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Seismic Isolation of Buildings Subjected to Typical Subduction Earthquake Motions for the Mexican Pacific Coast

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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 is presented. The subject hypothetical buildings are located on hard soil conditions at 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. The studied base isolation systems reported in this study are lead-rubber bearings (LRB) and steel-hysteretic dampers (SHD). For the original design case studies, 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 with 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 the studied base isolators can be controlled by acceleration records associated to moderate earthquakes when these records are near the fault plane and by torsional responses.

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. Comprehensive experimental programs have been conducted. Early development and testing of steel hysteretic dampers (SHD) and lead-extrusion dampers (LED) date from the early 1970's (Skinner et al, 1993). Laminated-rubber bearings have been tested and used primarily for bridge isolation since the 1970's, although they have been used in building structures as well. Lead-rubber bearings (LRB) were developed by Robinson in New Zealand in 1975 (Skinner et al, 1993)

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and tested and used since then. Extensive testing of teflon bearings (Mokha et al, 1990 and 1991a) and friction-pendulum (FP) isolation systems (Mokha et al, 1991b and Zayas et al, 1993) have been conducted during the last decade. Testing of base isolation systems based on pendular action has recently been done (Juhn et al, 1992 and Foutch et al, 1993). Several shaking table tests of base-isolated models have been done for different base-isolators and structural systems (eg., Chalhoub and Kelly, 1990a/b; Griffith et al, 1990; Mokha et al, 1991b; Yaghoubian, 1991; Paulson et al, 1991; Constantinou et al, 1992; Nagarajaiah et al, 1992; Juhn et al, 1992; Zayas et al, 1993 and Foutch et al, 1993). A full-scale dynamic test of a building seismically isolated with laminated-rubber bearings was done in Ancona, Italy (Giuliani, 1993).

Comprehensive analytical research, associated in many instances to experimental research programs, has also been conducted. Simple and complex constitutive models for different base isolators have been developed (e.g., Koh and Kelly, 1989 and 1990; Constantinou et al, 1990; Skinner et al, 1993; Buckle and Liu, 1993; Nagarajaiah et al, 1993a/b and Ali and Abdel-Ghaffar, 1995). Approximate and rigorous methods of dynamic analysis have been proposed for base-isolated structures. These methods range from the use of equivalent linearization or non-classical damped modes for the isolation-superstructure system (Skinner et al, 1993) to 3-D analyses where the response of the isolators can be nonlinear whereas the superstructure is considered elastic always using its most representative fixed-base mode shapes (Nagarajaiah et al, 1991a/b). Standard 2-D and 3-D finite element software for nonlinear dynamic analysis has also been used to study base-isolated structures.

Numerical simulations and parametric studies on the seismic response of hypothetical, shaking table models and full-size base-isolated structures have been reported in the sizable literature on base isolation. These studies can be classified as: a) Parametric studies to show the effectiveness of base-isolation in reducing the seismic response of structural systems when subjected to real or artificial acceleration records (eg., Lee and Medland, 1979; Fan and Ahmadi, 1990; Kartoum et al, 1992 and Chen and Ahmadi, 1992); b) Comparative studies on the seismic response of different isolators when subjected to real earthquake motions (e.g., Su et al, 1990 and Fan and Ahmadi, 1990); c) Correlation of numerical simulations with shaking table test results (e.g., Mokha et al, 1991b; Juhn et al, 1992 and Nagarajaiah et al, 1992) and, d) Studies on the effect of nonlinear dynamic torsional coupling in base-isolated systems (e.g., Nagarajaiah et al, 1993a/b, and Jangid and Datta, 1994a/b). The most popular acceleration records used for these studies have been the well-known El Centro, Parkfield, Paicoma Dam and Taft Californian records, and the Hachinohe and Miyagiken-Oki Japanese records. The SCT-EW record registered on soft soil conditions in Mexico City during the 1985 earthquake has also been used.

The comprehensive experimental and analytical research conducted during the last twenty five years has proved the usefulness of most isolators in reducing the seismic response of low-rise structures when subjected to earthquake motions typical of hard soil conditions. Interstory drift angles and base shear of the superstructure are considerably reduced when compared with counterpart rigid-base structures. Special devices for displacement-control and uplift restraint have been developed and successfully tested (Griffith et al, 1990 and Nagarajaiah et al, 1992), so the use of base isolation on mid-rise structures subjected to important overturning moments seems promising as well. It has been shown that the effectiveness of base isolators can be considerably diminished when torsional coupling associated to the superstructure and/or the isolation system is significant. Therefore, special attention has to be paid to the design of base-isolated structures under those conditions. Nevertheless, the torsional response of the superstructure of base-isolated structures can be more favorable than that of their counterpart rigid-base structures, as torsional coupling in the superstructure can be diminished because of the lateral flexibility of the isolators.

As a result of the extensive research done in the last 25 years and briefly summarized above, 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 (Skinner et al, 1993), and at least three buildings were under construction (Giuliani, 1993). As of December of 1990, there were 45 seismically isolated buildings and bridges in New Zealand ("UBC", 1993). As of February of 1993, there were 47 seismic isolated buildings in the United States ("UBC", 1993) and 26 projects under way, 14 for seismic retrofit and 12 for original design (Kelly, 1993). The number of base-isolated structures in Japan is more than 70 nowadays (Kelly, 1995).

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; Mayes et al, 1992) and the Uniform Building Code of 1991 (UBC; "Uniform", 1991). Some analytical and experimental studies have been carried out to assess the effectiveness of those provisions (Chalhoub and Kelly, 1990; Mostaghel and Mortazavi, 1991; Paulson et al, 1991 and Shenton and Lin, 1993). Reasonable correlations have been observed on these studies.

MEXICAN EXPERIENCE ON BASE ISOLATION

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 four base-isolated structures in Mexico. The first two structures are a four-story school building and a church isolated with a sliding device composed of steel spheres, which was developed by González-Flores in the 1970's. The third 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). The last structure is a highway bridge distributor using rubber bearings. These structures are found in Mexico City metropolitan area and suburbs 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. Mexico City is not the best place for base isolation. 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 relatively low because of the low-period ground motions associated to the design earthquake in these regions. Thus, a conventional design procedure for building structures is economical in these zones, so base isolation seems to offer no substantial advantages. Also, building officials in Mexico City are skeptical about the use of base isolation in the city, particularly in the lake-bed zone, for the reasons explained above. Finally, real-state companies nationwide are not willing to invest their money on a technology that has not been used in Mexico before. There have been base-isolated projects for buildings in Mexico using isolation systems tested and developed in the United States. However, these projects have not been constructed as the investors do not want to be the first ones to finance a base-isolation project in Mexico (Del Valle, 1995 and Martínez-Romero, 1994).

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, for example, Acapulco, Ixtapa, Manzanillo, Taxco, Puerto Vallarta, Oaxaca, Huatulco, Lázaro Cárdenas, etc. The soil conditions and earthquake motions for the Mexican Pacific Coast, particularly the coast of Guerrero and Michoacán states, lead one to believe that base isolation is attractive both for original design and retrofit. In addition, the southern part of the coast of the Gulf of Mexico is a region of moderate seismic hazard as it is affected by normal faulting 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 hazard in Mexico (Tena et al, 1995). The project is assessing the effectiveness of some commercial base isolators such as LRB, HSD and FP 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 studied structural systems are hotel (2), school (5), apartment (2) and office (2) buildings, whose configurations correspond to existing building structures. The studies done on four of the buildings are summarized in following sections. The final objective of the on-going research program is to develop reliable design procedures and to elaborate a set of recommendations for the seismic design of base isolated structures for the Mexican Pacific Coast, taking into account the regional seismicity.

SUBJECT BUILDINGS

Eleven structures are currently under study in the research project of reference (Tena et al, 1995): two hotel buildings, two office buildings, two apartment buildings and five school buildings. A brief description for four of them is given below.

Hotel Building H1

Hotel H1 is a typical architectural project for a five-star hotel for the touristic ports of Mexico. A view of the architectural model is depicted in Fig. 1. The hotel consists of three six-story buildings properly separated one from each other. The two twin L-shaped buildings with irregularities both in plan and elevation are destined for the guest rooms and are the buildings under study. The typical plan view and overall dimensions for these buildings are shown in Fig. 2. The structural system for lateral loading is an RC dual system composed of ordinary moment resisting frames 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 change their dimensions because of the architectural design. 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, including the final cross-section dimensions and reinforcement for the structural elements.

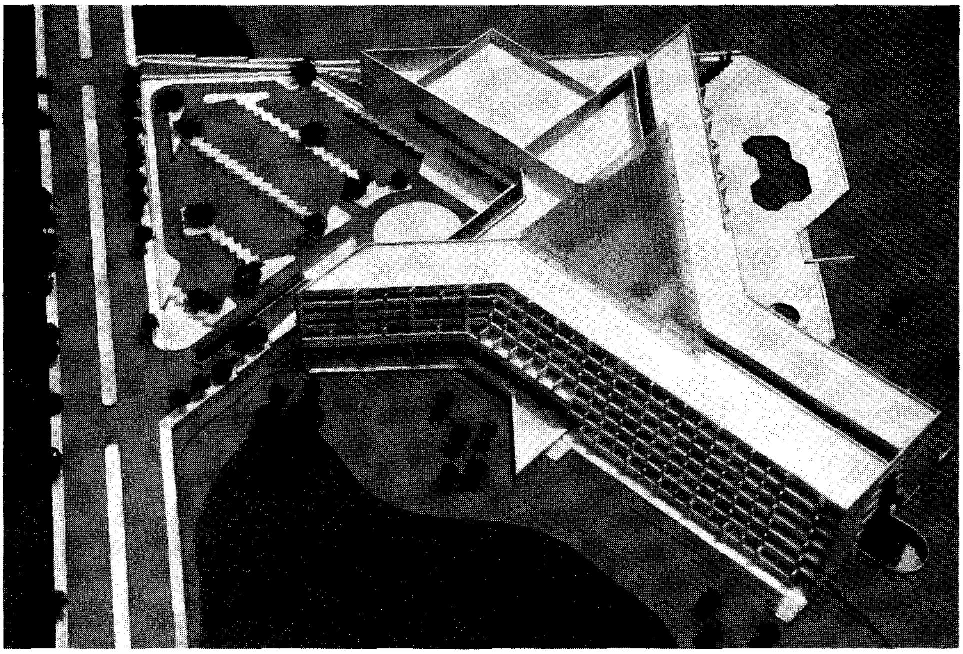


Figure 1. Architectural model for hotel H1

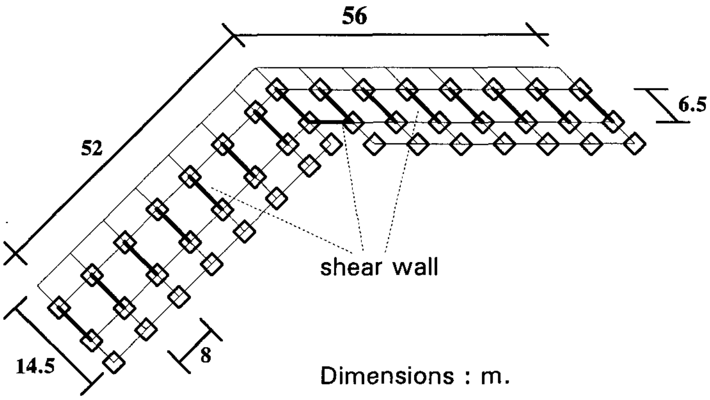


Figure 2. Plan view for one of main buildings of hotel H1

Hotel Building H2

Hotel H2 is based on a project for a five-star hotel complex for the touristic port of Manzanillo (Martínez-Romero, 1994). 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. 3. 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. 10). 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. The Manzanillo project was borrowed and designed as an Acapulco project in the present study. The same design strategies undertaken for hotel H1 were followed. The details will be described in following sections.

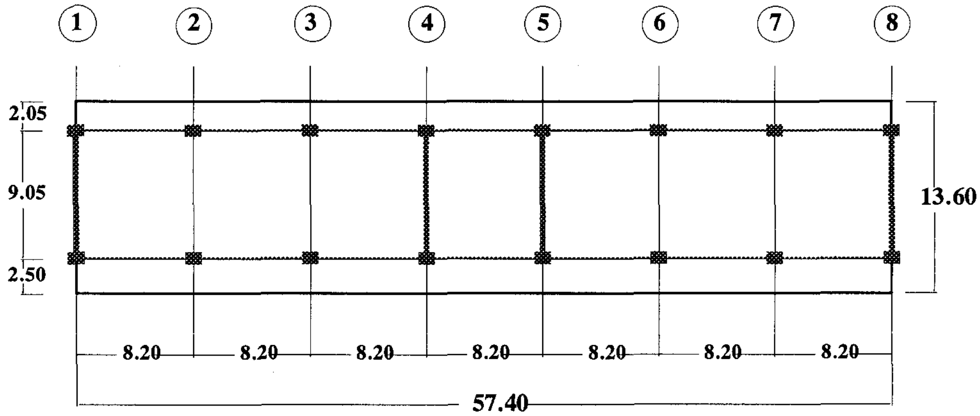
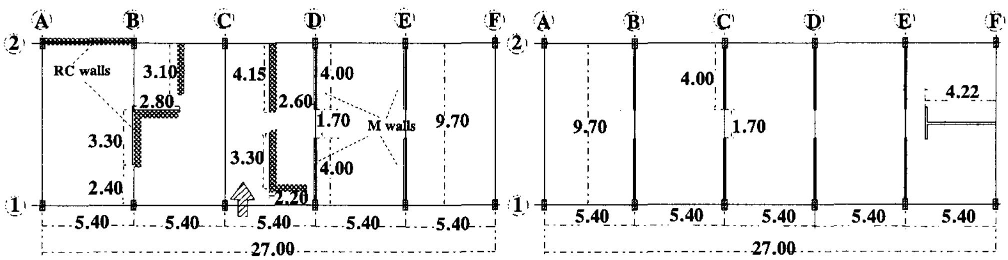


Figure 3. Plan view for hotel H2 (dimensions in m.)

Office Building O1

Office building O1 is a typical low-rise office building in Mexico. Typical plan views and overall dimensions are depicted in Fig. 4. The three-story building consists of five bays 5.40 m wide in the longitudinal direction and one 9.70 m bay in the transverse direction. The typical story height is 2.90 m. The structural system for lateral loading is composed of RC ordinary moment resisting frames, unreinforced brick masonry infill walls in all stories and RC shear walls in the first story (Fig 4). The plane and L-shaped RC shear walls are supposed to be 25 cm thick (Fig 4). Infill masonry walls are standard 12.5 cm thick bricks produced in Mexico. The building layout is regular in plan and elevation, but the poor judgment on the distribution of structural walls in plan introduces torsional responses. The building was also designed as a fixed-based and base-isolated structure. As for buildings H1 and H2, details will be described in following sections.

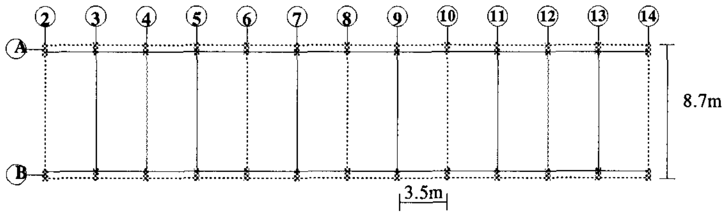


a) First floor b) Second and third floors

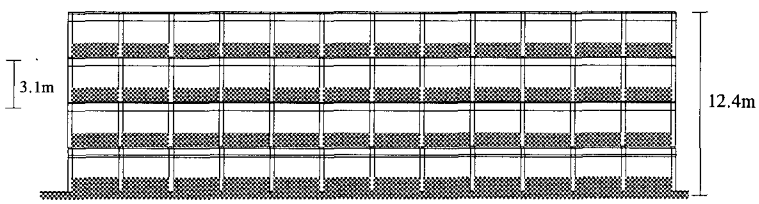
Figure 4. Plan views for office building O1 (dimensions in m)

School Building EP2

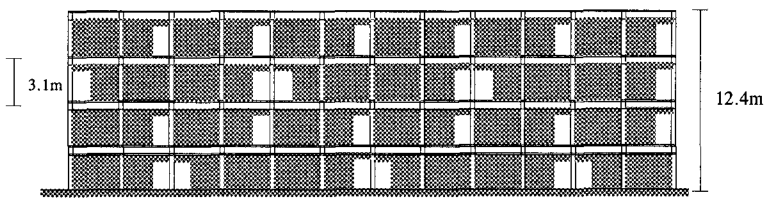
School building EP2 is a typical design for a four-story school building in Mexico in the 1970's. Typical plan view, elevation and overall dimensions are depicted in Fig. 5. The building is 12.4 m tall (story height of 3.1 m) with typical bay width of 3.5 m in the longitudinal direction and 8.7 m in the transverse direction. Transverse infill unreinforced concrete masonry (URM) walls are placed every two bays. Transverse outer walls are reinforced concrete walls. Longitudinal clay URM walls do not run all the story height, and they provide lateral confinement to the columns along their lower portion, but leave their upper portion free, acting as short columns (Fig. 5). The main structural system consists of 25x50 cm rectangular RC columns oriented in the transverse direction, 25x50 cm rectangular RC beams, and 10 cm thick RC slabs. Longitudinal reinforcement is the same in all columns ($\rho=0.046$). The longitudinal reinforcement supplied at the top and bottom of the beams is symmetric and varies from $\rho=0.0057$ for the top story beams to $\rho=0.0095$ for the first story beams ($\rho=\rho_{top}=\rho_{bottom}$). For both buildings, the specified strength of the concrete (f'_c) was 200 kg/cm^2 and the yielding strength of the reinforcement steel (f_y) was 4200 kg/cm^2. The assumed compressive strength of the masonry (f^*_m) was 15 kg/cm^2 for the clay units and 20 kg/cm^2 for the concrete units, according to the masonry provisions referenced by the seismic code of the state of Guerrero (RCGS-90, 1990). Because this building represents many schools of the times in regions of high seismicity nationwide, a seismic retrofit plan was designed to assess the effectiveness of base isolation for the retrofit of similar buildings in the Mexican Pacific Coast.



a) Plan View



b) Elevation axis A



c) Elevation axis B

Figure 5. Plan view and elevations, school building EP2

ACCELERATION RECORDS

Typical accelerograms for the Mexican subduction zone recorded during recent earthquakes were selected for the present study. Subduction earthquakes are the main events for the design spectra of the Mexican Pacific Coast (Ordaz, 1996). 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 in Figs. 6 to 8 respectively. It can be observed that the 1985 records have longer strong-phase durations; however, the highest peak ground accelerations and acceleration spectral ordinates are similar to records for moderate earthquakes with short strong motion durations (stations VIGA and LLAV, Figs. 7 and 8). Most stations are within the Mexican Pacific Coast line, but stations MSAS and PARS, that are within 50 km of the coast line. All stations are located in Guerrero state. Station SMR2 for the 1989 earthquake has a strong pulse associated to an intermediate period (Fig. 7). This peculiarity is a consequence of station SMR2 being an epicentral record for the 1989 event (Pérez-Rocha, 1995). The selection of acceleration records for moderate to strong earthquakes is based on recommendations given in the literature. According to Clark et al (1993), severe events should be used for the design of the superstructure and isolators, whereas minor events should be used to check that non-structural damage may not occur during low levels of earthquake shaking.

CONVENTIONAL DESIGN

Buildings H1, H2 and O1 were designed as conventional structures according to the provisions of the seismic code of the state of Guerrero (RCGS-90. 1990). All building structures were assumed to be located in the port of Acapulco. 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. 9. 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=2 for buildings that comply with the regularity conditions of the code. However, since the subject buildings violate at least one of the eleven regularity conditions outlined in RCGS-90, response modification factors should be reduced by a 0.8 factor. Then, Q=1.6 was used for the design of buildings H1, H2 and O1.

Subject buildings were designed to comply with the requirements of RCGS-90 using 3-D response spectra analyses with ETABS (Habbibulah, 1991). 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 and office building O1. Young modulus 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. 10. The dynamic characteristics of the final fixed-base designs are summarized in Table 1. It can be observed the strong torsional coupling for buildings H1 and O1. In contrast, torsional coupling is moderate for building H2 and only for the transverse direction because of the setback (Fig. 10).

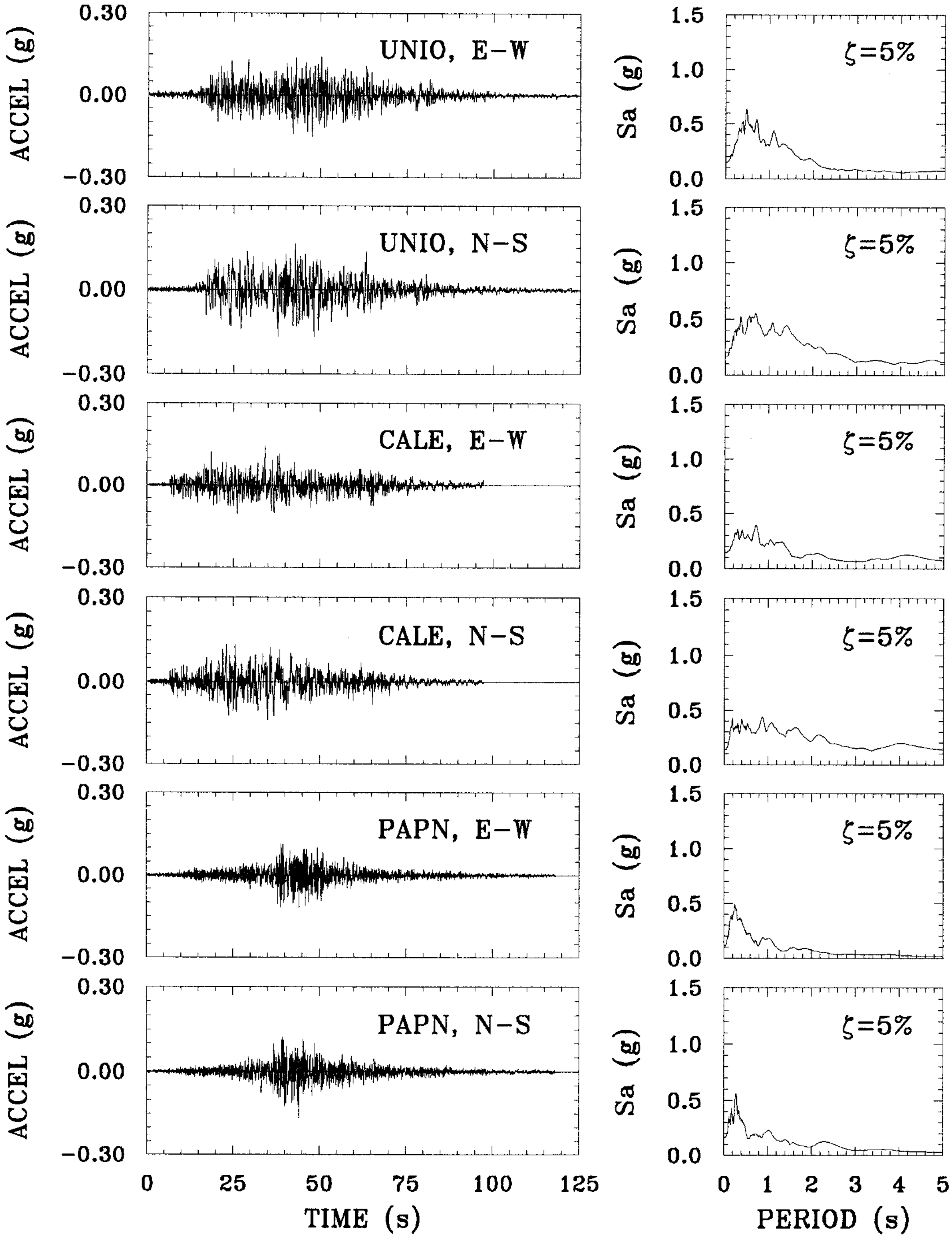


Figure 6. Accelerograms for the 09/10/85 earthquake.

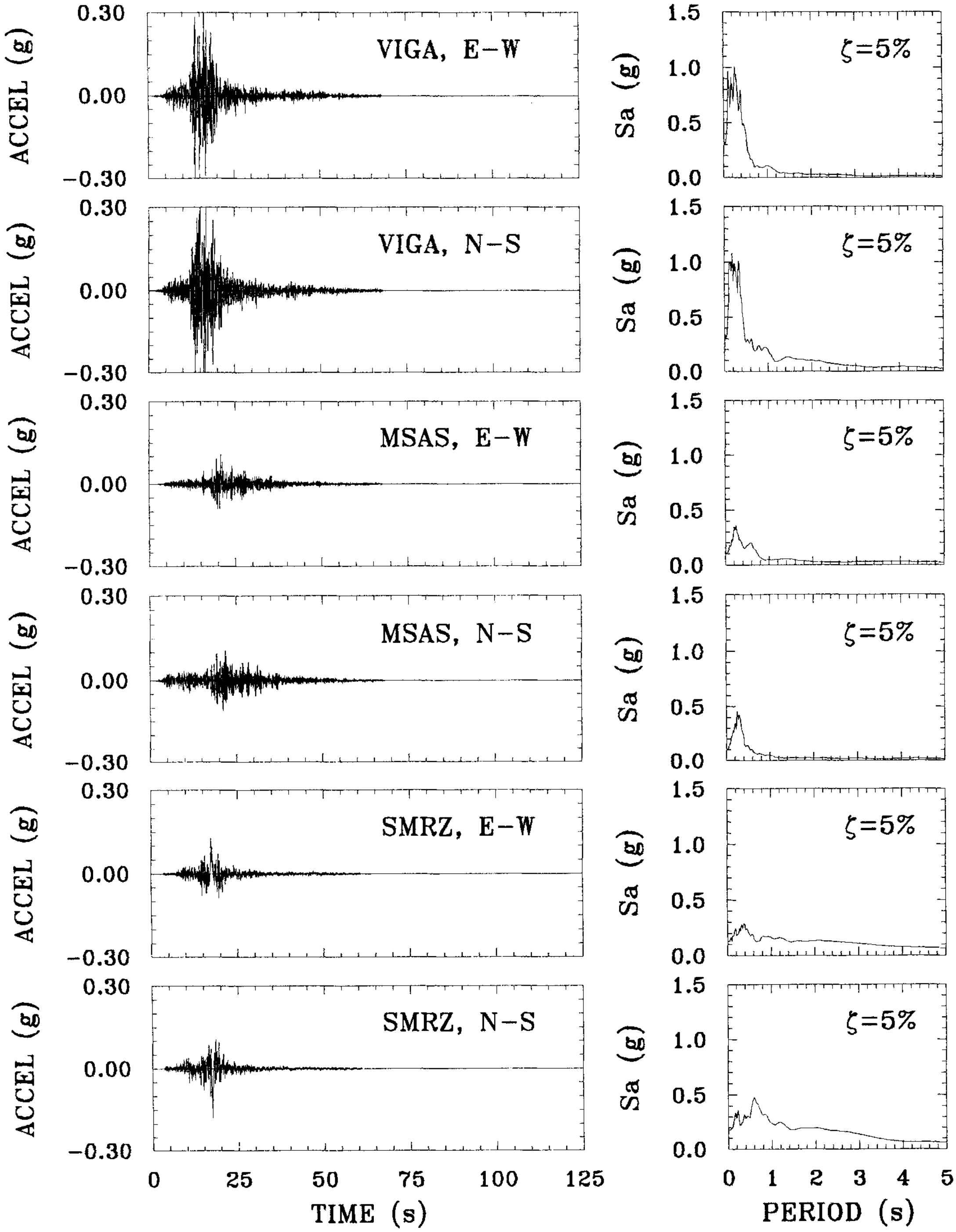


Figure 7. Accelerograms for the 04/25/89 earthquake.

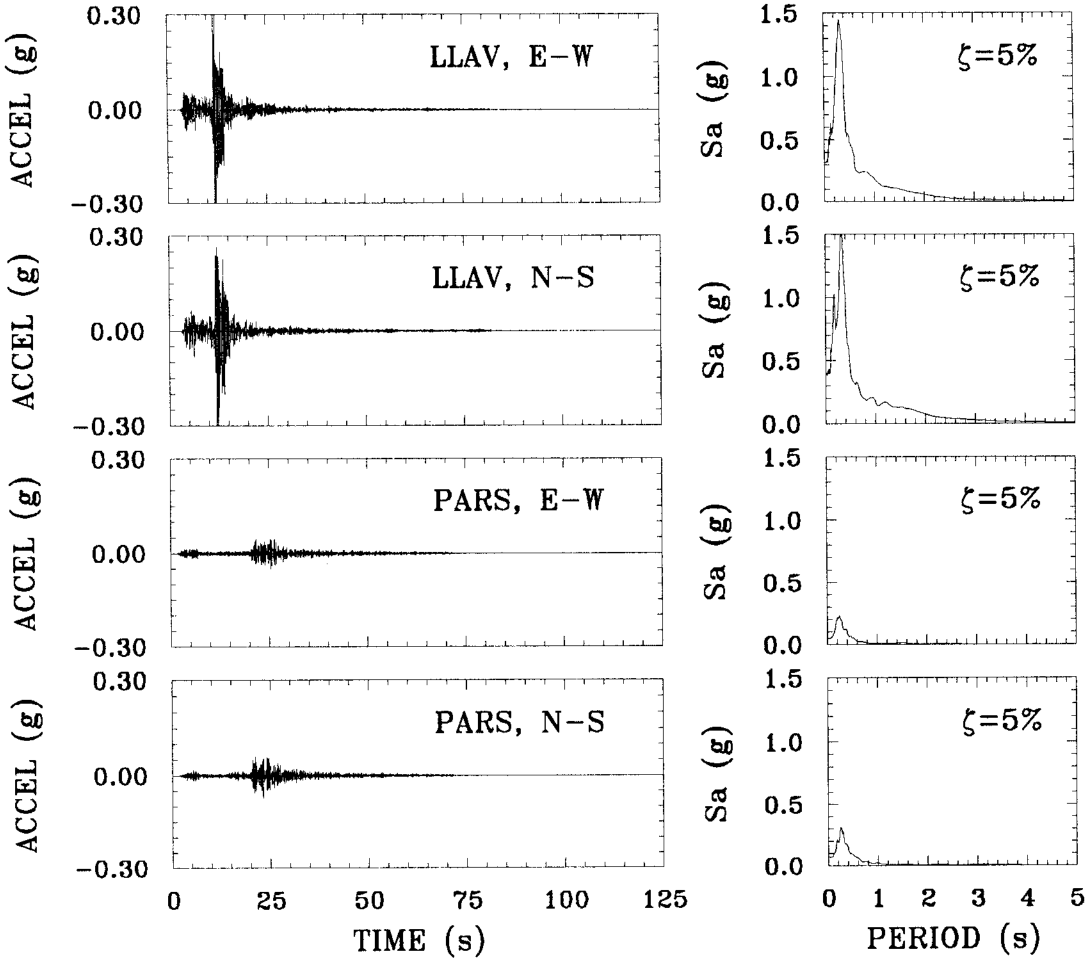


Figure 8. Accelerograms for the 05/13/90 earthquake.

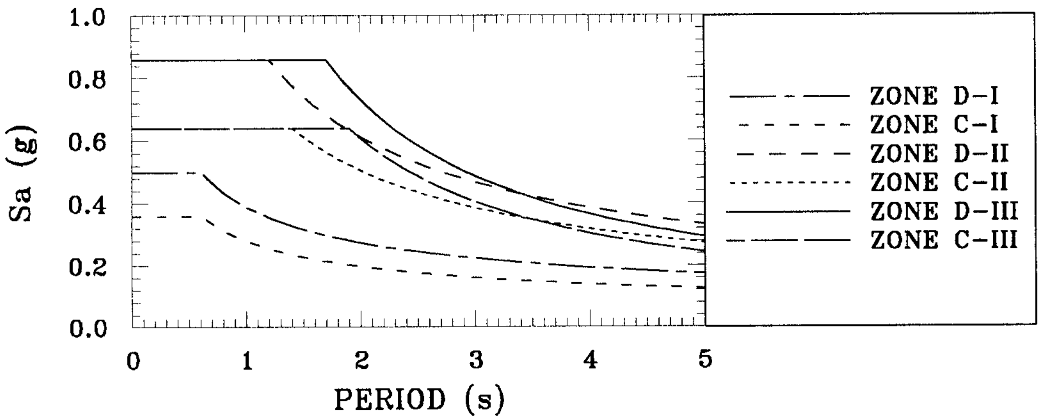
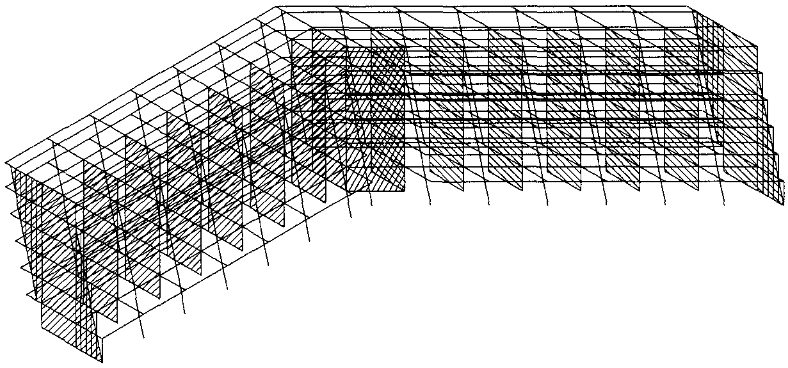
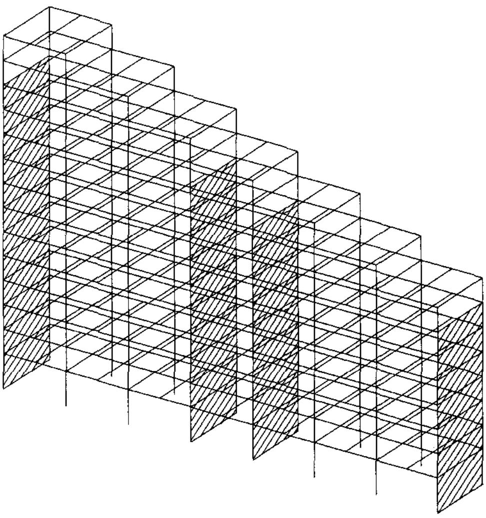


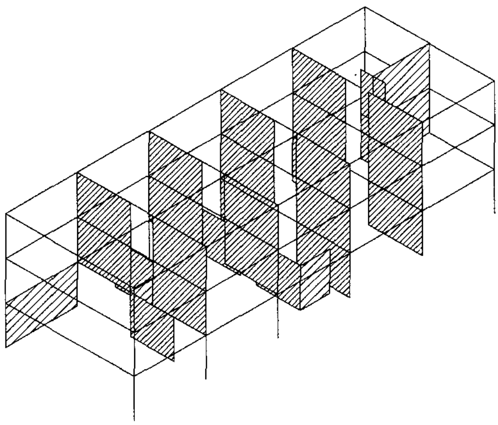
Figure 9. Elastic design spectra for RCGS-90 code.



a) Hotel H1



b) Hotel H2



c) Office building O1

Figure 10. ETABS Models for the buildings under study

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 of the two outer shear walls is 50 cm whereas for the remaining interior walls is 30 cm. Longitudinal reinforcement steel ratios vary from $\rho=0.010$ to $\rho=0.0353$ for the columns. Longitudinal reinforcement ratios for beams vary from $\rho=0.0037$ to $\rho=0.0111$. Shear reinforcement ratios (volume of confining stirrups over volume of confined concrete) at the ends of the members vary from $\rho_{sh}=0.0023$ to $\rho_{sh}=0.0042$ for the columns and from $\rho_{sh}=0.0021$ to $\rho_{sh}=0.0047$ for the beams. Shear reinforcement ratios for the walls vary from $\rho=0.0049$ to $\rho=0.0079$ for both the horizontal and vertical directions.

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

Building	Mode	Period (s)	Modal Mass (%)		
			Longitudinal	Transverse	Rotational
H1	1	0.389	52.53	18.06	4.84
	2	0.239	28.05	39.49	7.86
	3	0.124	9.52	4.68	1.00
H2	1	1.258	78.18	0.00	0.00
	2	0.751	0.00	43.21	7.23
	3	0.468	0.00	24.92	37.02
01	1	0.363	64.26	2.65	11.01
	2	0.347	11.30	35.62	28.00
	3	0.302	1.37	35.42	38.40
EP2	1	0.631	88.35	0.00	0.00
	2	0.119	0.00	77.40	0.01

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 $\rho=0.007$, slightly higher than the minimum required by the RCGS-90 code ($\rho=20/f_y=0.005$) for ordinary moment resisting frames for dual systems. Longitudinal reinforcement ratios for beams vary from $\rho=0.0072$ to $\rho=0.0100$. 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 $\rho_{sh}=0.0030$ to $\rho_{sh}=0.0117$ for the beams. Shear reinforcement ratios for the walls vary from $\rho=0.0025$ to $\rho=0.0049$ for both the horizontal and vertical directions.

Office Building O1

The seismic design for office building O1 consists of rectangular columns with cross sections measuring 45 x 70 cm for all stories. Interior beams are 40 x 55 cm in the first story and 40 x 50 in the upper stories. Exterior beams are 40 x 50 and 30 x 45 cm for the first and second floors and 30 x 45 and 25 x 45 cm for the roof level. Additional 15 x 55 cm spandrel beams placed below and casted monolithically with the exterior beams are provided at the boundaries to prevent slab deflections and for architectural purposes. The thickness of the RC shear walls is 30 cm. Longitudinal reinforcement steel ratios for the columns vary from $\rho=0.010$ for the top floor to $\rho=0.0130$ for the first and second floors. Longitudinal reinforcement ratios for beams vary from $\rho=0.0086$ to $\rho=0.0130$. Shear reinforcement ratios at the ends of the members vary from $\rho_{sh}=0.0056$ to $\rho_{sh}=0.0112$ for the columns and from $\rho_{sh}=0.0053$ to $\rho_{sh}=0.0080$ for the beams. Shear reinforcement ratios for the walls vary from $\rho=0.0025$ to $\rho=0.0044$ for both the horizontal and vertical direction.

SEISMIC ISOLATION DESIGN: BASE ISOLATORS

Base isolators were designed as an original design for buildings H1, H2 and O1 and as retrofit for building EP2. For these buildings, only lead-rubber bearings (LRB) and steel hysteretic dampers (SHD) were considered, primarily because there is more information on design procedures for this type of isolators available in books (e.g., Skinner et al, 1993). 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 by Tena et al (1995). There are no code design procedures for base-isolated structures available in Mexico yet, so the design criteria were based upon recommendations available in the literature, but the procedure did not follow any building code or recommendation in particular.

Proposed design methods by the 1994 UBC code ("UBC", 1994) consist of either a static analysis for regular structures less than four stories tall located farther than 15 km from active faults, or dynamic analyses for taller, more irregular structures that are closer to a fault. The dynamic analyses' options include a) Response spectrum analysis using the code design spectra, b) Time-history analysis and, c) Response spectrum analysis using site-specific design spectra in lieu of the code design spectra. The UBC code also specifies that the deformation characteristics of the isolation system shall be based on properly substantiated tests according to procedures outlined by the code, and other criteria that the interested reader can consult elsewhere ("UBC", 1994).

Given the irregularities of most of the selected buildings, with important torsional responses, the design of the isolators was based upon 3-D time-history analyses using bidirectional input. The criteria for dynamic analysis of the UBC code was not used in the design procedure, as it was recognized that the UBC procedure is based on the seismicity of the United States. Therefore, the UBC criteria cannot be applied directly to Mexico without conducting detailed studies devoted to find how these procedures can be adapted in Mexican codes (design criteria and design spectra). Some studies are currently being done in this direction (Gómez-Soberón, 1996). Thus, the two horizontal components for the recorded ground motions during the strong $M_S=8.1$, 1985 Michoacán Earthquake (stations UNIO, CALE and PAPN, Fig. 6) were used in the time-history analyses, as the use of bi-directional input has been suggested in the literature (eg., Clark et al, 1993). The design of the different base isolators was done using the 3D-Basis computer program (Nagarajaiah et al, 1991a). The first six to twelve mode shapes for the fixed-base structural models, the characteristics of which are summarized in 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, LRB isolators were designed for global yield strengths of a) 0.05W and, b) 0.10W, for both the longitudinal and transverse directions, where W is the total weight of the superstructure. The post yield stiffness of the LRB isolators was taken as ten percent of their initial elastic stiffness (Skinner et al, 1993). Material properties for the rubber bearing and the lead core were based upon the data reported by Skinner et al (1993). The base isolated structure was designed for an effective isolated natural period, T_I, of at least 1.5 s in both directions based upon calculated effective stiffness for the isolators. All LRB isolators were of circular cross section. One LRB isolator is placed under each base column. The final design for the case when $V_{yield}=0.10W$ included 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 were $X_M=Y_M=21.7$ cm. The maximum effective isolated natural period was $T_I=2.36$s.

Peak dynamic responses for this isolation design when subjected to the 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. 7) 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 earthquake ground motions.

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

Event	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	CALE	0.21	0.14	0.398	0.557	845.3	1112.8	0.068	0.090
	PAPN	0.14	0.09	0.192	0.275	760.9	846.5	0.061	0.068
	UNIO	0.23	0.16	0.450	0.841	910.8	1152.9	0.073	0.093
04/25/89	VIGA	0.24	0.16	0.153	0.109	694.4	616.6	0.056	0.050
	MSAS	0.12	0.08	0.106	0.183	543.8	354.8	0.044	0.029
	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
	PARS	0.10	0.07	0.070	0.052	265.2	250.0	0.021	0.020

The torsional response of the LRB isolators is schematically depicted in Fig. 11 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 concluded 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. 11 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 design was done for a yielding force V_{yield}=0.04W and consisted of a total of 45 rectangular Type E SHD isolators in each direction (90 total). The post yield stiffness for these SHD isolators was 18% of their initial elastic stiffness. The maximum allowable isolator displacements for dynamic stability are $X_M=Y_M=52$ cm. The maximum effective isolated natural period was T_I=3.23s. Details for the design of the isolators are in Tena et al, 1995.

Table 3. Peak dynamic displacements and base shear vs. allowable limits for the Type-E, SHD base isolation project for Hotel H1

Event	Station	Max relative roof displacements wrt the isolators (cm)		Peak SHD 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	CALE	1.2	0.9	0.500	0.567	1092.1	1122.9	0.088	0.090
	PAPN	0.4	0.4	0.090	0.150	333.1	493.4	0.027	0.040
	UNIO	0.7	0.5	0.323	0.488	624.4	687.8	0.050	0.055
04/25/89	VIGA	0.4	0.3	0.094	0.165	328.8	415.9	0.026	0.033
	MSAS	0.3	0.2	0.054	0.073	265.6	222.2	0.021	0.018
	SMR2	0.6	0.4	0.360	0.810	641.0	452.5	0.052	0.036
05/31/90	LLAV	0.5	0.4	0.092	0.122	385.9	448.9	0.031	0.036
	PARS	0.2	0.1	0.031	0.026	127.4	153.5	0.010	0.012

Peak dynamic responses for the SHD isolation design when subjected to the acceleration records for the 1985, 1989 and 1990 Mexican earthquakes are summarized in Table 3. It can be observed that the proposed design performs very well when subjected to all the considered ground motions. Peak roof displacements and base shears are low and the dampers are dynamically stable. The dynamic stability is consequence of the higher post yield stiffness and higher allowable displacements. Higher deformation demands for the SHD isolators are still observed for 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.

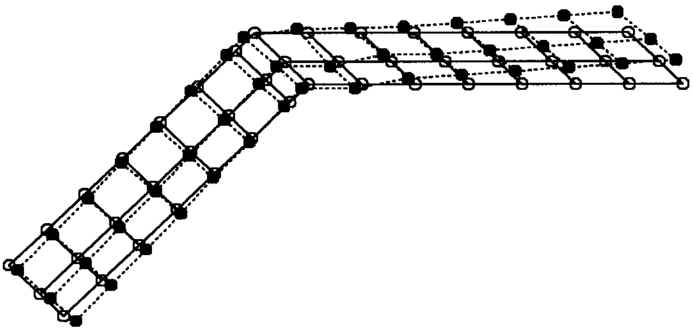


Figure 11. Torsional response of the LRB isolators of hotel H1

Hotel Building H2

For hotel H2, LRB isolators were designed in the same fashion as for hotel H1, but for a global yield strength of 0.10W only. 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 $T_I = 2.72$s.

Peak dynamic responses for this isolation design when subjected to the acceleration records of the 1985, 1989 and 1990 Mexican earthquakes are summarized in Table 4. 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 (marked with an asterisk), regardless the fact that torsional coupling is smaller. A small torsional response was observed in the isolators in the transverse direction (y), mostly influenced by the torsional eccentricity due to the superstructure. Peak roof displacements are small, taken into account the height of the structure (H=50.50 m).

Table 4 Peak dynamic displacements and base shear vs. allowable limits for the LRB base isolation project for Hotel H2

Event	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	CALE	2.9	2.2	0.524	0.731	580.2	780.4	0.075	0.100
	PAPN	1.9	1.7	0.251	0.370	428.7	539.8	0.055	0.069
	UNIO	3.6	2.9	0.486	0.953	649.7	898.4	0.084	0.116
04/25/89	VIGA	1.3	2.6	0.167	0.449	185.6	667.0	0.024	0.086
	MSAS	2.3	1.2	0.159	0.172	300.9	317.0	0.039	0.041
	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
	PARS	0.9	0.7	0.073	0.086	120.2	170.3	0.016	0.022

A base isolation design with SHD was also done. Torsional-beam dampers with transverse loading arms (Type E damper) of rectangular cross section and tapered-cantilever bending-beam dampers (Type T damper) of square cross sections were selected (Skinner et al, 1993). The type E dampers yield better results and are presented herein. 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 (32 total). The post yield stiffness for these SHD isolators was 6% of their initial elastic stiffness. The maximum allowable isolator displacements for dynamic stability are $X_M=Y_M$=46.3 cm. The maximum effective isolated natural period was T_I=3.88s. Details on the design can be consulted elsewhere (Tena et al, 1995).

Table 5 Peak dynamic displacements and base shear vs. allowable limits for the Type E, SHD base isolation project for Hotel H2

Event	Station	Max relative roof displacements wrt the isolators (cm)		Peak SHD 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	CALE	5.0	3.0	0.221	0.374	556.2	724.1	0.072	0.093
	PAPN	3.2	2.2	0.113	0.168	421.9	585.9	0.054	0.075
	UNIO	5.8	3.8	0.244	0.436	506.8	826.1	0.065	0.106
04/25/89	VIGA	1.4	2.6	0.079	0.210	185.5	651.5	0.024	0.084
	MSAS	2.4	1.2	0.073	0.080	296.6	315.8	0.038	0.041
	SMR2	5.5	3.5	0.501	0.602	639.7	679.1	0.082	0.087
05/31/90	LLAV	3.1	1.8	0.073	0.131	273.2	416.8	0.035	0.054
	PARS	1.0	0.7	0.034	0.040	120.9	169.8	0.016	0.022

Peak dynamic responses for the SHD isolation design when subjected to the acceleration records of the 1985, 1989 and 1990 Mexican earthquakes are summarized in Table 5. It can be observed that the proposed design performs very well when subjected to all the considered 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, the peak roof displacements for the SHD alternative are higher than those computed for LRB option.

Office Building O1

LRB isolators for building O1 were designed in the same fashion as for hotel H2. The final design was a total of 12 LRB 45 x 55 cm rectangular isolators 30 cm in height, with a lead core 9.9 cm in diameter. The orientation of the isolators is in the transverse (y) direction. The allowable isolator displacements for dynamic stability are X_M= 9 cm and Y_M=12.5 cm, according to the procedure outlined in Tena et al (1995). The effective isolated natural periods were T_{Ix}=1.55s and T_{Iy}=1.59s.

Peak dynamic responses for this isolation design when subjected to the acceleration records of the 1985, 1989 and 1990 Mexican earthquakes are summarized in Table 6. The proposed design does very well when subjected to most of the considered ground motions, but dynamic instability is again detected for the LRB when the building is subjected to the SMR2 records (marked with an asterisk). A small torsional response was also observed in the isolators, mostly influenced by the torsional eccentricity associated to the superstructure. Peak roof displacements are small. For this case, the design of the isolators was near optimal conditions, as all of them yield for the $M_S=8.1$ earthquake.

A base isolation design with SHD was also done. Torsional-beam dampers with transverse loading arms (Type E damper) of rectangular cross section and tapered-cantilever bending-beam dampers (Type T damper) of square cross sections were also selected. The type T dampers yield better results for this building. The design was done for a yielding force $V_{yield}=0.14W$ and consisted of a total of 12 square Type T SHD isolators in each direction (24 total). The post yield stiffness for these SHD isolators was 3% of their initial elastic stiffness. The maximum allowable isolator displacements for dynamic stability are $X_M=Y_M=14.3$ cm. The effective isolated natural period was $T_I=1.88$s. Details on the design can be consulted elsewhere (Tena et al, 1995).

Table 6 Peak dynamic displacements and base shear vs. allowable limits for the LRB base isolation project for Office building O1

Event	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	CALE	0.5	0.8	0.484	0.761	127.0	171.2	0.112	0.151
	PAPN	0.4	0.6	0.321	0.265	100.9	114.6	0.089	0.101
	UNIO	0.6	0.9	0.671	0.932	142.8	183.7	0.126	0.162
04/25/89	VIGA	0.5	0.7	0.213	0.277	93.0	121.8	0.082	0.108
	MSAS	0.3	0.4	0.101	0.087	67.1	76.2	0.059	0.067
	SMR2	0.5	0.7	1.136*	1.023*	141.4	175.6	0.125	0.155
05/31/90	LLAV	0.8	0.8	0.595	0.360	133.1	115.2	0.118	0.102
	PARS	0.2	0.2	0.047	0.034	30.5	31.0	0.027	0.027

Table 7 Peak dynamic displacements and base shear vs. allowable limits for the Type-T, SHD base isolation project for Office building O1

Event	Station	Max relative roof displacements wrt the isolators (cm)		Peak SHD 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	CALE	0.4	0.8	0.247	0.512	148.6	172.5	0.131	0.152
	PAPN	0.4	0.7	0.209	0.255	128.6	142.6	0.114	0.126
	UNIO	0.5	0.8	0.321	0.583	161.2	170.8	0.142	0.151
04/25/89	VIGA	0.4	0.8	0.121	0.229	104.8	149.6	0.093	0.132
	MSAS	0.3	0.5	0.056	0.079	70.9	89.3	0.063	0.079
	SMR2	0.4	0.6	0.559	0.673	129.6	151.9	0.115	0.134
05/31/90	LLAV	0.6	0.9	0.283	0.218	139.5	136.8	0.123	0.121
	PARS	0.1	0.2	0.024	0.025	30.5	30.5	0.027	0.027

Peak dynamic responses for the SHD isolation design when subjected to the acceleration records of the 1985, 1989 and 1990 Mexican earthquakes are summarized in Table 7. As for other buildings, it can be observed that the proposed design performs very well when subjected to all the considered 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 for the epicentral record SMR2. The torsional response is less important than for hotel H1. The peak roof displacements for SHD alternative are similar to those computed for LRB option. All dampers yield under the excitation of the 1985 earthquake, with a reasonable margin of safety for the isolators' displacements.

School Building EP2

The retrofit LRB isolators for school building EP2 were designed as for previous buildings. The final design was a total of 26 circular LRB isolators 45 cm in diameter and 40 cm in height, with a lead core 8.0 cm in diameter. The maximum allowable isolator displacements for dynamic stability are $X_M = Y_M = 15.0$ cm. The effective isolated natural periods were $T_{Ix} = T_{Iy} = 1.71$s.

Peak dynamic responses for this isolation design when subjected to the acceleration records for the 1985, 1989 and 1990 Mexican earthquakes are summarized in Table 8. The proposed design performs very well when subjected to all of the considered ground motions. Dynamic stability is achieved even when the building is subjected to the SMR2 records. This is in part related to the fact that the building has almost no torsional response for the superstructure. Peak roof displacements are small. For this case, the design of the isolators was near optimal conditions, as all of them yield for the $M_s = 8.1$ earthquake. Also, the isolators respond in the elastic range for MSAS and PARS stations. The retrofit design was successful to achieve dynamic stability of the isolators and to warrant low levels of deformation for the superstructure.

Table 8 Peak dynamic displacements and base shear vs. allowable limits for the LRB base isolation retrofit project for school building EP2

Event	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	CALE	1.58	0.06	0.621	0.342	184.2	131.1	0.120	0.086
	PAPN	1.15	0.05	0.208	0.170	119.2	139.5	0.078	0.091
	UNIO	1.86	0.06	0.902	0.444	206.7	140.1	0.135	0.092
04/25/89	VIGA	1.51	0.04	0.192	0.119	154.3	90.6	0.101	0.059
	MSAS	0.75	0.03	0.071	0.062	79.9	67.2	0.052	0.040
	SMR2	1.53	0.06	0.894	0.745	188.7	153.0	0.123	0.100
05/31/90	LLAV	1.74	0.06	0.363	0.305	135.4	121.3	0.088	0.079
	PARS	0.42	0.01	0.030	0.028	42.3	32.1	0.028	0.021

SEISMIC ISOLATION DESIGN: SUPERSTRUCTURE

The superstructures of buildings H1, H2 and O1 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 (Habbibulah, 1991). 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 (Table 3). Cross sections of rectangular columns measure 90 x 110 cm and 90 x 130 cm in 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 it 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 direction.

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 (Table 5). 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 for 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 direction.

Office Building O1

The base-isolation design for building O1 was based on the peak dynamic base shear obtained for station UNIO for the SHD isolation (Table 7). The seismic design for office building O1 consists of rectangular columns with cross sections measuring 45 x 70 cm for all stories. Interior beams are 40 x 55 cm in the first story and 40 x 50 in the upper stories. Exterior beams are 40 x 50 and 30 x 45 cm for the first and second floors and 30 x 45 and 25 x 45 cm for the roof level. Additional 15 x 55 cm spandrel beams placed and casted monolithically below the exterior beams are provided at the boundaries to prevent slab deflections and for architectural purposes. The thickness for the RC shear walls is 25 cm. Longitudinal reinforcement steel ratios for the columns vary from $\rho=0.009$ for the top floor to $\rho=0.0109$ for the first and second floors. Longitudinal reinforcement ratios for beams vary from $\rho=0.0086$ to $\rho=0.0130$. Shear reinforcement ratios at the ends of the members vary from $\rho_{sh}=0.0056$ to $\rho_{sh}=0.0075$ for the columns and from $\rho_{sh}=0.0053$ to $\rho_{sh}=0.0106$ for the beams. Shear reinforcement ratios for the walls vary from $\rho=0.0025$ to $\rho=0.0046$ for both the horizontal and vertical direction.

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. 12. 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, LLAV station and the SHD project for office building O1 and LLAV station and LRB isolation for school EP2. 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 not 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. 12. 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. For building O1, the base-isolated response is even more favorable regarding peak dynamic interstory drift angles. The fixed-base responses yield deformation levels acceptable for RC elements but not for masonry elements. The masonry walls of office building O1 should crack under these interstory drift angle levels. Similar observations can be drawn for the retrofit project of school building EP2 for the longitudinal (X) 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, 20.3% and 9.0% for hotel H2, and 4.2% and 16.5% for office building O1. These savings could be increased if a) an optimum design criterion for each cross section had been followed, b) a suitable response modification factor had taken for the base-isolated projects and/or c) the buildings had been located on different soil conditions, that is, soil types II or III, where higher ordinates are defined for the pseudo-acceleration design spectra (Fig. 9).

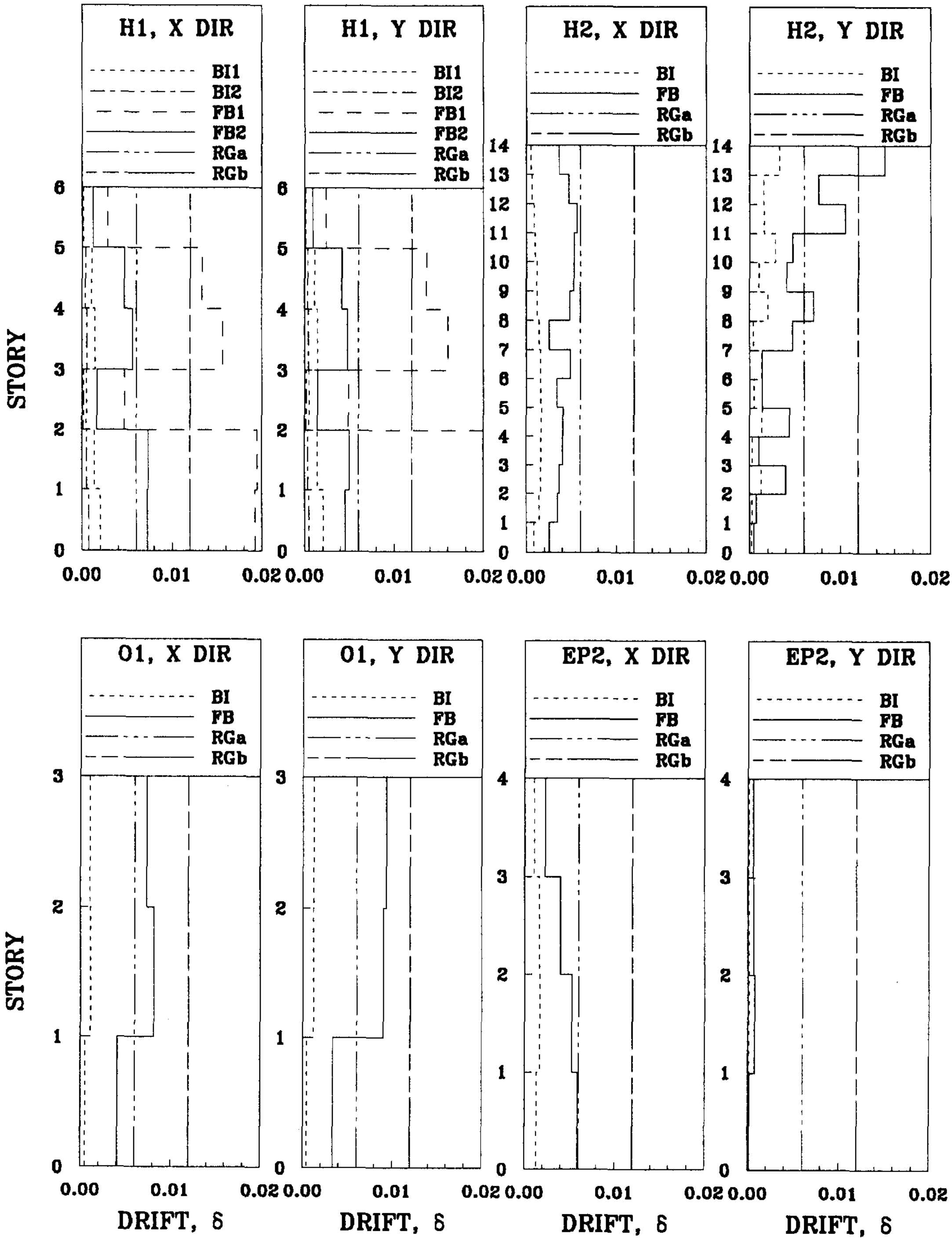


Figure 12. Maximum interstory drift angle envelopes.

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. 12, Tables 2 to 8). 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. 12). 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 at the Mexican Pacific Coast was presented, mainly LRB and SHD. 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. Subduction earthquakes are the main events considered in defining the design spectra of states of the Mexican Pacific Coast, as they have usually been the most severe events recorded and studied by seismologist in this area.

Dynamic responses of the isolated structures compared favorably against those of 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 LRB 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 LRB isolators yield and displace substantially while others respond in the elastic range. The torsional response of these 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 LRB 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 actions and/or disruption of the operation of the building, then, increasing its cost-effective long term analysis.

Finally, the present study leads the authors to the following reflection. The seismic design of the base isolators is generally controlled by the maximum allowable displacement of the isolators for dynamic stability rather than strength. Although this fact has been recognized before in the sizable literature on base isolation, the design practice on base isolators is currently based on pseudo-acceleration design spectra format. The 1994 UBC code provisions share this design philosophy. It seems logic that if an optimal design of base-isolated structures is desired, one has to look at all the required information. Pseudo-acceleration response spectra give only one third of the required information. An inelastic tripartite design spectrum (Newmark and Hall, 1982) gives all the needed information, as the nonlinear action of the base-isolated structure is concentrated on the isolators. Nevertheless, using what the authors call capacity response spectrum could be more reasonable. The inelastic design spectrum is based upon the concept of displacement ductility, that is not the best concept for base isolators. For a given excitation, a displacement ductility demand can be associated to different yield strengths, that is, this ductility is not unique. Base isolators are usually designed for a fixed yield strength and post yield stiffness. Therefore, constructing a capacity (inelastic) response spectrum will be more useful as most isolators are defined in this fashion. This capacity response spectrum will relate peak pseudo-accelerations, pseudo-velocities, pseudo-displacements and effective isolated natural periods for systems with given yield strength and post yield stiffness. Current research efforts of the authors are being directed in this direction and will be reported in the near future.

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APPENDIX I : CONVERSION TO SI UNITS

To convert	To	Multiply by
Ton	KN	9.81
kg/cm^2	Pa	0.0981

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