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ANALYTICAL STUDY ON THE SEISMIC ISOLATION OF TWO IRREGULAR BUILDINGS AT THE MEXICAN PACIFIC COAST

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ABSTRACT

An analytical study on the application of different base isolation systems for original design of two typical building structures of the Mexican Pacific Coast is presented. The subject hypothetical buildings are located on hard soil conditions at the touristic port of Acapulco. Typical accelerograms for the Mexican subduction zone recorded during recent earthquakes were used for 3-D time-history analyses. Bidirectional input was used for the time-history analyses. A set of eight stations was selected, three for the September 19, 1985 earthquake (M_S=8.1), three for the April 25, 1989 earthquake (M_S=6.9) and two for the May 30, 1990 earthquake (M_S=6.1). The studied structural systems were an irregular low-rise and a mid-rise hotel buildings. The studied base isolation systems were lead-rubber bearings (LRB) and steel-hysteretic dampers (SHD). The superstructures were designed: a) according to the seismic provisions of the building code of Guerrero state (RCGS-90) for the fixed-base condition and, b) according to an elastic design based upon a 3-D lateral force distribution consistent with dominant mode shapes for the isolated structure to yield the peak dynamic base shear transmitted by the isolation system. Material volumes for the superstructure were estimated for both the fixed-base code designs and the base-isolation designs. Important savings on the volume of concrete and steel reinforcement can be attained for the base isolated designs with respect to their counterpart fixed-base designs. Dynamic responses for the isolated structures compare favorably against those for the fixed-base structures. The study confirms many findings published in the literature regarding the effectiveness of base isolation and the effect of torsional responses. However, the study also shows that the dynamic stability of isolators is not always achieved using rational design procedures. The dynamic stability and design of base isolators can be controlled by acceleration records associated to moderate earthquakes when these records are near the fault plane and by torsional responses.

KEYWORDS

Base isolation, nonlinear dynamic analysis, Mexican Pacific Coast, irregular buildings, dynamic torsion

INTRODUCTION

Base isolation technology has emerged as a structural option in the last decade thanks to the great effort done by researchers and practicing engineers worldwide. Extensive experimental and analytical research has been devoted to different base isolators and base-isolated structures, particularly in the last twenty five years. As a result of the extensive research done in this time frame, the seismic isolation of building structures and bridges is starting to become an accepted structural solution in earthquake-prone countries, specially in the United States, New Zealand, Japan and Italy. At the end of 1992, there were 68 base-isolated bridges in Italy and at least three buildings were under construction. As of December of 1990, there were 45 seismically isolated buildings and bridges in New Zealand. As of February of 1993, there were 47 seismic isolated buildings in the United States and 26 projects under way, 14 for seismic retrofit and 12 for original design. The number of base-isolated structures in Japan is more than 70 nowadays. The growing interest on the use of base isolation in the United States seems to be related to the publication of seismic design provisions for base-isolated structures in American codes, such as the American Association of State Highway and Transportation Officials code of 1990 (AASHTO) and the Uniform Building Code (UBC) of 1991.

The Mexican experience on base isolation is modest compared with the experiences in the United States, Japan, New Zealand or Italy. To the authors' knowledge, there are only three base-isolated structures in Mexico. The first two structures are a four-story school building and a church isolated with a sliding isolation device composed of steel marbles, which was developed by González-Flores in the 1970's. The remaining isolated structure is the press machine for the Reforma newspaper, using a seismic isolation system based on pendular action developed by Garza-Tamez and tested at the University of Illinois (Foutch et al, 1993). These structures are found in Mexico City in near hard soil conditions. Although base isolation is attracting the interest of many Mexican researchers and practicing engineers, its development has been slow for the following reasons. The soft soil conditions of Mexico City's lake-bed zone, with dominant site periods that vary from 1.0s to 3.5s, make base-isolation unattractive, particularly because important differential soil settlements are also common in this region. In addition, the pseudo-accelerations for the design spectra for the hard soil zones in Mexico City are low because of the ground motions associated to the design earthquake in these regions. A conventional design procedure for building structures is economical in these zones, thus, base isolation seems to offer no substantial advantages. Finally, investors nationwide are not willing to spend their money on a technology that has not been used in Mexico before.

It is true that Mexico City is not a natural market for seismic isolation, but Mexico City is not the only city in Mexico affected by the earthquake hazard. In fact, most of the Mexican Pacific Coast is classified as region of high seismic risk. Many touristic spots, industrial and essential facilities, which are important for the Mexican economy, are found in this region. The soil conditions and earthquake motions for the Mexican Pacific Coast, particularly the coast of Colima, Guerrero and Michoacán states, lead one to believe that base isolation is attractive both for original design and retrofit. In addition, the coast of the Gulf of Mexico is a region of moderate seismic risk as is affected by subduction earthquakes from the Caribbean plate. Base isolation is also attractive for this region, where many industrial facilities for the oil and chemical industry are located. An analytical research project was started to assess the advantages of base isolation for original design and retrofit of structures in regions of high seismic risk in Mexico (Tena et al, 1995). The project is assessing the effectiveness of leading commercial base isolators in reducing the seismic response of typical building structures of the Mexican Pacific Coast when subjected to ground motions recorded during moderate and strong earthquakes in this region. The studies done on two buildings are summarized in following sections.

SUBJECT BUILDINGS

Hotel Building H1

Hotel H1 is a typical architectural project for a five-star hotel for the touristic ports of Mexico. The hotel consists of three six-story buildings properly separated among themselves. Two twin, L-shaped buildings with irregularities both in plan and elevation are designed for the guest rooms and are the buildings under study. The typical plan view and overall dimensions for these buildings are shown in Fig. 1. The structural system for lateral loading is an RC dual system composed of ordinary moment resisting frames (OMRF) and shear walls. In the longitudinal direction, the typical bay width is 8.0 m, whereas in the transverse direction the central bay is 6.5 m wide. The typical story height is 4.0 m. The exterior bays in the transverse direction changes their dimensions because of the architectural design (Fig. 3). Because of its form, the building is prone to significant torsional response. Two design strategies were undertaken for this building: a) a conventional design according to the seismic code of Guerrero (RCGS-90, 1990) and, b) a base-isolation design based upon 3-D nonlinear dynamic analyses for the isolators and an equivalent 3-D static analysis for the structure. The design process will be described in detail for each design strategy in following sections.

Hotel Building H2

Hotel H2 is based on a project for a five-star hotel complex for the touristic port of Manzanillo. The hotel consists of seventeen fourteen-story buildings. A plan view and overall dimensions for the first eight stories of a typical building of the hotel are depicted in Fig. 2. The building consists of seven bays 8.20 m wide in the longitudinal direction and one 9.05 m bay in the transverse direction, with exterior 2.05 m and 2.50 m alleys for room balconies and access to the rooms. The typical story height is 3.50 m but the first floor, which is 5.0 m tall. The building is irregular in elevation, with a stair-like setback that reduces one bay per story in the longitudinal direction from the eight to the fourteen stories (Fig. 3). This setback introduces dynamic torsional flexibility in the upper stories. The structural system for lateral loading is an RC dual system composed of ordinary moment resisting frames and shear walls.

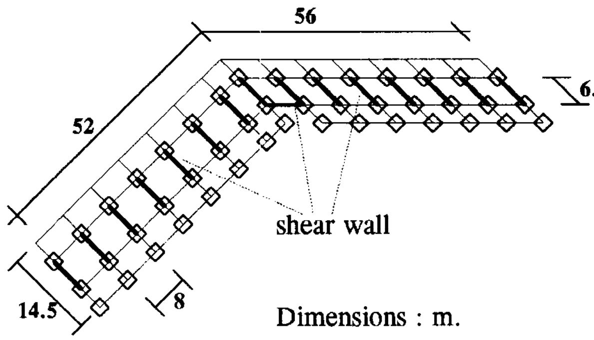

Fig. 1. Plan view for main building of hotel H1

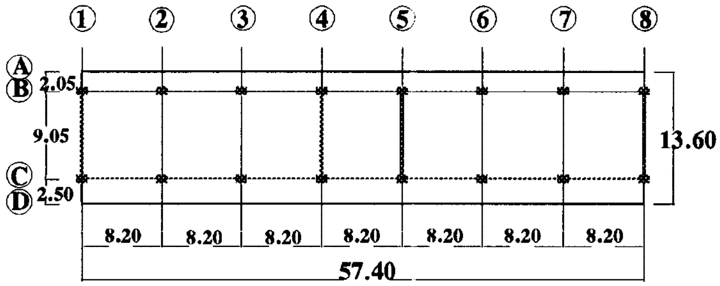

Fig. 2. Plan view for hotel H2 (dimensions in m.)

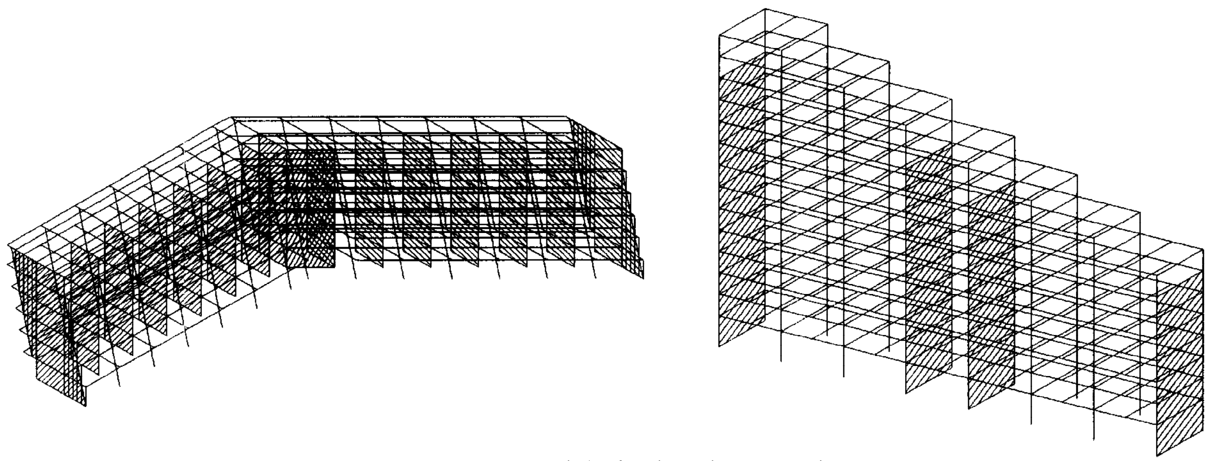

Fig. 3. ETABS models for hotels H1 and H2

ACCELERATION RECORDS

Typical accelerograms for the Mexican subduction zone recorded during recent earthquakes were selected for the present study. A set of eight stations was selected, three for the M_s=8.1, September 19, 1985 earthquake (UNIO, CALE and PAPN), three for the M_s=6.9, April 25, 1989 earthquake (VIGA, MSAS and SMR2) and two for the M_s=6.1, May 30, 1990 earthquake (LLAV and PARS). The selected acceleration records for the two horizontal components for the 1985, 1989 and 1990 earthquakes and their associated acceleration response spectra for 5% viscous damping are depicted elsewhere (Tena et al, 1995). For space limitations, only a few records are depicted in Fig. 4. The 1985 records have longer strong-phase durations, however, the highest peak ground accelerations and acceleration spectral ordinates are associated to records for moderate earthquakes with reduced strong motion durations (stations VIGA and LLAV). All stations are located in Guerrero state. Station SMR2 for the 1989 earthquake has a strong pulse associated to an intermediate period (Fig. 4). This peculiarity is because station SMR2 is an epicentral record for the 1989 event.

CONVENTIONAL DESIGN

Buildings H1 and H2 were designed as conventional structures according to the provisions for the seismic code of Guerrero state (RCGS-90. 1990). According to the RCGS-90, in Guerrero state there are two seismic zones (C and D) and three soil conditions (I for firm soils, II for intermediate or transition soils and III for soft soils). The elastic acceleration design spectra for the different geotechnical zones for Guerrero state are depicted in Fig. 5. Acapulco is classified to be zone D according to RCGS-90. Subject buildings were designed for zone D-I. The design spectrum was reduced with the response modification factors (Q) for global ductility as outlined in RCGS-90. The response modification factor for RC dual systems allowed by RCGS-90 is Q=1.6 for buildings that do not comply with the regularity conditions of the code, Then, Q=1.6 was used for the design of H1 and H2. Buildings H1 and H2 were designed to comply with the requirements of RCGS-90 using 3-D response spectra analyses with ETABS. The assumed material properties for the concrete were f'_c=350 kg/cm^2 for hotel H1 and f'_c=250 kg/cm^2 for hotel H2. Young moduli was taken as E_c=14000 $(f'_c)^{1/2}$. The yield strength of reinforcement steel was f_y= 4200 kg/cm^2. The ETABS models for the buildings under study are depicted in Fig. 3. The dynamic characteristics of the final fixed-base designs are summarized in Table 1. It can be observed the strong torsional coupling for building H1. In contrast, torsional coupling is moderate for building H2 and only for the transverse direction because of the setback (Fig. 3).

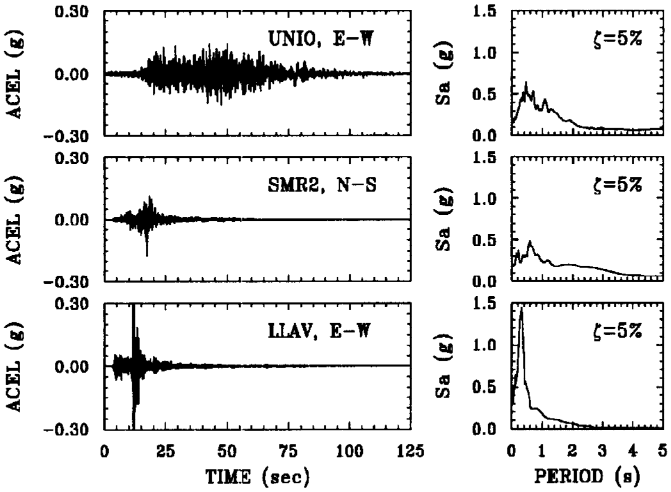

Fig. 4. Selected acceleration records

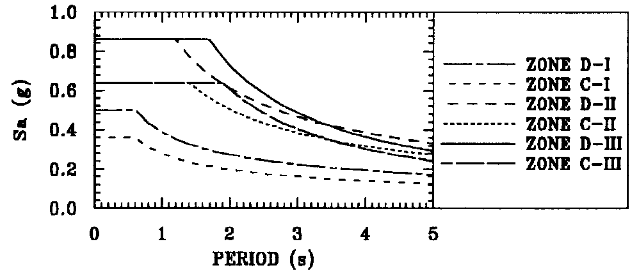

Fig. 5 Elastic design spectra for RCSG-90 code

Table 1. Dynamic characteristics for the fixed-base models under study

Bldg.	Mode	T (s)	Modal Mass (%)			Bldg	Mode	T (s)	Modal Mass (%)		
			L	T	R				L	T	R
H1	1	0.389	52.53	18.06	4.84	H2	1	1.258	78.18	0.00	0.00
	2	0.239	28.05	39.49	7.86		2	0.751	0.00	43.21	7.23
	3	0.124	9.52	4.68	1.00		3	0.468	0.00	24.92	37.02

Hotel Building H1

The traditional design for hotel H1 yields rectangular columns whose cross sections measure 90 x 125 cm and 100 x 130 cm for the first two stories, 65 x 110 cm in the third and fourth stories and 60 x 90 cm in the fifth and sixth stories. Interior, boundary and cantilever beams are 65 x 135 cm in all stories. The thickness for the two outer shear walls is 50 cm whereas for the remaining interior walls is 30 cm. Longitudinal reinforcement steel ratios vary from ρ=0.010 to ρ=0.0353 for the columns. Longitudinal reinforcement ratios for beams vary from ρ=0.0037 to ρ=0.0111. Shear reinforcement ratios (volume of confining stirrups over volume of confined concrete) at the ends of the members vary from ρ_{sh}=0.0023 to ρ_{sh}=0.0042 for the columns and from ρ_{sh}=0.0021 to ρ_{sh}=0.0047 for the beams. Shear reinforcement ratios for the walls vary from ρ=0.0049 to ρ=0.0079 for both the horizontal and vertical directions.

Hotel Building H2

The RCGS-90 design for hotel H2 yield almost square columns whose cross sections measure 140 x 150 cm for the first two stories, 140 x 150 cm and 105 x 110 cm from the third to eight stories, 105 x 110 cm from the ninth to eleventh stories and 90 x 100 cm for the top three stories. Interior beams are 40 x 100 cm in all stories and exterior beams are 60 x 110 and 40 x 100 cm from the first to the eight floor and 40 x 110 and 40 x 80 cm from the ninth floor to the roof. The thickness for the shear walls is 50 cm for stories 1 to 3, 40 cm for stories 4 and 5, 35 cm for stories 6 and 7, 25 cm for stories 8 and 9 and 20 cm for stories 10 to 12. Longitudinal reinforcement steel ratios for the columns are around ρ=0.007, slightly higher than the minimum required by the RCGS-90 code (ρ=0.005) for OMRFs. Longitudinal reinforcement ratios for beams vary from ρ=0.0072 to ρ=0.0100. Shear reinforcement ratios at the ends of the members vary from ρ_{sh}=0.0082 to ρ_{sh}=0.0123 for the columns and from ρ_{sh}=0.0030 to ρ_{sh}=0.0117 for the beams. Shear reinforcement ratios for the walls vary from ρ=0.0025 to ρ=0.0049 for both the horizontal and vertical directions.

SEISMIC ISOLATION DESIGN: BASE ISOLATORS

Lead-rubber bearings (LRB) and steel hysteretic dampers (SHD) were considered. The stiffness and strength properties used for the LRB and SHD isolators were based upon recommendations available in the literature (Skinner et al, 1993) and are presented in detail in Tena et al (1995). There are not code design procedures for base-isolated structures available in Mexico yet, so the design criteria were based upon recommendations available in the literature. Given the irregularities of the selected buildings, with important torsional responses, the design of the isolators was based upon 3-D time-history analyses using bidirectional input. Two horizontal components for the recorded ground motions during the strong M_S=8.1, 1985 Michoacán Earthquake (stations UNIO, CALE and PAPN) were used. The design of the different base isolators was done using the 3D-Basis computer program (Nagarajaiah et al, 1991). The first six to twelve mode shapes for the fixed-base structural models (eg, Table 1), were considered for the preliminary design process. The final design was done using the mode shapes for the fixed-base model of the base-isolated designed superstructure.

Hotel Building H1

For hotel H1, the post yield stiffness for the LRB isolators was taken as ten percent of their initial elastic stiffness. Material properties for the rubber bearing and the lead core were based upon the data reported by Skinner et al (1993). All LRB isolators were of circular cross section. One LRB isolator is placed in every column line. The final design when V_{yield}=0.10W (W is the total weight of the superstructure) was a total of 45 circular LRB isolators 65 cm in diameter and 40 cm in height, with a lead core 6 cm in diameter. The maximum allowable isolator displacements for dynamic stability are X_M=Y_M=21.7 cm. The maximum effective isolated natural period was T_I=2.36s.

Table 2. Peak dynamic displacements and base shear vs allowable limits for the LRB base isolation project for Hotel H1

Event	Critical Station	Max relative roof displacements wrt the isolators (cm)		Peak LRB isolator displacements		Base Shears (Ton)		$\frac{V_x}{W}$	$\frac{V_y}{W}$
		Δx_{max}	Δy_{max}	X/X_M	Y/Y_M	V_x	V_y		
09/19/85	UNIO	0.23	0.16	0.450	0.841	910.8	1152.9	0.073	0.093
04/25/89	SMR2	0.18	0.12	0.841	1.740*	800.6	942.9	0.064	0.076
05/31/90	LLAV	0.32	0.21	0.263	0.353	909.0	973.6	0.073	0.078

Peak dynamic responses for this isolation design when subjected to the critical acceleration records for the 1985, 1989 and 1990 Mexican earthquakes are summarized in Table 2. From now on, the longitudinal direction is identified as "x" direction and the transverse direction as "y" direction. It can be observed that although the proposed design performs very well when subjected to most of the considered ground motions as relative peak roof displacements and base shears are low, dynamic instability is detected for the LRB when the isolated H1 building is subjected to the recorded accelerograms for SMR2 station during the M_S=6.9, 1989 earthquake (marked with an asterisk). The instability is caused by the strong low frequency pulse contained in this record (Fig. 4) and the strong torsional response of the isolators. SMR2 is an epicentral record for the 1989 earthquake. Then, it is clear that designing base-isolators based upon accelerograms recorded during strong events only may not be safe enough. The 1994 UBC code requires time-history analyses with at least three appropriate pairs of horizontal time-history components and implicitly considers the possibility that near-fault time-history records may control the design of base isolated structures (Section 2375(d)2). On the other hand, the isolators work effectively for the rest of the moderate earthquakes ground motions. The torsional response of the LRB isolators is schematically depicted in Fig. 6 when subjected to the critical station SMR2. The torsional response diminishes the effectiveness of the isolation scheme, as some isolators are subjected to reduced deformation and strength demands whereas some others are subjected to high demands. This can also be precluded from Table 2, as the maximum dynamic base shear transmitted to the structure is lower than the theoretical yielding base shear for the whole isolation scheme, evidencing that some isolators yield while others respond in the elastic range. It can also be observed from Fig. 6 that although most of the isolators displace unevenly but within allowable limits, the last nine isolators from the right margin of the figure considerably surpass the allowable displacement for dynamic stability. Clearly, a more refined design method is needed for isolated structures with strong torsional responses to prevent or diminish the torsional response on the isolator system.

A base isolation design with SHD was also done. A torsional-beam damper with transverse loading arms (Type E damper, Skinner et al, 1993) of rectangular cross section was selected. Material properties were based upon those reported by Skinner et al (1993). The maximum effective isolated natural period was T_I=3.23s. Details for the design of the isolators are found in Tena et al (1995). Peak dynamic responses for the SHD isolation design when subjected to the acceleration records for the 1985, 1989 and 1990 Mexican earthquakes were improved under this design. Peak roof displacements and base shears are low and the dampers are dynamically stable. Higher deformation demands for the SHD isolators are still observed by the epicentral record SMR2. Torsional response is still important. The peak roof displacements for this alternative are higher than those computed for LRB, however, they are still small.

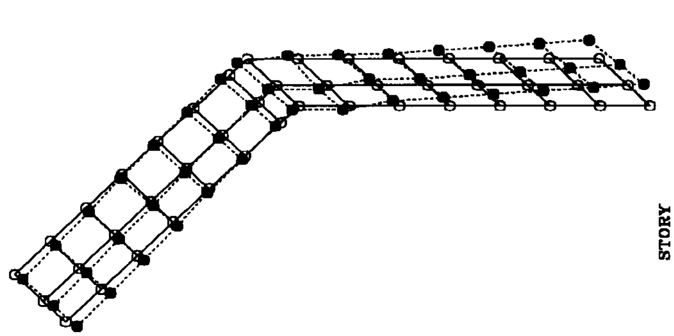

Fig. 6. Torsional response for LRB isolators, hotel H1

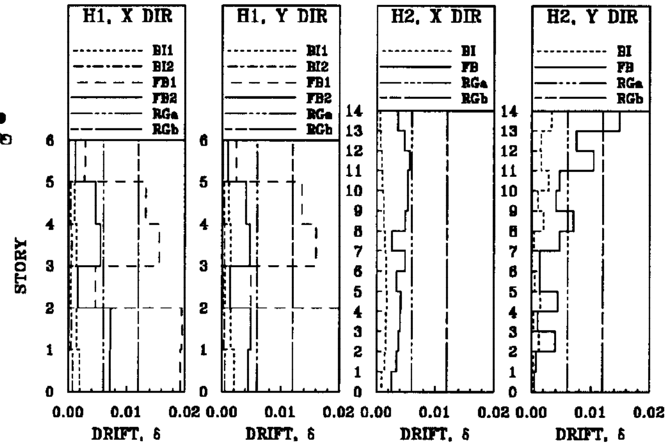

Fig. 7 Maximum interstory drift angle envelopes

Hotel Building H2

For hotel H2, LRB isolators were designed for a global yield strength of 0.10W. The final design was a total of 16 circular LRB isolators 65 cm in diameter and 50 cm in height, with a lead core 22.5 cm in diameter. The maximum allowable isolator displacements for dynamic stability are X_M=Y_M=21.7 cm. The maximum effective isolated natural period was T_I=2.72s. Similarly to what was observed in hotel H1, the proposed design performs very well when subjected to most of the considered ground motions, but dynamic instability is detected for the LRB when the building is subjected to the SMR2 records (Table 3), regardless the fact that torsional coupling is smaller. A reduced torsional response was observed in the isolators in the transverse direction (y), mostly influenced by the torsional eccentricity due to the superstructure. A base isolation design with SHD was also done. Torsional-beam dampers with transverse loading arms (Type E damper) of rectangular cross section were selected. The design was done for a yielding force V_{yield}=0.10W and consisted of a total of 16 rectangular Type E SHD isolators in each direction. The post yield stiffness for these SHD isolators was 6% of their initial elastic stiffness. The maximum effective isolated natural period was T_I=3.88s. Details on the design can be consulted elsewhere (Tena et al, 1995). The proposed design performs well when subjected to all the selected ground motions. Peak roof displacements and base shears are reasonably low and the dampers are dynamically stable. Higher deformation demands for the SHD isolators are still observed by the epicentral record SMR2. Torsional response is less important than for hotel H1. As for hotel H1, peak roof displacements for the SHD alternative are higher than those for the LRB option.

Table 3 Peak dynamic displacements and base shear vs allowable limits for the LRB base isolation project for Hotel H2

Event	Critical Station	Max relative roof displacements wrt the isolators (cm)		Peak LRB isolator displacements		Base Shears (Ton)		$\frac{V_x}{W}$	$\frac{V_y}{W}$
		Δx_{max}	Δy_{max}	X/X_M	Y/Y_M	V_x	V_y		
09/19/85	UNIO	3.6	2.9	0.486	0.953	649.7	898.4	0.084	0.116
04/25/89	SMR2	0.6	0.4	1.005*	1.212*	726.0	772.0	0.094	0.099
05/31/90	LLAV	2.9	1.9	0.156	0.276	277.5	419.2	0.036	0.054

SEISMIC ISOLATION DESIGN: SUPERSTRUCTURE

The superstructure were also designed as base-isolated structures. Since there is no basis to establish response modification factors for global ductility for base isolated structures in Mexican codes, an elastic design based upon a 3-D lateral force distribution consistent with dominant mode shapes for the isolated structure and the peak dynamic base shear transmitted by the isolation system was selected. This procedure is conservative in nature and will insure the superstructure to remain elastic. Further research is needed to define suitable response modification factors for the design philosophy of Mexican codes. Subject buildings were designed to comply with the material strength and detail requirements of RCGS-90 using an equivalent 3-D static analysis with ETABS. The assumed material properties were the same outlined in previous sections.

Hotel Building H1

The base-isolation design for hotel H1 was based on the peak dynamic base shear obtained for station CALE for the SHD isolation. Cross sections for rectangular columns measure 90 x 110 cm and 90 x 130 cm for the first two stories, 60 x 90 cm in the third and fourth stories and 60 x 60 cm in the fifth and sixth stories. Interior, boundary and cantilever beams are 60 x 130 cm in all stories. The thickness for the two outer shear walls is 40 cm whereas for the remaining interior walls is 30 cm. Longitudinal reinforcement steel ratios vary from $\rho=0.0100$ to $\rho=0.0379$ for the columns. Longitudinal reinforcement ratios for beams vary from $\rho=0.0040$ to $\rho=0.0106$. Shear reinforcement ratios at the ends of the members vary from $\rho_{sh}=0.0020$ to $\rho_{sh}=0.0050$ for the columns and from $\rho_{sh}=0.0021$ to $\rho_{sh}=0.0051$ for the beams. Shear reinforcement ratios for the walls vary from $\rho=0.0050$ to $\rho=0.0067$ for both the horizontal and vertical directions.

Hotel Building H2

The base-isolation design for hotel H2 was based on the peak dynamic base shear obtained for station UNIO for the SHD isolation. The design yields columns whose cross sections measure 140 x 140 cm for the first two stories, 140 x 140 cm and 105 x 110 cm from the third to eight stories, 105 x 110 cm from the ninth to eleventh stories and 90 x 100 cm for the top three stories. Interior beams are 40 x 100 cm in all stories and exterior beams are 40 x 100 and 40 x 90 cm from the first to the eight floor and 40 x 100 and 40 x 80 cm from the ninth floor to the roof. The thickness for the shear walls is 40 cm fors stories 1 to 3, 35 cm for stories 4 and 5, 30 cm for stories 6 and 7, 25 cm for stories 8 and 9 and 20 cm for stories 10 to 12. Longitudinal reinforcement steel ratios for the columns vary from $\rho=0.005$ to $\rho=0.006$. Longitudinal reinforcement ratios for beams vary from $\rho=0.0045$ to $\rho=0.0124$. Shear reinforcement ratios at the ends of the members vary from $\rho_{sh}=0.0082$ to $\rho_{sh}=0.0123$ for the columns and from $\rho_{sh}=0.0024$ to $\rho_{sh}=0.0078$ for the beams. Shear reinforcement ratios for the walls vary from $\rho=0.0025$ to $\rho=0.0047$ for both the horizontal and vertical directions.

BASE ISOLATION DESIGN VS TRADITIONAL DESIGN

For all buildings under study, the dynamic response of the superstructure was considerably reduced for the base-isolated projects with respect to their counterpart rigid-base designs when dynamic stability of the isolators was achieved. Substantial reductions in story displacements, shears and accelerations were observed not only for response maxima but also for all the duration of response time histories (Tena et al, 1995). Peak dynamic interstory drift angles for the fixed-base and base-isolated models are compared in Fig. 7. For hotel H1, drift angles correspond to CALE station and the SHD isolation project, whereas UNIO station and the SHD isolation project were selected for hotel H2. In this figure, BI stands for the base-isolated results whereas FB for the fixed-base results, and the indices 1 and 2 for hotel H1 identify the "minimum" and "maximum" peak responses of the superstructure because of the torsional response. Also, RGa represents the limiting drift angle of RCGS-90 for a structure that could have nonstructural elements no properly separated from the structural systems, whereas RGb is the maximum drift angle allowed by this code when nonstructural elements are properly separated from the structural system. The strong torsional response detected for the isolators of hotel H1 is shared by the superstructure, as it can be observed from the interstory drift curves presented in Fig. 7. Nevertheless, the peak dynamic interstory drifts are considerably reduced for the base isolated option, and an elastic response for the superstructure is warranted. On the other hand, interstory drift angles for the fixed-base option suggest strong nonlinear response and the possibility of severe structural damage. The peak dynamic interstory drift for the fixed-base model was $\delta_y=0.022$ at the first story for the exterior column line (FB2). Similarly, the maximum interstory drift angles for the base-isolated project of hotel H2 suggest linear elastic response for the superstructure whereas the fixed base response could be highly nonlinear, specially for the top stories in the transverse (Y) direction.

Initial volumes of materials needed to construct the buildings under study as base-isolated or traditional designs were also computed. The savings on the volume of concrete and reinforcement steel needed to build the base-isolated project were, respectively, 16.2% and 12.0% for hotel H1 and 20.3% and 9.0% for hotel H2. These savings could be increased if a) an optimum design criterion for each cross section would have been followed, b) a suitable response modification factor would have taken for the base-isolated projects and/or c) the buildings would have been located on different soil conditions (Fig. 5). Perhaps the savings on conventional materials would not be enough to make the initial cost of a base-isolated building cheaper than for a traditional design, as the cost of the isolators could be higher than those savings. Nevertheless, it seems reasonable to think that, in the end, a base-isolated project may be cheaper as the superstructure would remain undamaged after a strong earthquake (Fig. 7). Then, most of the long term costs would be related to the initial investment and maintenance costs. On the other hand, the traditional designs are susceptible to damage when subjected to strong earthquakes (Fig. 7). Therefore, retrofit costs, besides the possible interruption of the use of the building, could considerably increase the long term cost analyses.

SUMMARY AND CONCLUSIONS

An analytical study on the application of different base isolation systems for original design or retrofit of typical building structures of the Mexican Pacific Coast was presented. The subject hypothetical buildings were designed both as base-isolated structures and traditional fixed-base structures founded on the hard soil conditions of the touristic port of Acapulco. Typical accelerograms for the Mexican subduction zone recorded during recent earthquakes were used for time-history analyses. Dynamic responses for the isolated structures compared favorably against those for the fixed-base structures. The study confirms many findings published in the literature regarding the effectiveness of base isolation. However, the study also shows that the dynamic stability of isolators is not always achieved using rational design procedures. The dynamic stability and design of base isolators can be controlled by epicentral acceleration records associated to moderate earthquakes. The 1994 UBC code considers the possibility that near-fault time-histories (Section 2375(d)2) and site-specific design spectra (Section 2373C3) may control the design of base isolated structures. These analyses are only enforced for structures found within 15 km of an active fault.

Torsional responses diminish the effectiveness of base isolation, as some isolators yield and displace substantially while others respond in the elastic range. The torsional response of the isolation system is greatly affected by the own torsional response of the superstructure. Clearly, more refined design methods are needed for isolated structures with strong torsional responses to prevent or diminish the torsional response on the isolation system. Important savings on the volume of concrete and steel reinforcement can be attained for the base isolated designs with respect to their counterpart fixed-base designs. However, the initial construction cost of a base-isolated structure could be higher because of the cost of the isolators. Nevertheless, it seems reasonable to think that base-isolation is a better investment for long term cost analyses with respect to fixed-base designs. For base-isolated structures, the superstructure would remain undamaged when subjected to a strong earthquake, whereas the traditional design is susceptible to strong nonlinear response that may lead to future retrofit plans and/or disruption of the operation of the building, then, increasing its cost-effective long term analysis.

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