

Acceleration-Based Design of Low-Rise Based-Isolated Buildings

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SUMMARY:

Four versions of a four-story building, one that considers a rigid base and three that consider seismic isolation, are designed and their seismic performance evaluated. The building, which houses a hospital facility, is assumed to be located on a firm soil site located at the Mexican Pacific Coast, and a set of twenty-two ground motions is used to perform nonlinear time-history analyses. The structural layout of the super-structure can be considered to be regular in plan and height in terms of strength, stiffness and mass. While the conception and preliminary design of the isolated versions were achieved through an acceleration-based format, the rigid base version was designed according to current design practice. The weight of the isolated versions of the building is similar to that of their rigid-based counterpart; and while the seismic performance of all versions satisfies the operational performance level from structural and nonstructural points of view, significant damage is expected on the acceleration-sensitive contents of the rigid-base version. In the case of the base-isolated versions, the absolute acceleration demands are adequately controlled within design thresholds established according to the required seismic performance of their contents.

Keywords: Seismic base isolation, Acceleration-based design, Rubber bearings, Acceleration-sensitive contents

1. INTRODUCTION

The response of structural systems supported on a fixed base during recent earthquakes has caused large losses on acceleration sensitive contents and non-structural elements (Todd et al. 1994, Elnashai et al. 2010). The available evidence suggests that the acceleration demands on such systems reach unacceptable levels in terms of the level of damage acceptable to make possible an operational facility after the occurrence of intense ground motion (Comerio and Holmes 2004, Villaverde 2006). As a result, many essential facilities, such as hospitals, police stations, fire centrals and telecommunication centrals, have continuously been shut down after severe ground motion despite their importance and the fact that their structural systems remained practically undamaged. In some cases, such as that of hospital facilities, the replacement cost of contents and non-structural elements can significantly exceed that of the structural system (Taghavi and Miranda 2003, Takahashi and Shiohara 2004). Hence, adequate seismic design of essential facilities implies the need to control the acceleration demands on the different acceleration-sensitive contents of a building.

An effective way to control the acceleration demands in buildings is the use of base isolation systems (Clark et al. 2002, Nagarajaiah and Xiaohong 2000). In spite of this, the design of this type of systems is usually based on displacement and strength considerations, with no explicit effort to control the acceleration demands in contents. Within this context, it is important to understand that some contents in isolated structures have the potential to exhibit high acceleration demands due to dynamic interactions and the effect of higher modes (Kelly 1982, Kelly and Tsai 1985).

2. BASIS FOR ACCELERATION-BASED DESIGN OF RUBBER ISOLATION SYSTEMS

The acceleration-based control concepts discussed by Zuñiga and Teran-Gilmore (2012) in an accompanying paper are used herein to formulate an acceleration-based format for the conception and design of a base-isolation system for low-rise buildings. The acceleration-based methodology, applicable to rubber isolation systems with additional energy dissipation capacity provided by viscous dampers, establishes an acceleration threshold (as a function of the required performance for acceleration-sensitive contents) that should be met by the isolated structural system when subjected to the design ground motion. In quantitative terms, the lateral stiffness and damping requirements for the isolation system are established through the use of a pseudo-acceleration response spectrum.

2.1. Participation factors

Through the study of the modal participation factors associated to rubber isolation systems with additional damping, Zuñiga and Teran (2012) have formulated recommendations in terms of the structural properties required by low-rise isolation systems to minimize the effect of upper modes. Within this context, it is of particular importance the value exhibited by the ratio between the effective period of the isolation system (the period the isolation system would have if all the mass in the structure was assigned to its translational degree-of-freedom) and the fundamental period of vibration of the super-structure on a rigid base (T_B/T_S). For values of additional damping that can be considered practical for a rubber base-isolation system, values of T_B/T_S greater than eight promote a dynamic response fully dominated by the first mode of vibration. This implies that the lateral-stiffness of the super-structure should be designed relative to that of its isolation system, and that T_B/T_S ratios ranging from two to three, usually recommended for the strength-based or displacement-based design of base-isolation systems (e.g., Naeim and Kelly 1999), may not result in adequate seismic performance from an acceleration point of view.

2.2. Mass ratio

Another variable relevant to acceleration demands on rubber base-isolated systems with additional damping is the mass ratio γ , defined as (Naeim and Kelly 1999):

$$\gamma = \frac{\sum_{i=1}^{N-1} m_i}{m_B + \sum_{i=1}^{N-1} m_i} \quad (1)$$

where m_i is the mass assigned to the i th translational degree-of-freedom of the super-structure; N the total number of translational degrees-of-freedom; and m_B the mass assigned to the isolation system.

Zuñiga and Teran-Gilmore (2012) observe that a decrease in the value of this parameter is reflected in improved acceleration control through the reduction of upper mode effects. Thus, although the mass distribution in the building is usually a given and cannot be changed for design purposes, an acceleration-based format needs to consider this parameter during seismic design.

2.3. Acceleration ratios

In terms of controlling the acceleration demands in a super-structure, it is important to define an acceleration ratio that relates the acceleration demand at the base of the isolation system with that at the roof of the super-structure. With this purpose, Zuñiga and Teran-Gilmore (2012) have defined an acceleration ratio R_n (where n denotes the number of stories in the building) that relates the maximum absolute acceleration demand in the roof with that of the isolation system. Through a nonlinear regression analysis, the following functional form has been assigned to R_n :

$$R_n = a \cdot e^{\left(\frac{b}{\left(\frac{T_B}{T_S} \right) + c} \right)} \quad (2)$$

where a , b and c are regression parameters whose value depends on γ , the number of stories in the building, and the percentage of critical damping under consideration for the base-isolation system (ζ_B). Table 1 summarizes values of the regression parameters for $\gamma = 0.8$ and different levels of damping. Note that a γ of 0.8 for a four-story building implies that the mass of the ground level is fairly equal to that assigned to the rest of its stories.

Table 1. Parameter values for four-story isolated structures ($\gamma = 0.80$)

ζ_B	a	b	c
10%	0.97	0.68	0.38
15%	1.00	0.64	0.42
20%	1.01	0.63	0.51
25%	1.02	0.73	0.79

2.4. Maximum acceleration values

Finally and in terms of an acceleration-based design format, it is necessary to consider that the different types of contents have inherent properties that made them particularly vulnerable (or not), to high acceleration demands. Recent studies have illustrated the high correlation that exists between floor acceleration demands in seismically isolated structures and the damage level suffered by its contents (Kelly 1982, Kelly and Tsai 1985). Within this context, an adequate seismic performance of the contents of an essential facility demands the definition of acceleration thresholds. As a reference and based on the discussion offered by Hamaguchi et al. (2004), a threshold value of $0.2g$ can be considered to provide overall protection to common contents. In the end, the maximum allowable acceleration used for the seismic design of the base-isolated structure should be established according to the type of contents and their dynamic characteristics.

3. DESIGN METHODOLOGY

The design methodology discussed herein is applicable to low-rise buildings that do not exhibit significant torsional or bidirectional effects, and consists of the following steps:

- The contents to be protected are characterized in terms of their dynamic characteristics and damping coefficient. In terms of acceleration-sensitive contents, Hamaguchi et al. (2004) observe that a period range going from 0 to 0.5 sec contemplates the most common contents used in buildings. Based on this and for illustration purposes, the contents will be assigned herein a percentage of critical damping of 2% and a period that is equal or smaller than 0.5 sec.
- An acceleration threshold (A_C) is established to promote an adequate seismic performance of the acceleration-sensitive contents.
- Based on the value of A_C and the dynamic characteristics of the contents and the isolated structure, a threshold is established for the maximum floor acceleration demand in the super-structure (A_e).
- The value of the mass ratio (γ) is estimated, and initial values assigned to the T_B/T_S ratio and the percent of critical damping assigned to the isolation system (ζ_B). As discussed by Zúñiga and Teran (2012), it is recommended that T_B/T_S equals eight.
- The acceleration ratio (R_n) is then established for the values of T_B/T_S , ζ_B and γ under consideration.
- With values of A_e and R_n , it is possible to determine the maximum acceleration demand allowable in the isolation level (A_B).
- As shown in Figure 1, the effective period of the isolation system (T_B) is established through the threshold value of A_B and an elastic pseudo-acceleration response spectrum corresponding to ζ_B .

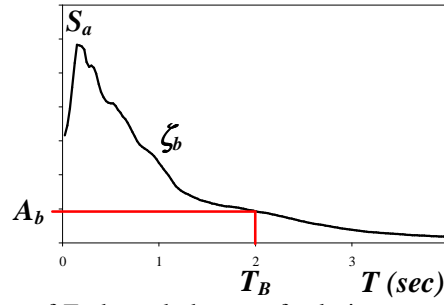


Figure 1. Determination of T_B through the use of a design pseudo-acceleration spectrum

- h) If the value of T_B is deemed acceptable, the methodology proceeds to the next step. If not, a design iteration is carried out.
- i) Once the value of T_B is available, T_S is estimated as:

$$T_S = \frac{T_B}{\frac{T_B}{T_S}} \quad (3)$$

- j) If the value of T_S is deemed acceptable, the methodology proceeds to the next step. If not, a design iteration is carried out.
- k) Once T_S is established, the maximum displacement demand at the isolation level (D_B) can be estimated, as shown in Figure 2, through the use of a displacement response spectrum.

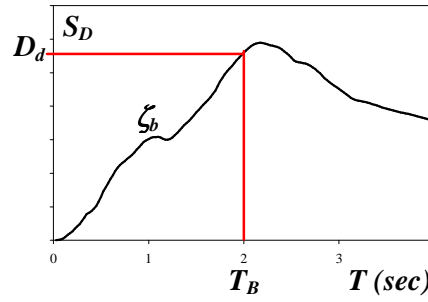


Figure 2. Determination of D_b through the use of a design displacement spectrum

- l) If D_B is deemed acceptable the design of the isolation system is carried out based on the values of T_B and ζ_B . If not, a design iteration is carried out.

4. BUILDING UNDER CONSIDERATION

The structural layout of the super-structure of the four-story building under consideration herein is shown in Figures 3 and 4. Figure 5 shows the arrangement of concentric braces used in both versions of the building to provide it with seismic resistance. In both versions, the frames were designed to essentially resist the gravitational forces of the building.

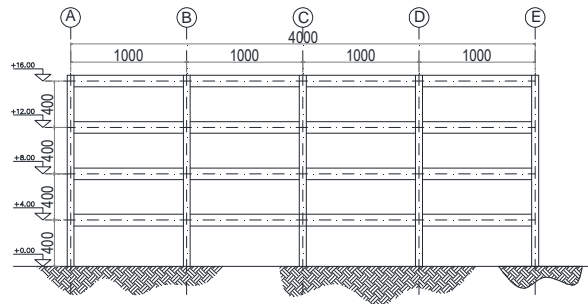


Figure 3. Elevation view of four-story building

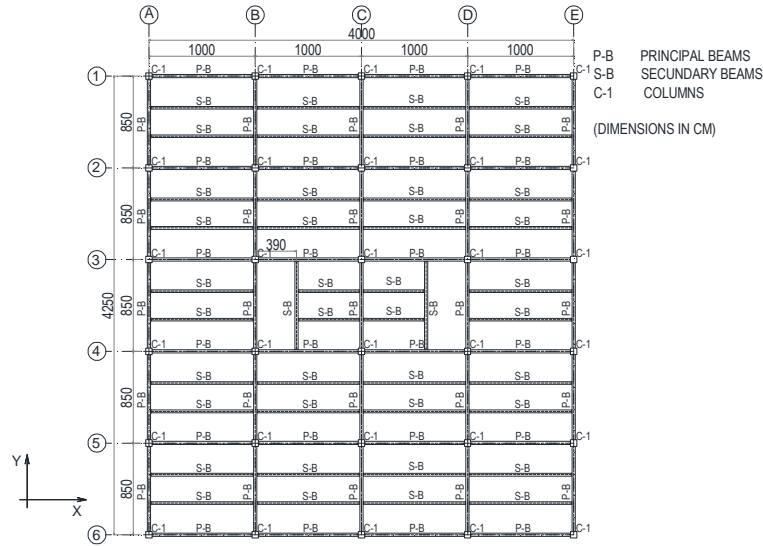


Figure 4. Plan view of four-story building

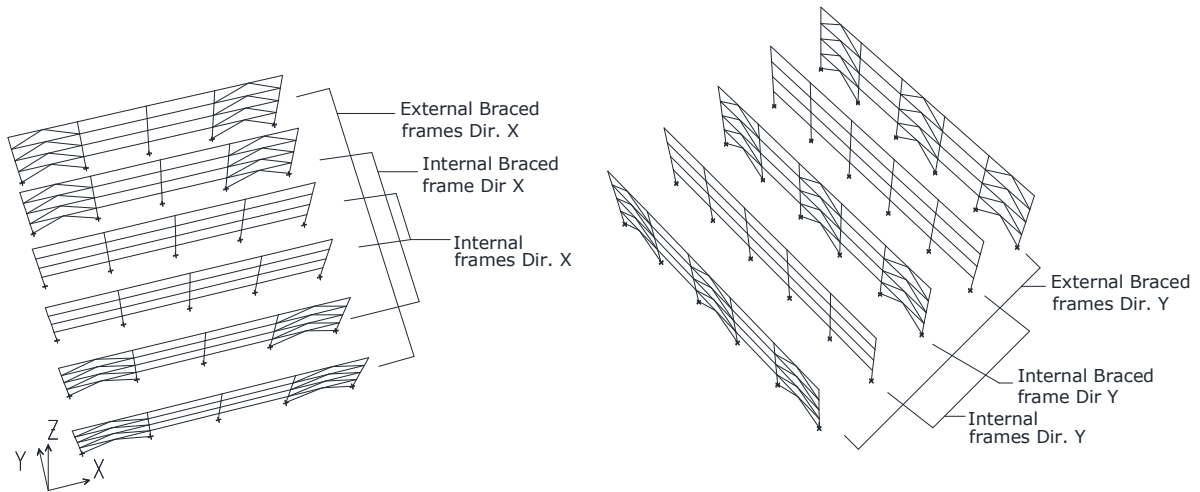


Figure 5. Structural layout of concentric braces used to provide seismic resistance

4.1. Seismically isolated structure

The gravitational reinforced concrete frames, formed by 30×100 cm principal beams, 25×90 cm secondary beams, and 60×60 cm columns; exhibit a fundamental period of vibration of 0.72 seconds. While a concrete with $f'_c = 300 \text{ kg/cm}^2$ was used, the f_y for the reinforcing steel was 4200 kg/cm^2 . Table 2 summarizes the three versions under consideration herein to illustrate the use and reach of the acceleration-based methodology.

Table 2. Versions under consideration for base-isolated building

Version	T_B/T_S	ζ_b	γ	$A_C (\text{cm/s}^2)$	$A_e (\text{cm/s}^2)$
1	6.0	10%	0.8	300	200
2	8.0	10%	0.8	300	200
3	8.0	15%	0.8	150	100

From the response of contents on various base-isolated low-rise building with T_B ranging from two to three seconds, it has been observed that to control the acceleration demands in contents with period of vibration equal or smaller than 0.5 sec, the maximum floor acceleration (A_e) should be controlled within a threshold equal to $A_C/1.5$. Within this context, Table 2 also summarizes the design

acceleration thresholds under consideration. With the values of T_B/T_S , ζ_B and γ corresponding to each version, Equation 2 yields values of R_d and their corresponding A_B thresholds. Table 3 summarizes these values for the three versions under consideration for the base-isolated building.

Table 3. Values of design parameters

Version	R_d	A_B (cm/s ²)	T_B (sec)	T_S (sec)
1	1.08	184	1.98	0.33
2	1.06	189	1.93	0.24
3	1.08	93	2.65	0.33

As shown in Figure 6, with the value of A_B and a design pseudo-acceleration response spectrum it is possible to determine the value of T_B . The design spectra under consideration herein were obtained by establishing the mean spectra for twenty two ground motions recorded at firm soil sites located in the state of Guerrero, Mexico. Note that the percentage of critical damping under consideration in the design spectrum is equal to ζ_B ; and that the values under consideration herein for T_B are the most conservative within the range of possibilities offered by the design spectra. Particularly, any base-isolated structure with a fundamental period of vibration larger than T_B would exhibit base acceleration demands smaller than those implied by the design threshold, and in this sense, the designs presented herein are the most conservative possible in terms of the lateral stiffness of the isolation systems and their corresponding super-structures.

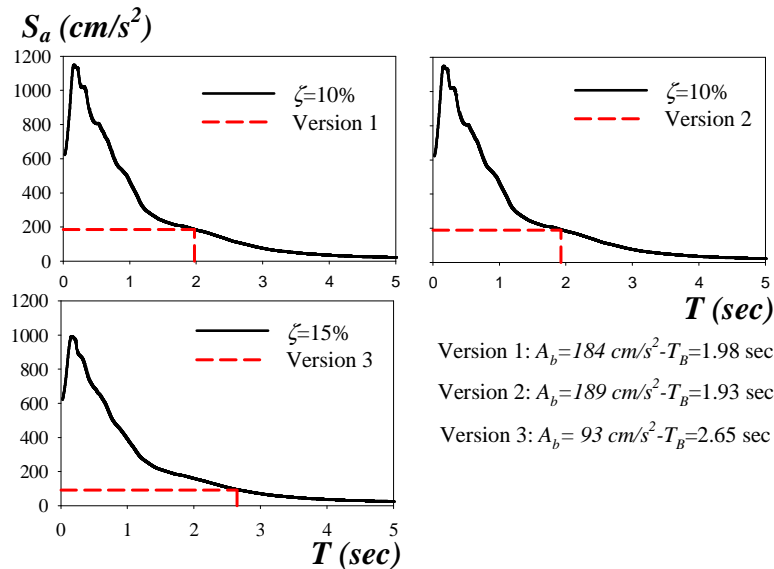


Figure 6. Determination of T_B ; a) Version 1; b) Version 2; and c) Version 3

Table 3 summarizes the T_S values corresponding to all versions. Note that T_S establishes the design requirements in terms of the lateral stiffness to be provided to the super-structure. Particularly, the sizes of the braces are established in such a manner that the fundamental period of vibration of the super-structure on fixed base equals T_S . By considering the structural layout shown in Figure 5, the areas summarized in Table 4 were estimated for each brace oriented in the X direction. While the braces are to be fabricated with steel having f_y of 2530 kg/cm², note that Version 2 demands a much higher area for its braces.

Table 4. Required area for braces of the isolated versions

Story	Area per brace (cm ²)		
	Version 1	Version 2	Version 3
4	67	200	67
3	67	200	67
2	133	400	133
1	133	400	133

Once the global structural properties of the base-isolation systems and their respective super-structures have been obtained, the displacement demands in the isolation systems can be estimated as shown in Figure 7. Because the displacement demands for the three versions are deemed acceptable, the methodology proceeds to the sizing of the rubber bearings. Note that unlike base-isolation systems located in epicentral areas of California and Japan, displacement demands in isolated buildings located in the Mexican Pacific Coast do not represent a limitation in terms of their acceleration-based conception. Once the values of T_B and ζ_B are available, the rubber bearings and complementing viscous dampers can be designed. The plan layout of the building exhibits thirty columns, and a rubber bearing was placed underneath each one. Table 5 summarizes the geometrical characteristics of the different bearings used for each one of the three versions of the isolated building. In terms of their mechanical properties, $G = 71.36 \text{ ton/m}^2$ and $E_c = 184,060 \text{ ton/m}^2$. Although not discussed herein in detail, viscous dampers should be used to increase the percentage of critical damping of the base-isolation system to the value of ζ_B under consideration.

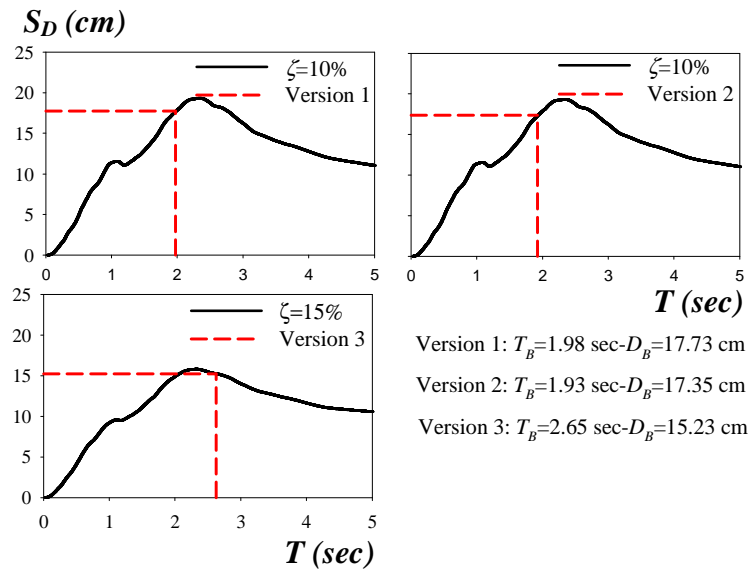


Figure 7. Determination of D_B : a) Version 1; b) Version 2; and c) Version 3

Table 5. Characteristics of the rubber bearings

Location	Number	Height (cm)	Diameter (cm)	Maximum Deformation (cm)
Corner (version 1 and 2)	4	24.0	57.0	36.0
Perimeter (version 1 and 2)	14	30.0	65.0	41.0
Internal (version 1 and 2)	12	45.5	80.0	46.0
Corner (version 3)	4	38.0	54.0	28.0
Perimeter (version 3)	14	43.0	65.0	37.0
Internal (version 3)	12	53.5	80.0	42.0

Lumped plasticity nonlinear models were developed for the gravitational frames. In terms of flexural stiffness the beams were assigned half their gross moment of inertia to account for possible cracking. In the case of the columns, they were assigned their full moment of inertia. The braces were assigned an axial stiffness that was 50% higher than that implied by the area of braces contemplated in Table 4 to account for the zones of larger axial stiffness located at their ends.

Figure 8 shows the mean acceleration floor spectra at the roof for the three versions of the isolated building. The floor spectra were established from the time-histories of absolute acceleration corresponding to the twenty two motions under consideration and a percentage of critical damping of 2%. While the spectral ordinates at the origin closely match the values of A_e under consideration for design purposes, within the period range under consideration (0 to 0.5 sec), the absolute acceleration demands are controlled in a reasonable manner within their design thresholds. Figures 8a and 8c show acceleration spikes that exceed the design thresholds at a period corresponding to the second mode of vibration of the base-isolated buildings. At this point, the structural engineer can decide if the

preliminary design merits adjustments to reach the final design. On one hand, T_B/T_S can be increased by increasing the lateral stiffness of the super-structure or decreasing that of the isolation system. On the other hand, the designer should consider that a highly concentrated spike on an acceleration floor spectrum usually over-estimates the acceleration demands on contents having a period close to that corresponding to one of the modes of vibration of the structure (Villaverde 2006), and that in cases like this, it may not be worth modifying the preliminary design.

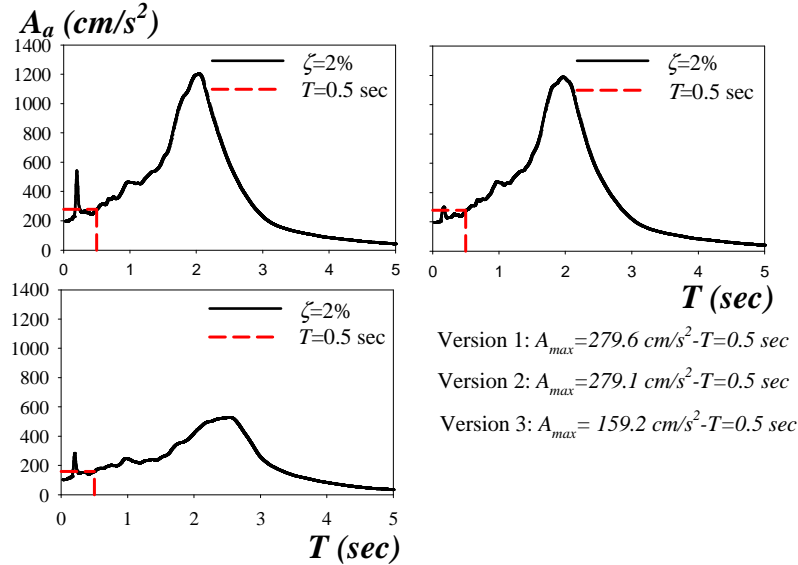


Figure 8. Roof acceleration spectra; a) Version 1; b) Version 2; and c) Version 3

It is important to mention that the mean maximum interstory drift index demands for all three versions of the isolated building is equal or smaller than 0.002. Under these circumstances, the buildings are far from exhibiting structural and non-structural damage, and given that they are capable of adequately controlling their acceleration demands, are in an excellent position to satisfy the operational performance level after the occurrence of the design ground motion.

4.2. Fixed base structure

The fixed-base version was considered to be located at the same site. Unlike the isolated versions, the braces in the fixed-base version need to provide lateral resistance to the building through their nonlinear behaviour. Buckling-restrained braces with f_y of 2530 kg/cm² were used with this purpose, and their areas estimated according to the strength-based format discussed by Teran and Ruiz (2010). In terms of the gravitational frames, while the sizes of the columns had to be increased from 60 × 60 cm to 80 × 80 cm so that they could accommodate the axial forces induced to them by the braces, the dimensions of the principal and secondary beams were kept equal with respect to those used in the base-isolated versions. Table 6 summarizes the area of braces required for the fixed-base version of the building. The fundamental period of vibration of the braced frames was 0.38 seconds. As may be concluded by comparing the areas included in Tables 4 and 6, the fixed-base version requires less area of braces than its base-isolated counterparts. The capacity curve shown in Figure 9a was derived from a nonlinear static analysis. A roof displacement demand of 10 cm closely corresponds to the lateral displacement demand at which the frames reach a maximum interstory drift index demand of 0.01.

Nonlinear time-history analyses were performed in order to assess the seismic performance of the fixed-base version of the building. Similar modelling considerations as those discussed before were used to prepare a nonlinear model of this version, and the same twenty ground motions were used. In terms of the seismic performance of the contents, Figure 9b shows the mean roof absolute acceleration spectrum. Within the period range under consideration for the contents, very high acceleration demands are expected, particularly at periods that match the fundamental and second periods of

vibration of the braced frames. In general, it can be said that the acceleration demands in the fixed-base building exceed in more than ten times those estimated for its base-isolated counterparts. Under these circumstances, it is difficult to think that the acceleration-sensitive contents of the hospital facility may remain operational after the ground motion.

Table 6. Required area for braces of fixed-base version

Story	Area per brace (cm ²)
4	49.6
3	76.5
2	91.4
1	91.4

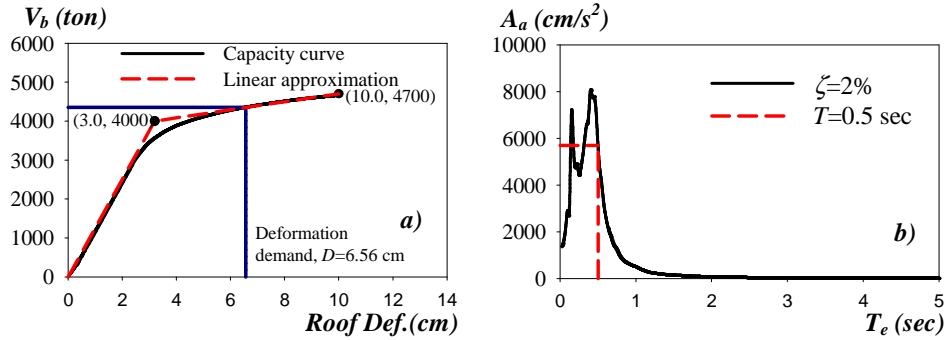


Figure 9. Fixed-base version: a) Capacity curve; and b) Acceleration floor response spectrum at roof

While a mean roof displacement demand of 6.56 cm was estimated from the nonlinear dynamic analyses, the mean maximum interstory drift index demand reached a value close to 0.005. For this roof displacement and drift demands, Figure 10 indicates with coloured circles the structural elements that develop nonlinear behaviour in the X direction. While the braces undergo a maximum ductility demand close to two, the plastic rotation demands in the beams and columns of the frames are very low. From their global and local deformation demands, it can be said that the braced frames are capable of achieving the immediate occupancy performance level from structural and nonstructural perspectives.

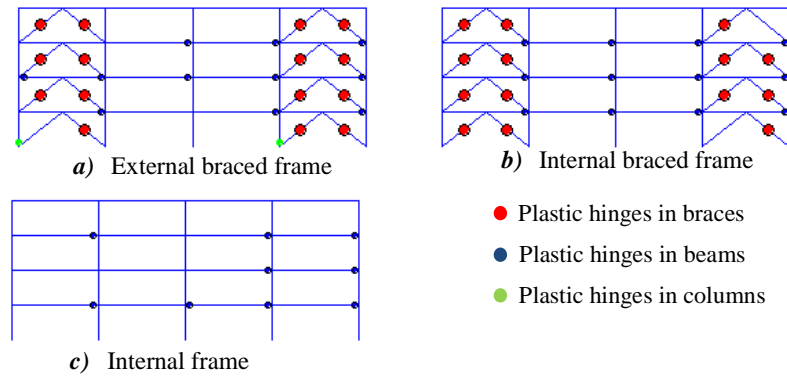


Figure 10. Location of nonlinear demands on the structural system of the fixed-base version

5. DISCUSSION

The results presented herein suggest that it is possible to formulate an acceleration-based approach to the seismic design of base-isolated structures. Within this context, the most important structural properties of the base-isolated structure are the lateral stiffness of the isolation system and that of its super-structure. The acceleration-based methodology requires first to establish acceleration thresholds in the super-structure as a function of the type of contents that require protection, and their level of

acceptable damage. Then the implicit use of an equivalent single-degree-of-freedom through the use of pseudo-acceleration and displacement design spectra allows for the determination of the global structural properties of the isolation system and the super-structure. Once the global characteristics of the structure are defined, the methodology proceeds to the sizing and design of the rubber bearings and of the structural elements of the super-structure. Finally, a series of dynamic nonlinear analyses should be performed to assess the seismic performance of the contents of the building and if needed, to provide information to adjust the preliminary design. The value assigned to the T_B/T_S ratio is fundamental in terms of making possible an effective control of the acceleration demands in common contents. Particularly, it is convenient to consider values of T_B/T_S equal or larger than eight. This may imply the use of robust bracing systems to provide sufficient lateral stiffness to the super-structure.

In terms of the fixed-base version, the use of a buckling-restrained system has given place to a building that has the capability of achieving immediate occupancy in terms of its structural and nonstructural systems, but that is likely to exhibit extensive damage in its contents.

In terms of comparing the weight of different versions of the four-story building, the weight of the braces and their connections for Version 3 and the fixed-base version are 160 and 120 tons, respectively. Thus, forty more tons of structural steel has to be invested in the bracing system of the base-isolated version. In terms of the columns, the weight of concrete is 380 and 680 tons, respectively; and that of their reinforcing longitudinal steel, 24 and 60 tons, respectively. Thus, 300 more tons of concrete and 36 more tons of rebar have to be invested in the columns of the fixed-base version.

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