified method for the design of conventional structures but the effective shear area factors (F_{AE}) that are defined for the walls are those for elastic response [1-5, 10]. The effects of the vertical component of the ground motions are neglected in the design process when using the simplified method.

The purpose of allowing such method for design purposes is to favor the application of base-isolation in regular and squatty shear wall structures by reducing its complexity. It makes no sense to complicate the design of base-isolated structures when the superstructure is simple, regular, squatty and very rigid, as their seismic response is relatively easy to

assess. This is the concept behind the simplified method of analysis for base-isolated structures in MOC-2012.

The requirements to apply this simplified method and the method itself are described in following sections. A design example is also provided to illustrate the application of the method and its effectiveness to lead to safe structural designs.

SIMPLIFIED METHOD

Requirements for Application

The base-isolation system and the structure above the isolation system can be designed using the simplified method of seismic analysis (SMSA) only if the building satisfies all the following requirements:

A. The base-isolated structure complies with the following twelve conditions of structural regularity:

1. The distribution in plan of story mass, shear walls and other lateral resisting elements must be approximately symmetrical with respect to two principal axes of the building.

2. The ratio of the height of the building to the smallest plan dimension shall not exceed 1.5 ($H/L_2 \leq 1.5$).

3. The ratio of the largest to the smallest plan dimensions shall not exceed 2.0 ($L_1/L_2 \leq 2.0$).

4. Plan configurations of structures shall not contain re-entrant corners or offsets exceeding 20 percent of the plan dimension of the structure in the given direction.

5. Each story has a rigid floor system with sufficient strength capacity.

6. There are no diaphragm discontinuities or variations in stiffness, including those having cutout or open ar-

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eas greater than 20 percent of the gross enclosed area of the diaphragm or 20 percent greater than the plan dimension of the structure in the parallel direction. In addition, the open areas shall not vary in position between adjacent stories or shall not introduce important asymmetries in plan.

7. The weight of any story, including the seismic accidental live loading, shall not be greater than 110 percent or less than 70 percent the weight of the adjacent story below the one in consideration ($0.7 \leq W_i/W_{i-1} \leq 1.1$). A roof which is lighter than the floor below need not to be considered.

8. The plan area of any story shall not be greater than 110 percent or less than 70 percent the area of the adjacent story below the one in consideration ($0.7 \leq A_i/A_{i-1} \leq 1.1$). The roof is exempt of this consideration. In addition, the plan area of any story shall not exceed in more than 50% the minimum area of the lower stories ($A_i/A_{min} \leq 1.5$).

9. All columns are laterally restrained in their main orthogonal directions at all stories by rigid diaphragms, floor slabs or beams.

10. The lateral shear stiffness or strength of any story shall not exceed more than 50 percent the shear stiffness or strength of the adjacent story below the one in consideration ($K_i/K_{i-1} \leq 1.5$). The top story is exempt from this requirement.

11. The torsional plan eccentricities (e_{si}), computed for any story from static seismic analysis, shall not exceed 5 percent of the plan dimension in the given direction of analysis ($e_{si} \leq 0.05B_i$).

12. The torsional plan eccentricity for the isolation system (e_{sa}), shall not exceed 2 percent of the plan dimension in the given direction of analysis ($e_{sa} \leq 0.02B_a$).

B. The structure above the isolation system is composed of shear walls and satisfies also the following additional requirements of the SMSA of conventional shear wall buildings:

1. The walls must carry more than 75% of the gravitational loads.

2. The structure shall not be taller than four stories in height or 13 meters, whichever is smaller.

C. The base-isolated structure is located at least 50 km away from any known active fault.

D. The base-isolated structure is located on firm soils or rock [the site factor ranges from 1.0 to 1.25 ($1.0 \leq F_s \leq 1.25$) or the shear wave velocity is $v_s \geq 250\,m/s$].

E. The effective period of the base-isolated structure at the design displacement, T_{as}, is greater or equal to 1.5 seconds but less or equal to 3.0 seconds, this is, $1.5s \leq T_{as} \leq 3s$.

F. The effective period of the base-isolated structure at the design displacement, T_{as}, is greater than five times the elastic, fixed-base period of the structure above the isolation system, T_E, this is, $T_{as} \geq 5T_E$.

G. The isolation system meets the following criteria:

1. The effective stiffness of the isolation system at the design displacement is greater than one-third of the effective stiffness at 20 percent of the design displacement (Fig. **1**).

2. The isolation system is capable of producing a restoring force as specified in required prototype testing section of the guidelines.

3. The isolation system has force-deflection properties that are independent of the rate of loading.

4. The isolation system has force-deflection properties that are independent of vertical load and bilateral load.

It is worth noting that requirements A2 and A3, which are more strict than those requested for most conventional structural systems, are specified by the SMSA for low-rise structures [i.e., 10].

Requirement A11 for base-isolated structures ($e_{si} \leq 0.05B_i$) is more strict compared to conventional structures and the SMSA ($e_{si} \leq 0.1B_i$), as for elastic response, a smaller static eccentricity should be allowed in the simplified method, according to the results of a parametric study [11]. Requirement B2 is a little more restrictive as well, as for conventional structures, the SMSA is allowed for structures up to five stories in height [10].

Finally, requirements A12, C and F are more strict with respect to what it is allowed in the static method for base-isolated structures [i.e., 4, 5]. Requirement A12 is set to limit one source that amplifies considerably the torsional response in base-isolated structures: eccentricities in the isolation system [i.e., 12-15]. Requirement C is specified in order that the effects of the vertical component of the ground motions could be neglected in the design process when using the simplified method. However, it is worth noting that the effects of the vertical components cannot be neglected for near-fault ground motions since the isolators can undergo tensile loads, especially when the vertical component of the ground motion has a relatively high peak value [16]. Requirement F is set to warrant a reasonable uncoupled dynamic response, controlled essentially by the isolation system.
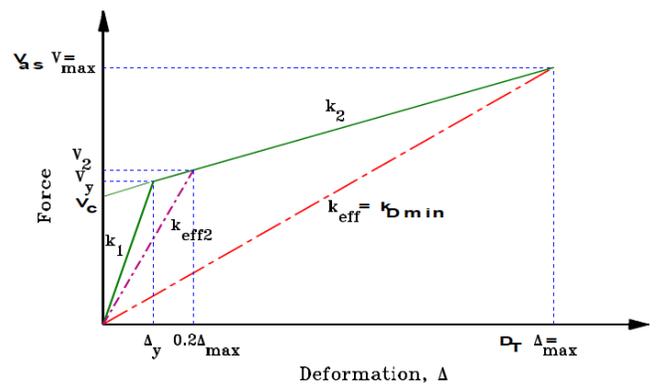

Fig. (1). Design envelope curve for bilinear isolators that follow criterion G1.

General Design Guidelines

As noted earlier, the simplified method is a simpler version of the static method allowed in MOC-2012 [4, 5] and, for the design of the isolation system, it is similar to the static procedure outlined in design guidelines for base-isolated structures of the United States [i.e., 6-9], with adaptations to the seismic design philosophy of current MOC code and original proposals from research studies conducted in Mexico and worldwide. The design of the superstructure is based upon the updated version of the SMSA of Mexican codes for low-rise, squatty shear-wall structures, where elastic response is expected [10].

Therefore, the natural period for the base-isolated structure, T_{as}, is estimated as:

$$T_{as} = 2\pi \sqrt{\frac{W}{gk_{D\min}}} \tag{1}$$

where W is the weight of the structure above the isolation system and $k_{D\min}$ is the effective lateral stiffness of the isolation system at the total design displacement, D_T.

The design displacement of the center of rigidity of the isolation system in the direction of analysis for the Maximum Credible Earthquake (MCE), D_D, is computed as:

$$D_D = F_c S_d(T_{as}) \tag{2}$$

where F_c=1.1 is the load factor for combinations of lateral and gravitational loads according to MOC.

This design displacement has to be amplified to account for orthogonal effects in the horizontal plane (D_{2D}) as follows, based on the extensive parametric study summarized in [17]:

$$D_{2D} = D_D(1.3 - 0.02T_{as}); \quad T_{as} \geq 1.5s \tag{3}$$

The amplification of the total design displacement (D_T) to account for torsional effects due to actual and accidental eccentricities is crudely estimated for design purposes as:

$$D_T = 1.1D_{2D} \tag{4}$$

The increment of 10% for the design displacement due to torsional response is based upon the study presented by Seguín [18]. In the cited study, it is shown that for elastomeric base-isolated structures, amplifications in the bearing displacement due to accidental eccentricities of five percent the longest plan dimension of the structure perpendicular to the direction of force under consideration do not exceed 10%, considering an ample range of the rotational to translational frequency ratio of the isolation system (Ω_{0as}).

The minimum design shear force for the isolation system, the foundation and structural elements below the isolation system, V_{as}, is computed as:

$$V_{as} = k_{D\min}D_T \tag{5}$$

where $k_{D\min}$ is the effective stiffness of the isolation system at the design displacement in the horizontal direction under consideration (Fig. **1**).

The equation to define V_E, the minimum design shear force for the structure above the isolation system is:

$$V_E = \frac{V_{as}}{Q'_{as}} \tag{6}$$

where Q'_{as} is the seismic reduction factor for base-isolated structures that is estimated as outlined in [5]. The value of V_E shall not be taken as less than the following:

1. The lateral seismic force required for a fixed-base structure of the same effective weight, W, and a period equal to the isolated period, T_{as}.

2. The base shear corresponding to the factored design wind load.

It is worth noting that the limits in V_E mostly coincide with current proposal of ASCE-7 [7] as per section 17.5.4.3, but item 3 ("the lateral seismic force required to fully activate the isolation system multiplied by 1.5") is not included in MOC-2012 [4].

The vertical distribution of lateral forces in the superstructure is given by the following expression:

$$F_i = V_E \frac{W_i}{\sum W_i} \tag{7}$$

where F_i, and W_i are respectively the lateral force and the weight above the isolation system of level i. and the remaining terms have been already defined. As observed, in the simplified method of MOC-2012 a uniform lateral load distribution is allowed, as it is required that base-isolated structures should be reasonably uncoupled ($T_{as} / T_E \geq 5$).

Finally, using the SMSA, it is allowed to design the structure above the isolation system by just computing the seismic shear forces that each wall has to carry according to its relative shear stiffness and then assessing the strength required by the walls to carry the attracted shear forces.

As the SMSA is based on an idealized distribution of lateral forces of symmetric shear-wall structures with rigid diaphragms [10], the shear force attracted by any wall j at level i, V_{ji}, is computed from the shear force acting at level i, V_i, as follows:

$$V_{ji} = V_i \frac{F_{AE_{ji}} A_{T_{ji}}}{\sum F_{AE_{ji}} A_{T_{ji}}} \tag{8}$$

where $F_{AE_{ji}}$ is the effective shear area factor of wall j at level i (proposed by SMSA) and $A_{T_{ji}}$ is the cross section area (axial area) of wall j at level i.

For base-isolated structures, elastic response for the superstructure is desirable, so the effective shear area factors, F_{AE} for this condition are:

$$F_{AE} = \begin{cases} 1.5 + \dfrac{h'}{L} - 1.5\left(\dfrac{h'}{L}\right)^2 & if \quad \dfrac{h'}{L} \leq 1 \\\\ 2.2 - 1.5\dfrac{h'}{L} + 0.3\left(\dfrac{h'}{L}\right)^2 & if \quad 1 \leq \dfrac{h'}{L} \leq 2.5 \end{cases} \tag{9}$$

where h' and L are respectively the height and the length of the wall under consideration.

Once all walls are correctly designed by shear forces, the design procedure using the SMSA is over, as no further reviews are needed, for example, assessing torsional effects, overturning moments, P-Δ effects, or limit states such as if lateral drifts meet code requirements, etc. Therefore, the

SMSA is very attractive as it allows studying quickly different options for the design of low-rise buildings with a small computational effort, thus a modest computer with a spreadsheet program becomes a powerful tool to implement the SMSA.

DESIGN EXAMPLE

The application of the simplified method is illustrated with the design of the four-story, 10.8 m in height confined masonry shear wall building the plan of which is shown in (Fig. **2**).

The typical story height for the building is 2.7m. Confined masonry walls are built with fired clay bricks with thickness t=12.5 cm and will be jointed with mortar type I according to Mexican masonry guidelines [i.e., 19]; this mortar is similar to mortar types M or S specified in US masonry codes. The confining RC elements shall be designed to fulfill the requirements specified in Mexican masonry guidelines. Dead loads due to the RC floor system and finishings are ω_D=500 kg/m² at the roof level and ω_D=450 kg/m² at the remaining stories. According to MOC, live loads for vertical load combinations are ω_L=100 kg/m² and ω_L=250 kg/m² for the roof and intermediate levels respectively, and for combinations with lateral loads are respectively ω_L=70 kg/m² and ω_L=180 kg/m². The volumetric weight for the mancraft fired clay brick is γ=1.3 ton/m³ and no finishings will be placed on the walls as the architectural design will take advantage of their aesthetics in both exterior and interior.

The building is located at a distance of 80 km away from the nearest active fault. The characteristics of the soil type profile for the site are available from a detailed geotechnical study and, according to MOC-2008 guidelines [1, 2], the characteristics are: peak acceleration in the bedrock $a_0^r = 0.157g$, fundamental site period $T_s = 0.5s$ and site factor $F_s = 1$, so their corresponding acceleration design spectra is shown in (Fig. **3a**) and the control periods T_a and T_b that define the plateau are $T_a = 0.175s$ and $T_b = 0.6s$.

It is required to perform a preliminary design for a base-isolation project with lead-rubber bearings (LRB) using the simplified method.

Review of Requirements for Application

The first step is to ensure that all requirements to use the simplified method are satisfied. Regarding the 12 conditions of structural regularity, from the description of the building, the structural system and the plan shown in (Fig. **2**), it can be concluded that conditions A1, A4, A5, A6, A7, A8, A9 and A10 are reasonably satisfied. Condition A2 (slenderness) is also satisfied, as H/L₂=10.8/9.5=1.14<1.5. Condition A3 is also met as L₁/L₂=18/9.5=1.89<2.

The plan for the structure (Fig. **2**) is completely symmetric with respect to the Y axis; however, there is a small eccentricity offset with respect to the X axis that has to be evaluated in order to discern if regularity conditions A11 and A12 are satisfied. This evaluation is done using a simplified calculation to estimate the static eccentricity that is allowed when using the simplified method (Eq. 10), as schematically shown in (Fig. **4**) and explained in greater detail elsewhere [i.e., 10, 19, 20]:

$$e_{sj} = \frac{\left| \sum_{i=1}^{n} x_i F_{AEi} A_{Ti} \right|}{\sum_{i=1}^{n} F_{AEi} A_{T_i}} \leq 0.05 B_j \tag{10}$$

From the dimensions of the plan shown in (Fig. **2**) and the illustration in (Fig. **4**), this review is easily done with the help of Table **1**. From the results computed in Table **1** it is obtained that:

$$e_{sy} = \frac{\left| \sum_{i=1}^{n} y_i F_{AEi} A_{Ti} \right|}{\sum_{i=1}^{n} F_{AEi} A_{T_i}} = \frac{0.6173}{6.7163} = 0.092\,m \leq 0.05(9.5) = 0.475\,m$$
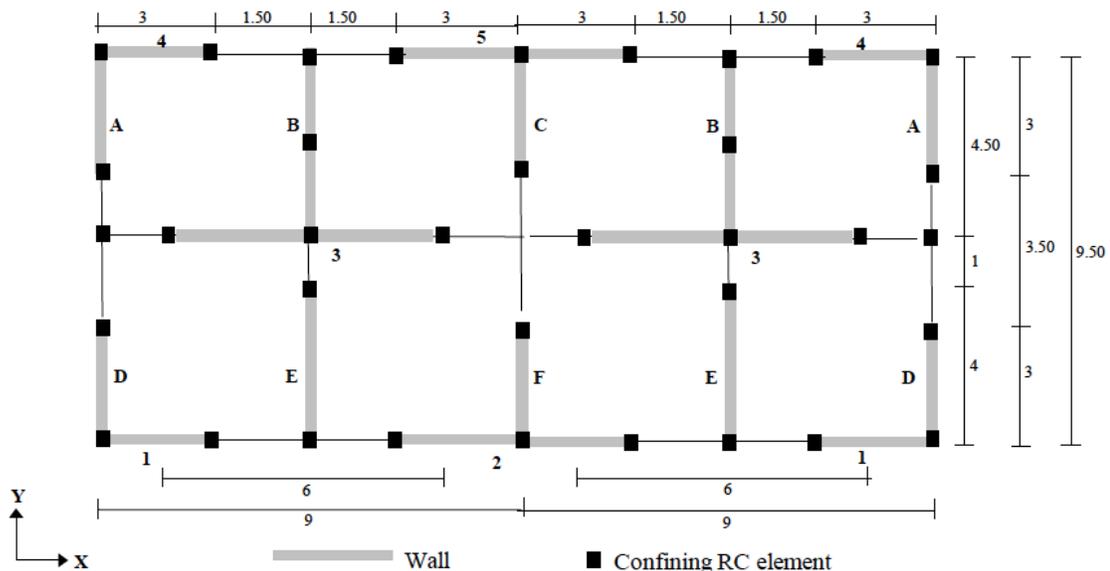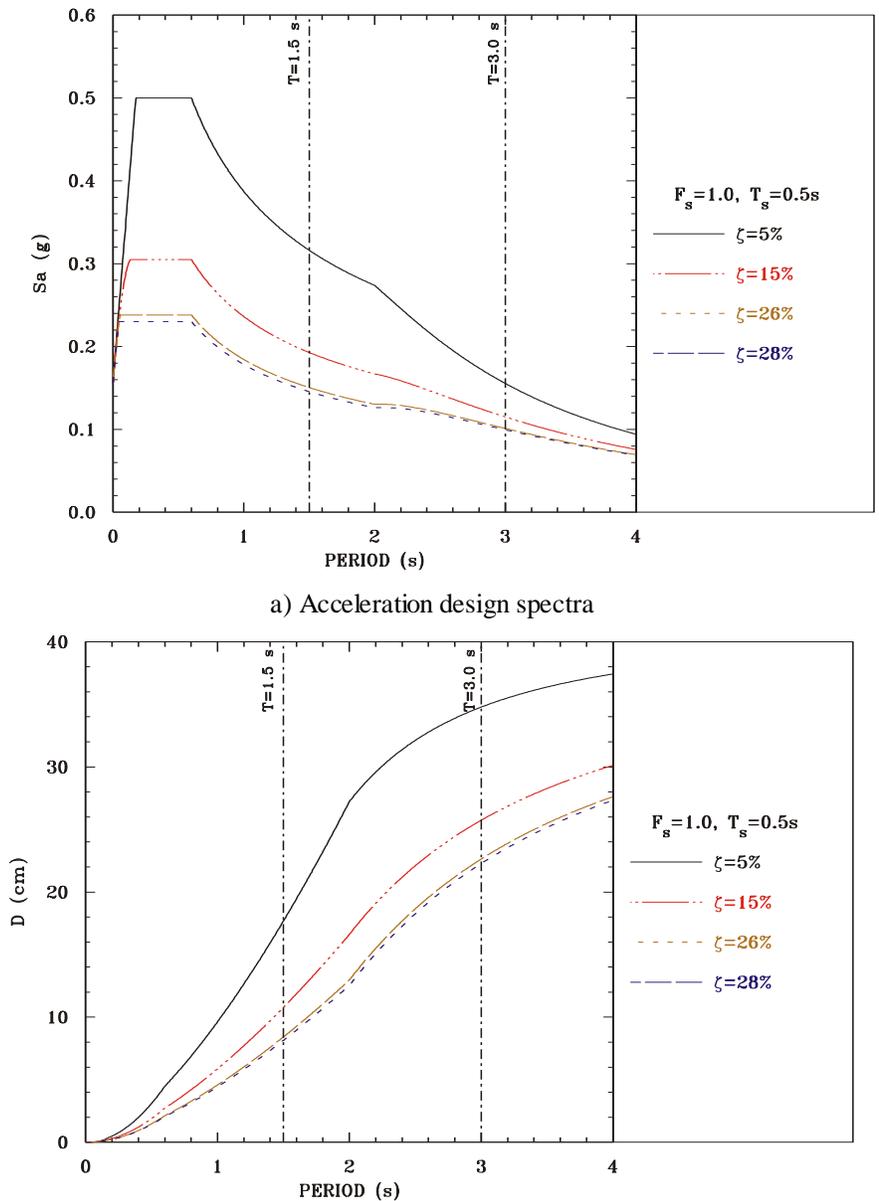

Fig. (2). Plan for a four-story, confined masonry shear wall building (dimensions in meters).

a) Acceleration design spectra

Fig. (3). Design spectra for a site with $a_0^r = 0.157g$, $T_s = 1.5$s, $P_s = 0.7832$ and $F_s = 1$.
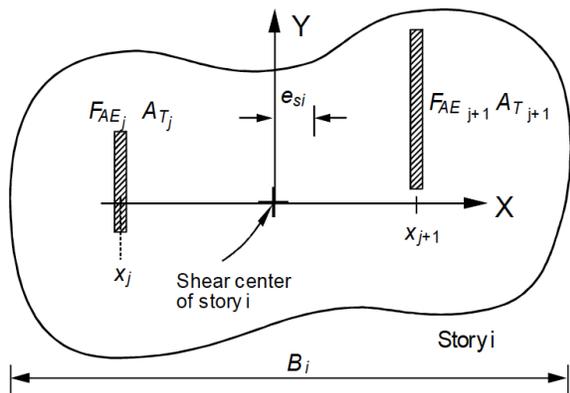
Fig. (4). Definition of static eccentricity for the SMSA of Mexican Codes.

therefore, condition A11 is satisfied as $e_{sy} / B = 0.092 / 9.5 = 0.01 \leq 0.05$. As the bearings will be placed as symmetric as possible, condition A12 should also be met, since e_{sa} should be similar to e_{sy}, therefore, within allowable limits.

From the definition of the problem and the plan of the building (Fig. **2**), it is found that requirement B1 is fulfilled. Requirement B2 is satisfied as the building has four stories and its total height is 10.8m. From the information that is given in the example, it is concluded that requirements C and D are also fulfilled. Finally, requirements E, F and G will be met within the design process. Therefore, it can be concluded that the building structure in (Fig. **2**) satisfies the requirements to use the simplified method.

Table 1.   Calculation of the static eccentricity for the SMSA of Mexican Codes.

Wall type	y_i (m)	A_{Ti} (m^2)	h/L	F_{AEi}	$F_{AEi} A_{Ti}$	$y_i F_{AEi} A_{Ti}$
1	-4.6875	0.375	0.90	1.185	0.4444	-2.0830
1	-4.6875	0.375	0.90	1.185	0.4444	-2.0830
2	-4.6875	0.75	0.45	1.6463	1.2347	-5.7876
3	0.25	0.75	0.45	1.6463	1.2347	0.3087
3	0.25	0.75	0.45	1.6463	1.2347	0.3087
4	4.6875	0.375	0.90	1.185	0.4444	2.0830
4	4.6875	0.375	0.90	1.185	0.4444	2.0830
5	4.6875	0.75	0.45	1.6463	1.2347	5.7876
Σ					6.7163	0.6173

Estimate of the Seismic Weight and Fixed-Base Periods

The estimate of the seismic weight is crudely done by computing the own weight of the structural system due to dead and live loads, plus the own weight of walls and the weight of the slab above the isolation system. For simplicity, it will be assumed that the slab above the isolation system weights exactly the same as the slabs in intermediate floors. It can be obtained from (Fig. **2**) that the total length of walls at each plan is L_m=71m and the linear weight of walls is ω_w=1.3(2.7)(0.125)=0.44 Ton/m. Therefore, from the remaining data given in the example the total weight W can be estimated as:

$$W = W_{structure} + W_{isolation\ slab} = W_{stories} + W_{walls} + W_{isolation\ slab}$$

$$W = \left[0.57 + 3(0.63)\right](18)(9.5) +$$

$$4(71\ m)(0.44\ Ton/m) + 0.63(18)(9.5)$$

$$W = 420.66 + 124.61 + 107.73 \rightarrow W = 653\ Ton$$

An estimate of the natural period for the fixed-base structure is also necessary. For this purpose, the approximate formula proposed by Murià and González [21] for fixed-base confined masonry buildings is used. Therefore, as the building is four stories in height (N=4), then:

$$T_E \approx 0.04N = 0.04(4) = 0.16s$$

Design of the Isolation System

There are some options already available in the literature for the preliminary design of lead-rubber bearings [i.e., 22-24]. In this example, the general procedure outlined in [24] will be adapted and used.

The primary curve for the isolation system should be defined in order to satisfy requirement G1 schematically shown in (Fig. **1**); therefore, $k_{D\min} = k_{eff} \geq \frac{1}{3}k_{eff2}$. As the postyielding stiffness ratio k_2/k_1 depends primarily on the characteristics of the isolation system and since $k_2/k_1 \cong 0.1$ for lead-rubber bearings, to satisfy the primary curve for $k_2/k_1 = 0.1$, then:

$$V_2 = 0.6V_{\max} = 0.6V_{as} \tag{11}$$

$$V_{as} = 1.8V_y \tag{12}$$

$$\Delta_y = \frac{D_T}{9} \tag{13}$$

$$k_1 = 5\frac{V_{as}}{D_T} = 5k_{eff} \tag{14}$$

$$k_2 = 0.5k_{eff} \tag{15}$$

The initial yield force $V_y = \kappa W$ for the isolation system should be proposed or estimated as a function of the expected design displacement D_T and the equivalent viscous damping at that design displacement β_D, which can be estimated for a bilinear system as:

$$\beta_D = \frac{1}{2\pi}\left[\frac{E_{cycle}}{k_{D\min}(D_T)^2}\right] = \frac{1}{2\pi}\left[\frac{E_{cycle}}{V_{as}D_T}\right] \tag{16}$$

$$E_{cycle} = 4V_c(D_T - \Delta_y) = 4(V_{as} - k_2 D_T)(D_T - \Delta_y) \tag{17}$$

where V_c is the characteristic shear force (Fig. **1**).

If $k_2/k_1 = 0.1$, these expressions can be further reduced from Eqs. 11 to 15 as:

$$E_{cycle} = 3.6V_y(D_T - \Delta_y) = 28.8V_y\Delta_y$$

$$\beta_D = \frac{1}{2\pi}\left[\frac{28.8V_y\Delta_y}{1.8V_y(9\Delta_y)}\right] = 0.283$$

Therefore, it is observed that β_D is constant once the primary curve is defined in terms of k_2/k_1 and Δ_y/D_T, regardless of the yield force V_y, a fact that eases considerably the design procedure.

In addition, T_{as} should be proposed to meet requirements E and F. Therefore, $T_{as} = 2s$ is initially proposed, as $T_{as}/T_E \approx 12.5 > 5$.

Then, k$_{Dmin}$ is computed from Eq. 1:

$$k_{D\min} = \frac{4\pi^2 W}{gT_{as}^2} = 6.57\ Ton/cm$$

Assuming $\kappa = 0.11$, the yield shear force and the maximum shear force for the isolation system are:

$$V_y = \kappa W = 71.83 \; Ton$$

$$V_{as} = 1.8 V_y = 129.29 \; Ton$$

It is clear from Eq. 5 and (Fig. **1**) that the total design displacement D_T is:

$$D_T = \frac{V_{as}}{k_{D\min}} = 19.68 \; cm$$

The total design displacement D_T can be estimated in terms of the displacement D_D from Eqs. 2 to 4 as:

$$D_{2D} = 1.1 D_D (1.3 - 0.02 T_{as}) = 1.386 D_D$$

$$D_T = 1.1 D_{2D} = 1.525 D_D$$

Then:

$$D_D = \frac{D_T}{1.525} = 12.91 \; cm$$

This displacement capacity D_D has to be compared with the demand spectral displacement S_d, and for an acceptable design, $S_d \le D_D$. Given that $T_{as} = 2s$ and $\zeta_{as} = \beta_D = 0.28$, from the displacement design spectra for the site shown in (Fig. **3b**), it is obtained that $S_d = 12.54 \; cm < D_D = 12.91 \; cm$, then, the global design is acceptable. Therefore, the remaining parameters that define the primary curve for the isolation system are:

$$\Delta_y = \frac{D_T}{9} = 2.19 \; cm$$

$$k_1 = 5 k_{D\min} = 32.848 \; Ton/cm$$

$$k_2 = 0.1 k_1 = 3.285 \; Ton/cm$$

A preliminary design for the isolation system is required. This is done with the general procedure outlined in [24], but it is clear that any suitable method can be used for this purpose. Assuming that six LRB will be used as schematically shown in (Fig. **5**) ($N_{ais} = 6$), and considering that $G_{ais} = 10.2 \; kg/cm^2$ and $\tau_{pb} = 107 \; kg/cm^2$, one can approximately determine their dimensions as follows:

$$\phi = 3 D_T = 59.04 \; cm$$

$$h = \frac{\pi \varphi^2 G_{ais}}{4 k_2 / N_{ais}} = 51.01 \; cm$$

$$h/\phi = 51.01/59.04 = 0.86 > 0.8 \text{, but close enough}$$

$$A_{pb} = \frac{V_y - k_2 \Delta_y}{N_{ais} \tau_{pb}} = 100.70 \; cm^2$$

$$\phi_{pb} = \sqrt{\frac{4 A_{pb}}{\pi}} = 11.32 \; cm$$

$$\varphi_{pb}/\varphi = 11.32/59.04 = 0.19 < 0.25$$

Therefore, from the preliminary design one can conclude that six LRB with diameter $\phi = 60 \; cm$ and height $h = 45 \; cm$ can be used, adjusting the diameter of the lead plug ϕ_{pb}. These required adjustments can be easily done as follows:

$$h/\phi = 45/60 = 0.75 < 0.8$$

$$D_T = \frac{\phi}{3} = 20 \; cm$$

$$\Delta_y = \frac{D_T}{9} = 2.22 \; cm$$

$$k_2 = \frac{\pi \varphi^2 G_{ais}}{4 h / N_{ais}} = 3.845 \; Ton/cm$$

$$A_{pb} = \frac{V_y - k_2 \Delta_y}{N_{ais} \tau_{pb}} = 98.57 \; cm^2$$

$$\phi_{pb} = \sqrt{\frac{4 A_{pb}}{\pi}} = 11.2 \; cm$$

$$\varphi_{pb}/\varphi = 11.2/60 = 0.19 < 0.25$$

These adjustments also modify k_2/k_1, $k_{D\min}$ and β_D. Given that the adjustment in h is somewhat important, it is reviewed how these adjustments modify the design properties and if the design still satisfies the displacement demand. Therefore:

$$k_1 = \frac{V_y}{\Delta_y} = \frac{71.83}{2.22} = 32.354 \; Ton/cm$$

$$k_2/k_1 = 3.845/32.354 = 0.119$$

$$V_{as} = V_y + k_2 (D_T - \Delta_y) = 140.19 \; Ton$$

$$k_{D\min} = \frac{V_{as}}{D_T} = 7.009 \; Ton/cm$$

Reviewing that the characteristics for the primary curve (Fig. **1**) are satisfied (requirement G1):

$$\Delta_2 = 0.2 D_T = 4 \; cm$$

$$V_2 = V_y + k_2 (\Delta_2 - \Delta_y) = 78.67 \; Ton$$

$$k_{eff2} = \frac{V_2}{\Delta_2} = 19.667 \; Ton/cm$$

$$k_{D\min}/k_{eff2} = 0.36 > \frac{1}{3} \text{, therefore, o.k.}$$

Then, β_D for this isolation system is:

$$E_{cycle} = 4 (V_{as} - k_2 D_T)(D_T - \Delta_y) = 4,500.62 \; Ton - cm$$

$$\beta_D = \frac{1}{2\pi} \left[\frac{E_{cycle}}{V_{as} D_T} \right] = 0.26$$

Then T_{as} and D_D are:

$$T_{as} = 2\pi \sqrt{\frac{W}{g k_{D\min}}} = 1.94 \; s$$

$$D_D = \frac{D_T}{1.526} = \frac{20}{1.526} = 13.11 \; cm$$

Given that $T_{as} = 1.94 \; s$ and $\zeta_{as} = \beta_D = 0.26$, from the displacement design spectra for the site shown in (Fig. **3b**), it

is obtained that $S_d = 12.38\ cm < D_D = 13.11\ cm$. Therefore, the preliminary design is verified and 6 LRB can be used with the following general characteristics: $\phi = 60\ cm$, $h = 45\ cm$ and $\phi_{pb} = 11.2\ cm$.

It is worth noting that one has still to dimension the thicknesses of the rubber and steel layers of the LRBs, which can be preliminary done with procedures available in the literature [i.e., 22, 23]. Also, a final design necessarily requires an open and close communication with the manufacturer to adjust the characteristics of the isolators, besides many other issues and details of paramount importance in the design and construction of a base-isolation project.

Design of the Structure above the Isolation System

The minimum design shear force for the structure above the isolation system V_E is defined in Eq. 6; therefore, it is required to compute Q'_{as} as specified in [5].

As $T_{as} / T_E = 1.94 / 0.16 = 12.13 > 5$,

then $Q'_{as} = R_{as}\rho_{as} \geq 1$. Given that the structure above the isolation system is composed of confined masonry shear walls made with solid units, then the overstrength index factor is $R_{a0} = 1.6$. As $T_E = 0.16s < T_a = 0.175s$, then, the overstrength factor R_{as} is:

$$R_{as} = R_{a0} + 0.3\left(1 - \sqrt{\frac{T_E}{T_a}}\right) = 1.61$$

It can be observed from Fig. (**5**) that the redundancy factor ρ_{as} (explained in [5]) is ruled by the isolation system. Therefore, from (Fig. **5**), $\rho_{as} = 0.8$ should be taken in the Y direction as the isolation system is forming an equivalent "three parallel one-bay frames system", whereas in the X direction, $\rho_{as} = 1$ because the isolation system is forming an

equivalent "two parallel two-bay frames system". Therefore, as ρ_{as} is different in both orthogonal directions, then:

$$Q'_{as_x} = 1.61(1) = 1.61$$

$$V_{E_x} = \frac{V_{as}}{Q'_{as_x}} = \frac{140.19}{1.61} = 87.1\quad Ton$$

$$Q'_{as_y} = 1.61(0.8) = 1.29$$

$$V_{E_y} = \frac{140.19}{1.29} = 108.8\quad Ton$$

It is worth noting again that the values of V_E shall not be taken as less than: (1) the lateral seismic force required for a fixed-base structure (V_{Efb}) of the same effective weight, W, and a period equal to the isolated period, T_{as} or, (2) the base shear corresponding to the factored design wind load (V_{Ew}). The computation V_{Efb} and V_{Ew} according to MOC-2012 requires of additional information and explanations that are not yet available in English language, particularly for the wind design. Therefore, V_{Ew} was not assessed as it is not germane to illustrate the method.

On the other hand, V_{Efb} was assessed according to the seismic guidelines for conventional buildings of MOC-2008 [1, 2]. Because of space constraints, and in the interest of the understanding of the simplified method, this calculation will be briefly described as follows. With $T_{as} = 1.94s$ and $\varsigma = 0.05$, it is obtained from the acceleration design spectra shown in (Fig. **3a**) that $S_a = 0.278g$. As $a' = S_a / Q'R\rho$ then, it is required to define the overstrength factor R, the redundancy factor ρ and Q' factor for the structural system shown in (Fig. **4**). From the analysis of Fig. (**4**), it is obtained that $\rho = 1$ in both orthogonal directions. As $T_{as} = 1.94s > T_a = 0.175s$, then $R = R_0 = 2.5$ for confined masonry shear wall structures. The computation of Q' is
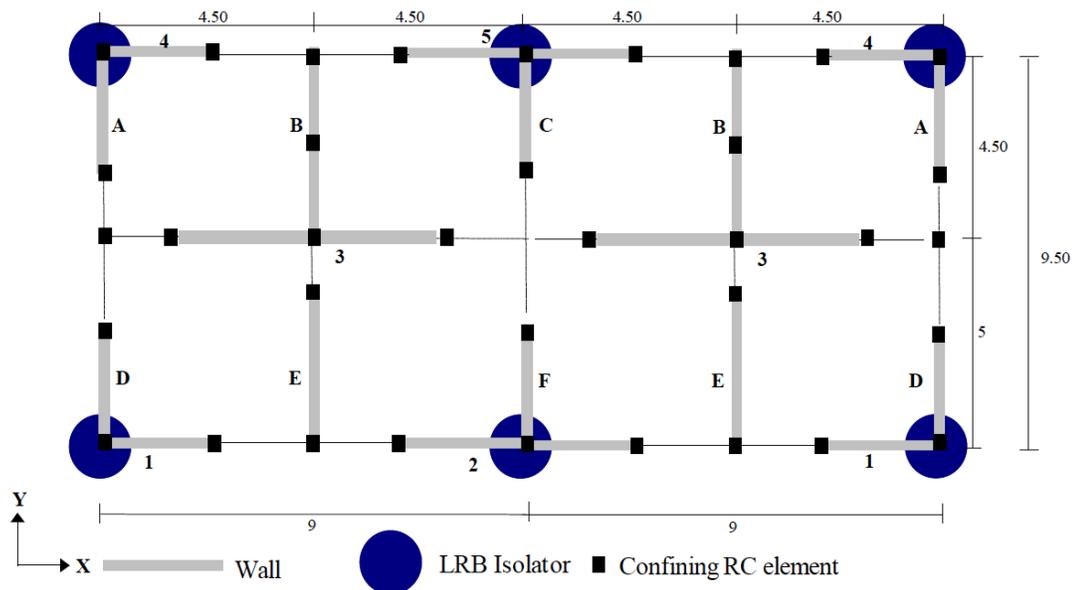

Fig. (5). Schematic location of LRB for the isolation project (dimensions in meters).

more complex, so following the guidelines described in [2], it is obtained that $Q'=1.80$. Then:

$$a' = \frac{0.278g}{1.80(2.5)(1)} = \frac{0.278g}{4.50} = 0.062g$$

$$V_{Efb} = 1.1a'W = 44.4\,Ton$$

Therefore, it is clear that $V_{E_x} > V_{Efb}$ and $V_{E_y} > V_{Efb}$, so V_{E_x} and V_{E_y} should be used for the design of the structure above the isolation system.

The vertical distribution of lateral forces in the super-structure is given by Eq. 7. Therefore, from the information provided in this example, and considering that for seismic analysis, the masses (weights) of walls within an interstory are distributed in halves between the corresponding adjacent stories, it is obtained that:

$$F_{4x} = 87.1\frac{113.09}{530} = 18.59\,Ton \;;$$

$$F_{1x} = F_{2x} = F_{3x} = 87.1\frac{138.97}{530} = 22.84\,Ton$$

$$F_{4y} = 108.8\frac{113.09}{530} = 23.22\,Ton \;;$$

$$F_{1y} = F_{2y} = F_{3y} = 108.8\frac{138.97}{530} = 28.53\,Ton$$

The predesign of the walls using the SMSA requires first to assess the compressive strength required for the masonry (f^*_m) and the solid clay bricks (f^*_p) to withstand the vertical load combinations according to Mexican masonry guidelines [19]. It is beyond the scope of this paper to illustrate this simple procedure, which is described in detail in [25]. Summarizing the design for gravitational loads, the critical element is wall F at the first story (Fig. 3), and accordingly, it is required for the masonry to have a minimum design compressive strength $f_m^* = 22.75\,kg/cm^2$, and since mortar type I is used, therefore, the fired clay bricks should possess a compressive strength $f_p^* = 65\,kg/cm^2$. With such properties for the bricks and as mortar type I is used, the index shear strength for design for the masonry is $v_m^* = 3.5\,kg/cm^2$.

The seismic design for the shear walls using the SMSA will be illustrated for the walls of the first story in the critical Y direction, where $V_{E_y} = 108.8\,Ton$. From the information given in the definition of the example, the plan shown in (Fig. 4) and Equation 9, it is obtained that, in the Y direction:

$$A_y = \sum A_{Ti}F_{AE_i} = 6(0.375)(1.185) +$$
$$2(0.563)(1.56) + 2(0.5)(1.4916) = 5.91\,m^2$$

The shear forces that each wall type have to resist in the Y direction (V_{ji_y}) are computed from Eq. 8 taking $V_i = V_{1y} = V_{E_y} = 108.8\,Ton$. These computed shear forces V_{ji_y} are compared in Table **2** with the estimated shear strength capacity for each wall according to Mexican masonry guidelines, V_{mR} [i.e., 19, 26]. Details on the assessment of V_{mR} for each wall considering that horizontal shear reinforcement is not provided and are presented in [25].

From the observation of the data presented in Table **2**, it can be concluded that only wall B does not have enough shear strength capacity without horizontal shear reinforcement. However, the SMSA also allows to do a gross strength evaluation where $\sum V_{mR} \geq V_{E_y}$ and, from the data given in (Table **2** and Fig. **3**), it is obtained that $\sum V_{mR_y} = 114.5\,Ton > V_{E_y}$, so the design is tight, but acceptable. In addition, the design for the walls in the Y direction can be easily improved providing the minimum horizontal shear reinforcement as established by the Mexican masonry guidelines [i.e., 19, 26]. It is beyond the scope of this paper to present such refinement.

As reported [25], all walls in the X direction have enough shear strength capacity to resist the earthquake demands without horizontal shear reinforcement and for this direction, the gross evaluation yields $\sum V_{mR_x} = 108.8\,Ton > V_{Ex} = 87.1\,Ton$.

This example illustrates that the simplified method for the seismic design of low-rise, shear wall base-isolated structures prescribed in MOC-2008 is easy to apply. Most of the required calculations can be easily programmed or implemented in spreadsheet software (for example: http://www.researchgate.net/profile/Arturo_Tena-Colunga/contributions/?ev=prf_act).

REVIEW USING NONLINEAR DYNAMIC ANALYSIS

The effectiveness of the design obtained using the simplified method is assessed with nonlinear time-history analyses done according to the same MOC-2012 guidelines [1, 2].

For this purpose, 16 pairs of orthogonal horizontal ground motion components corresponding to eight different earthquakes events M>6.4 recorded in firm soils in Mexico were selected, as identified in Tables **3** and **4**. The selected records were individually scaled in order that the dominant ground component will match the spectral acceleration $S_a = 0.274g$ for $T_{as} = 2s$ for the design spectrum for $\zeta = 0.05$ shown in (Fig. **3a**). The same factor was used to scale the two orthogonal components (E-W and N-S), for each pair of acceleration records under study. These records were corrected, filtered and processed similarly as reported in a previous study [17], as they belong to the same database.

Nonlinear time-history analyses were conducted using the 3D-Basis software [27]. Bidirectional input of the scaled ground motions was considered, switching components in order to crudely consider directivity effects. LRB were modeled individually using the hysteretic element with biaxial behavior available in 3D-Basis. The structure above the isolation system was modeled as a 3D shear building. In fact, this is the modeling assumption behind the SMSA allowed by Mexican codes for low-rise shear wall buildings [i.e., 10, 11]. Twelve modes were considered in the 3D-Basis analyses. It is worth noting that the translational fixed-base periods computed with 3D-Basis under this modeling assumption were $T_{E_x} = 0.187s$ and $T_{E_y} = 0.175s$, reasonably close to the natural period estimated for preliminary design purposes ($T_E \approx 0.04N = 0.16s$).

Peak dynamic responses obtained from the nonlinear dynamic analyses are summarized in Table **3** (structure above the isolation system) and Table **4** (isolation system). It is

Table 2.   **Review of the shear capacity of the first story walls in the Y direction (Fig. 3).**

Wall Type	A_{Ti}/A_y	V_{ji_y} (Ton)	V_{mR} (Ton)	Note
A	0.0752	8.18	8.46	✔
B	0.1484	16.15	14.07	✘
C	0.0752	8.18	10.93	✔
D	0.0752	8.18	9.04	✔
E	0.1261	13.72	14.16	✔
F	0.0752	8.18	12.09	✔

Table 3.   **Peak dynamic responses for the superstructure of the base-isolated masonry building under bidirectional input of the ground motions.**

Station	Quake	Peak roof displacement (mm)		Peak drift angle		Peak roof acceleration (g)		Normalized peak base shear	
		X	Y	X	Y	X	Y	V_{xe}/W	V_{ye}/W
AZIH	*85/09/19*	1.23	1.65	0.00019	0.00026	0.134	0.157	0.108	0.127
CALE	*85/09/19*	1.19	0.97	0.00018	0.00015	0.130	0.095	0.105	0.075
PAPN	*85/09/19*	1.03	2.02	0.00016	0.00031	0.113	0.195	0.090	0.155
SUCH	*85/09/19*	1.58	1.24	0.00024	0.00019	0.173	0.120	0.139	0.095
UNIO	*85/09/19*	1.09	1.69	0.00016	0.00026	0.122	0.162	0.094	0.130
VILE	*85/09/19*	1.52	1.33	0.00023	0.00021	0.165	0.129	0.134	0.103
AZIH	*85/09/20*	1.70	1.43	0.00026	0.00022	0.184	0.138	0.149	0.109
COPL	*95/09/14*	1.08	1.92	0.00017	0.00030	0.120	0.184	0.095	0.148
PTSU	*95/10/09*	1.28	1.45	0.00020	0.00022	0.139	0.139	0.112	0.112
ZIIG	*96/07/15*	1.49	1.42	0.00023	0.00022	0.165	0.138	0.129	0.108
PTSU	*97/01/11*	1.18	1.56	0.00018	0.00024	0.129	0.152	0.104	0.120
UNIO	*97/01/11*	1.14	1.87	0.00017	0.00029	0.126	0.182	0.099	0.144
CUER	*99/06/15*	1.45	1.45	0.00022	0.00022	0.158	0.139	0.127	0.111
CSER	*99/09/30*	1.23	1.60	0.00019	0.00025	0.134	0.154	0.108	0.124
CUER	*99/09/30*	1.07	1.28	0.00016	0.00020	0.116	0.123	0.094	0.099
PPIG	*99/09/30*	1.02	1.29	0.00016	0.00020	0.111	0.123	0.090	0.100
σ		1.27	1.51	0.00019	0.00023	0.139	0.146	0.111	0.116
W=653 Ton= 6,405.93 kN									

worth noting that, according to the time-history procedure defined in MOC-2012 [4], if seven or more pairs of time-history analyses are performed, the average value of the response parameter of interest shall be used for design, which is why the mean value (μ) is also provided in these tables.

It can be deducted from the results presented in Table **3** that the isolation of the superstructure is effective. The superstructure practically responses as a rigid body with negligible peak roof displacements with respect to the isolation level and peak drift angles (at first story) considerable below the limit Δ=0.0015 allowed by MOC-2012 for base-isolated structures with confined masonry walls built with solid units [4, 5].

The peak acceleration profile is also fairly uniform, as the structure displaces almost as a rigid body. The average amplification factor between the peak acceleration at the first story (not shown) and the peak roof acceleration (Table **3**) is 1.03. Therefore, these results illustrate the rationality behind the uniform lateral force distribution for the superstructure proposed in Eq. 7. Finally, normalized shear strength capacities for the confined masonry structure are $\sum V_{mR_x}/W = 0.167$ and $\sum V_{mR_y}/W = 0.175$, higher than the average and maximum normalized peak base shear demands V_{xe}/W and V_{ye}/W reported in Table **3**.

It can be confirmed from the results presented in Table **4** that the base-isolation project is effective, as the reported

Table 4.   Peak dynamic responses for the isolation system of the base-isolated masonry building under bidirectional input of the ground motions.

Station	Quake	Peak bearing displacements (cm)				Normalized peak shear			
		D_{xas}(cm)	D_{yas}(cm)	D_{as}(cm)	D_{as}/D_T	V_{xa}/W	V_{ya}/W	V_{xe}/V_{xa}	V_{ye}/V_{ya}
AZIH	*85/09/19*	7.32	11.12	13.17	0.66	0.133	0.156	0.814	0.813
CALE	*85/09/19*	5.96	2.84	6.29	0.31	0.129	0.091	0.813	0.820
PAPN	*85/09/19*	8.32	17.10	18.96	0.95	0.110	0.189	0.820	0.818
SUCH	*85/09/19*	12.82	9.83	16.06	0.80	0.170	0.117	0.815	0.818
UNIO	*85/09/19*	6.45	11.02	11.79	0.59	0.115	0.161	0.821	0.812
VILE	*85/09/19*	12.71	8.36	14.84	0.74	0.165	0.127	0.812	0.810
AZIH	*85/09/20*	15.01	10.39	16.39	0.82	0.184	0.134	0.812	0.818
COPL	*95/09/14*	5.03	14.99	15.23	0.76	0.111	0.183	0.814	0.812
PTSU	*95/10/09*	8.46	7.30	8.70	0.44	0.138	0.138	0.813	0.813
ZIIG	*96/07/15*	12.75	9.35	15.06	0.75	0.157	0.132	0.821	0.822
PTSU	*97/01/11*	12.02	10.13	15.66	0.78	0.128	0.148	0.814	0.814
UNIO	*97/01/11*	9.51	16.32	16.47	0.82	0.120	0.176	0.824	0.817
CUER	*99/06/15*	10.59	8.22	10.65	0.53	0.157	0.135	0.813	0.823
CSER	*99/09/30*	7.27	11.01	11.97	0.60	0.133	0.152	0.812	0.813
CUER	*99/09/30*	4.90	6.44	6.91	0.35	0.115	0.122	0.812	0.813
PPIG	*99/09/30*	5.64	6.46	8.57	0.43	0.110	0.123	0.814	0.812
σ		9.05	10.06	12.92	0.65	0.136	0.146	0.815	0.816
D_T=20 cm, *W*=653 Ton= 6,405.93 kN									

peak LRB displacement demands D_{as} are smaller than the total design displacement D_T. In addition, the normalized shear strength capacity for the isolation system is $V_{as}/W = 0.215$, higher than the average and maximum normalized peak shear demands for the isolation system V_{xa}/W and V_{ya}/W reported in Table **4**. Finally, a fairly constant reduction in the shear force transmitted to the superstructure (81.5%) is also observed.

CONCLUDING REMARKS

A simplified method for the seismic design of low-rise, base-isolated shear wall structures proposed in the model code MOC-2012 of Mexico was presented. The spirit behind this simple method is to help promote the application of base-isolation in regular and squatty shear wall structures in Mexico by facilitating their seismic design process.

A simple design example was presented to help illustrate the application of the method. The effectiveness of the method in leading to safe structural designs was confirmed with non-linear time-history analyses (NTHA) done according to the same MOC-2012 guidelines. It was verified with NTHA that: a) the superstructure practically responses as a rigid body with negligible peak roof displacements with respect to the isolation level, b) peak drift angles at the superstructure are considerable below the limit Δ=0.0015 allowed by MOC-2012 for base-isolated structures with confined masonry

walls built with solid units, c) the peak acceleration profile for the superstructure is fairly uniform, as the structure displaces almost as a rigid body. The assessed average amplification factor between the peak acceleration at the first story and the peak roof acceleration is 1.03 only, d) the normalized shear strength capacities for the confined masonry are higher than the average and maximum normalized peak base shear demands, e) peak LRB displacement demands are smaller than the total design displacement the isolators can accommodate and, f) the normalized shear strength capacity for the isolation system is higher than the average and maximum normalized peak shear demands for the isolation system. Therefore, it can be concluded that this simplified method seems to be a promising tool to achieve safe designs for low-rise base-isolated shear wall structures in a practical way.

CONFLICT OF INTEREST

The author(s) confirm that this article content has no conflicts of interest.

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