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PERFORMANCE CONSIDERATIONS FOR ISOLATION SYSTEMS IN REGIONS OF HIGH SEISMICITY

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Abstract

Seismic isolation systems in the United States are usually designed using ground motion characteristics that represent the postulated seismic demands of what are called ''design basis'' and ''maximum capable'' earthquakes. The design basis earthquake (DBE) spectrum is used to design the isolation system, the superstructure, and the substructure while the maximum capable earthquake (MCE) spectrum is used to check the stability of the isolation system. For conventional buildings, elastic design forces obtained from a DBE spectrum are reduced by response modification factors which are intended to account for inelastic action in the building during severe earthquakes. This design process leads to buildings which perform elastically in frequent minor earthquakes, but which suffer structural and non-structural damage in less frequent, severe events. By contrast, isolation systems conservatively designed with respect to DBE and/or MCE spectra and incorporating lower response modification factors may not respond as anticipated during minor or moderate events if the deformation response of the isolators is too low to initiate their characteristic softening at large displacements. The result is that the superstructure responds as if it were fixed-base, with amplified accelerations in the upper stories which may approach those generated in the severe MCE event. That is, if the design of an isolation system is overly conservative with respect to the most severe event, non-structural damage may occur in the upper floors of the structure during low levels of earthquake shaking. Such behavior is clearly undesirable given the higher probabilities of minor and moderate events compared to those of the DBE and MCE.

This paper presents the results of a study of an existing seismically-isolated building in Southern California which is located near the San Andreas Fault, the San Jacinto Fault, and the South Frontal Fault Zone. Results and conclusions based on analyses for three levels of earthquake shaking (minor, moderate, and severe) are presented, and recommendations are made regarding the design of seismic isolation systems for a broad range of earthquake shaking.

Introduction

Current design codes for fixed-base structures have been developed with the intent of ensuring life safety during a major earthquake. Extensive damage to the structure and non-structural components during such an event is acceptable so long as collapse is prevented. A different design philosophy is implicit in the codes for seismically-isolated buildings [SEAONC, 1986, Uniform Building Code (UBC), 1991]. The goal of these codes is to provide enhanced performance to isolated buildings in the design earthquake — performance beyond that required to ensure life safety — although some inelastic behavior may occur in the superstructure. While this approach provides superior structural performance in base-isolated buildings during infrequent large earthquakes, it does not ensure that their response to more frequent minor or moderate events will be acceptable in terms of damage to non-structural components and building contents.

The first base-isolated building in the United States, the Foothill Communities Law and Justice Center (FCLJC) in Rancho Cucamonga, California, has recently experienced several moderate earthquakes, and its recorded response in each of these events shows some amplification of accelerations throughout the height of the building [OSMS, 1990]. Although one of the main performance requirements in the design of the FCLJC was to minimize damage during the MCE — a magnitude 8.3 earthquake on the San Andreas Fault 13 miles from the site — these recent observations have raised some concern about the effectiveness of isolation systems over a broad range of earthquake shaking. To investigate the implications of these observations on the design of seismically-isolated structures in general, a computer model of the FCLJC has been developed and analyzed for three levels of ground motion representing earthquakes with magnitudes of approximately 6.0, 7.5, and 8.3 in the site vicinity.

Design Philosophy for Seismically-Isolated Buildings

Design of a seismically-isolated building according to the Uniform Building Code (UBC) [UBC, 1991] requires that the isolation system be evaluated with respect to both DBE and MCE spectra, often using spectrum-compatible ground motions. The development of these design spectra often requires site-specific ground motion studies. A DBE spectrum is typically defined by a seismic event with a return period of 475 years, while an MCE spectrum is intended to represent the strongest level of ground shaking which may be expected at the site within the known geological framework. The return period for such an event may be taken as about 2500 years in seismically active regions, although this will become approximately 1000 years in the upcoming 1994 version of the code. The isolation system, superstructure, and substructure are designed for the DBE force levels while the isolators must be shown to be stable under displacements corresponding to the MCE. The superstructure element design forces may be calculated using the DBE forces reduced by an R_{W_I} factor, ranging from 1.5 to 3.0.

Current UBC requirements for fixed-base buildings specify the use of a DBE spectrum for determining elastic design forces without any reference to a seismic demand corresponding to an MCE-level event. These elastic demands are reduced by response modification factors which are up to four times larger than those for isolated structures. There are two reasons behind the more restrictive bounds on the response modification factors for isolated buildings. The first is the desire to provide enhanced performance in isolated structures beyond that expected in fixed-base structures. This philosophy was made explicit in the commentary to the first base isolation code to appear in the United States, the Structural Engineers Association of Northern California (SEAONC) *Tentative Seismic Isolation Design Requirements*, or "Yellow Book", which was published in 1986 [SEAONC, 1986]. The second reason for more stringent limits regarding inelastic action in isolated structures is the desire to maintain a reasonable separation between the period of the superstructure and the target period of the isolation system.

An additional consideration in the design of isolated structures is the need to minimize building response under wind load. To accomplish this, the isolation system must have a relatively high initial stiffness. However, the combination of a high initial stiffness with a high effective yield force can lead to an isolation system which does not respond well in frequent, minor earthquakes. To be effective in such events, the effective yield force should generally be chosen to be of the same order as the largest wind-induced base shear.

Study Objectives

The measured response of the FCLJC during several recent minor-intensity earthquakes indicates that the characteristics of the ground motion were not sufficient to initiate the softening behavior of the isolation system. Because of this, the superstructure responded as though it were fixed-base, and accelerations were amplified up the height of the building. Although such behavior has caused some concern among observers, it is to be expected given the nonlinear force-deformation characteristic of the FCLJC isolation system. The purpose of the present study is to illustrate how the performance of isolated structures varies as a function of earthquake intensity. To this end, the measured response of the FCLJC to several past earthquakes (particularly the February 1990 Upland M5.5 earthquake) is summarized, and then a model of the superstructure and isolation system is developed and analyzed for ground motions representative of magnitude 6.0, 7.5, and 8.3 earthquakes.

Description of the FCLJC

The Foothill Communities Law and Justice Center (FCLJC) was the first base-isolated building in the United States. It was designed in 1983 and completed in 1985. It is a four-story, braced steel frame with a full basement and a mechanical penthouse at the roof level. The building is 414 ft by 110 ft in plan, and the height of the main roof above the isolation bearings is 76.5 ft. Each of the 98 columns rests on a high-damping natural rubber bearing. There are a total of eight different bearing designs. The longitudinal axis of the building is aligned east-west, and a cross section of the building looking west is provided in Figure 1.

The primary lateral-load resisting system is a series of braced frames extending from the first floor level to the roof. Transverse and longitudinal 10-inch and 14-inch thick concrete shear walls extend the full height of the basement. These walls serve to spread overturning actions in the braced frames onto the bearings. The floor diaphragms are lightweight concrete slabs on 3-inch metal decking. The foundation system is approximately 20 feet below grade and consists of individual spread footings under the bearings.

Description of Geological Framework

Before construction was started on the FCLJC, a review was made of geologic data for the Rancho Cucamonga area, and 6 borings were made to depths of up to 40 feet below existing grade. The soil profiles obtained from the borings indicate that the site is underlain by wind-blown silty sands to a depth of approximately ten feet, and below this is an alluvial mixture of sand, silty sand, gravel, and some cobbles. The basement rock is at a depth of about 800 ft, and the site period is approximately 1.5 seconds.

Ground motion hazard at the site is dominated by potential earthquakes on the San Andreas Fault, situated approximately 13 miles northeast of the site, the San Jacinto Fault, situated approximately 10 miles east of the site, and the South Frontal Fault zone (including the Sierra Madre and Cucamonga Faults), situated approximately 4 miles north of the site. During the historical time period (approximately 200 years in southern California), the FCLJC site has experienced ground shaking from numerous moderate-to-large magnitude earthquakes. At least ten of these events have produced Modified Mercalli Intensity (MMI) VI effects in the site vicinity. Investigations prior to construction of the FCLJC defined the design MCE for the site as a Richter magnitude 8.3 event on the San Andreas Fault, 13 miles from the site. This event corresponds to a motion at the site with a peak ground acceleration (PGA) of approximately 0.4g, a spectrum with constant velocity of 50 in./sec. in the period range beyond 0.8 second (Figure 4), and a strong motion duration exceeding 35 to 40 seconds.

Design Criteria

Because the FCLJC was the first base-isolated building in the United States, a number of conservative assumptions were made in the design. The owner (the County of San Bernardino) desired that the building experience only minor structural damage in what they referred to as the Maximum Probable Earthquake — the maximum event that could be expected during a period of 100 years — and that it not suffer permanent damage in the MCE [Tarics, 1984]. To satisfy this stringent performance requirement, the lateral load-resisting system was designed as a braced frame rather than a moment-resisting frame, increasing the period separation between the superstructure and the target isolation period. Furthermore, the isolators were designed to provide a period of approximately two seconds at the design displacement so that spectral acceleration demands were reduced to about one-half of those for a fixed-base moment-resisting frame. A series of nonlinear time history analyses, using three different ground motions representative of the MCE, was performed during the design phase. The results indicated an average displacement demand in the bearings of 10.6 inches, and when 5 percent accidental torsion along the longitudinal axis was considered, the peak displacement at the corner bearings was 15 inches (approximately 125 percent shear strain in the bearings). This displacement became the baseline for the verification tests of full-size bearings, and was used to set the seismic gap around the building to 16 inches.

Description of Isolation System

The building is supported on 98 high-damping natural rubber bearings, and there are eight different bearing types incorporated in the design (denoted A through H). All of the bearings are 30 inches in diameter, and the total height of rubber in designs B through H is 11.47 inches. The rubber height in the type A bearings is 11.97 inches. The bearings possess substantial inherent damping, which varies from about 9 to 18 percent depending on the level of deformation in the isolators. Connection of the bearings to the foundation and superstructure is by steel pin shear dowels into the bearing end plates. A view of one of the bearings and the adjacent fail-safe stubs during construction is shown in Figure 2.

Four different high-damping rubber compounds were used in the manufacture of the isolators. The compounds are designated by the supplier, LTV Energy Products, Arlington, Texas (in order of decreasing shear modulus), as 246-70, 243-65, 2X-69, 2X-71. All of these compounds have a highly nonlinear stress-strain behavior that is particularly advantageous for seismic isolation systems. The materials are very stiff at small shear deformations. The effective modulus decreases as the strain increases up to about 100 percent, and for shear deformations beyond 100 percent it stiffens up again. Thus, the system is stiff for wind loads, environmental disturbance, and small earthquakes, and it softens for large earthquakes providing a long-period isolation effect. Although the rubber compounds stiffen at large shear strains, this behavior cannot be realized in the FCLJC bearings because of their doweled connections.

An important characteristic of elastomers which should be considered when estimating the response of structures incorporating elastomeric bearings is the material behavior known as ''scragging''. This is a reversible process in which the stiffness of an undeformed rubber compound is greater in the first cycle of loading than in subsequent cycles. An isolation system which has been undisturbed for a long period of time will exhibit a higher initial stiffness than assumed in a design using the scragged rubber properties. Such an effect is especially important in minor earthquakes in which the bearings respond primarily in the small strain (high stiffness) region while also exhibiting unscragged stiffnesses.

The damping ratio is another parameter which is significant in evaluating high-damping (filled) natural rubber compounds, because of its dependence on shear strain. It is typical of high-damping natural rubbers that the damping increases with decreasing shear strain. In the compounds used for the FCLJC bearings, the equivalent viscous damping decreases from 13—18 percent of critical at 2 percent strain to 8—10 percent of critical at 100 percent strain. A complete summary of the stiffnesses and damping ratios of the FCLJC compounds is given in [Tarics, 1984].

Each isolator installed in the FCLJC was tested by the manufacturer at shear strain levels of 2, 10, 25, and 50 percent [Tarics, 1984] to assure satisfactory bearing performance at the design level strain. Because one cycle at 50 percent was considered by the designers as sufficient proof of the integrity of each bearing, only first-cycle (unscragged) stiffnesses were obtained. A detailed summary of this data is given in [Tarics, 1984]. In addition to these moderate strain tests, four of the prototype bearings (design A) were tested to a displacement of 15 inches to verify MCE stability [Kelly and Celebi, 1984].

Instrumentation and Recorded Response in Previous Earthquakes

The FCLJC is instrumented with 16 accelerometers (both in the structure and at the foundation level), and 3 free-field accelerometers are located 350 ft southeast of the building. These accelerometers are arranged to record longitudinal, transverse, vertical, and torsional motions at the foundation, basement, and roof. One accelerometer on the second floor measures only transverse motion. Additional details of the instrumentation plan are given in [OSMS, 1986a].

Since its completion in 1985, motions from seven moderate-to-large earthquakes have been recorded at the FCLJC site [OSMS, 1986a-b, 1987, 1990, 1991, 1992a-b]. A complete listing of these earthquakes and a summary of the building responses in the north-south (transverse) direction is given in Table 1. The largest free-field PGA levels (≈0.25g) were recorded during the M5.5 Upland earthquake that occurred on 28 February 1990, centered approximately 7.5 miles west of the FCLJC site. Transverse ground accelerations recorded at the foundation level, approximately 20 ft below the ground surface, were 0.6 times the recorded free-field ground surface motion. Similar reductions due to embedment were seen in the other records. During the Upland event the isolation system reduced the transverse accelerations transmitted to the basement level from the foundation by 62 percent. The peak roof acceleration was approximately 2.9 times that in the basement, indicating significant response in the superstructure. Nonetheless, the roof-foundation peak acceleration ratio reflects an amplification of only 10 percent. Analyses have shown that the period of the isolated building during this event was approximately 0.75 second [Aiken et al., 1991, Maison and Ventura, 1992]. During all of the other earthquakes the free-field PGAs were approximately 0.12g or less, but each led to some amplification of accelerations over the height of the superstructure. Peak transverse accelerations recorded in the building during all of these earthquakes, normalized with respect to PGAs, are given in Figure 3.

Selection of Ground Motions for Analyses

Site-specific analyses were conducted to assess the characteristics of free-field ground motions expected at the FCLJC site during moderate-to-large earthquakes in southern California. Both probabilistic and deterministic approaches were used to develop smooth response spectra that were used as a basis for selecting ground motions for analyses of the structure. The probabilistic analysis described here (commonly termed a "seismic hazard analysis") was based on an assessment of the recurrence of earthquakes on potential seismic sources adjacent to the site and on ground motion attenuation relationships appropriate for the types of regional seismic sources and the subsurface conditions at the FCLJC site. The attenuation relationships developed by [Sadigh et al., 1986] for deep, firm soil sites were used in this study. Results of this analysis are expressed as hazard relationships between amplitudes of PGA and pseudo-acceleration (PSa) and the annual frequencies or return periods (return periods being the reciprocal of annual frequency for exceedance of those amplitudes). Smooth uniform-hazard response spectra were developed for free-field ground motions having 5- and 10-percent probabilities of exceedance in 50 years (975 and 475 year return periods, respectively) using values of PGA and PSa corresponding to the desired hazard level obtained from the suite of hazard results. The smooth uniform-hazard free-field response spectra for the two probability levels are illustrated in Figure 4.

The ground motions selected for the analytical study of the FCLJC were based on the above-determined probabilistic spectra and also considered the effect of earthquake magnitude on strong motion duration. Empirically-derived duration relationships presented by [Dobry et al., 1978] and [Bullen and Bolt, 1985] indicate that strong shaking duration for an event in the M7 to 7.5 range may vary between about 15 and 25 seconds and for an M8+ event between about 30 and 45 seconds. The Caleta de Campos recording of the 1985 M8.1 Michoacan earthquake (amplitude-scaled by 3.0) was selected to be representative of a magnitude 8 or greater event on the San Andreas Fault with a return period of approximately 1000 years because its duration of strong shaking is approximately 35 seconds, and its scaled response spectral characteristics reasonably match the shape of the smoothed probabilistic spectrum in the long period (greater than 0.5 second) range. The PGA is slightly lower than that specified for the 975 year return period event, but this is considered acceptable in light of the embedment of the foundation at the FCLJC. The Joshua Tree Fire Station recording of the 1992 M7.5 Landers earthquake (amplitude-scaled by 1.15) was selected as representative of the 475 year return period event. This record contains approximately 25 to 30 seconds of strong ground motion. One other record was selected to represent a more frequent earthquake of smaller magnitude: the recording of the 1986 M5.9 Palm Springs earthquake obtained at Desert Hot Springs (amplitude-scaled by 0.3). The FCLJC foundation record obtained during the 1990 M5.5 Upland earthquake was used unscaled to correlate the computer model. Both horizontal components from each of these records were used in the analyses; in each case the component with the largest PGA was applied in the transverse direction of the building. Table 2 summarizes the details of each of the records, and plots of the response spectra of the transverse components (scaled to account for magnitude and the effect of foundation embedment) are given in Figure 5

Analytical Model of FCLJC

A three-dimensional numerical model of the FCLJC, assuming an elastic superstructure, was developed. The beams and bracing elements in the braced steel frames were modeled by truss elements, and the columns were modeled by beam-column elements. The 14-ft high shear walls at the basement level were modeled by struts with lateral stiffnesses equivalent to those of the concrete walls. Rigid diaphragms were assumed at each floor level. To minimize the effects of torsion, the story masses were lumped at the center of each floor. The mass values used were the same as those assumed for the original design of the building. The first fifteen modes of the fixed-base superstructure were calculated using the program SADSAP [Wilson, 1992]. The periods of the first three fixed-base modes corresponding to transverse and longitudinal translation and torsion are 0.55, 0.53, and 0.45 second, respectively. These periods are similar to those determined from the response of the building during the Redlands earthquake [Papageorgiou and Lin, 1989].

The computer program 3D-BASIS was used for all of the nonlinear time-history analyses in this study [Nagarajaiah et al., 1991]. The fixed-base frequencies and mode shapes of the superstructure were combined with bilinear elements representing the elastomeric bearings. The properties of the bearing elements were selected to match force-displacement relationships of the unscragged bearings derived from the production bearing tests and described by [Tarics, 1984]. Bilinear models were developed for each of the eight bearing types, and these were appropriately adjusted to account for input intensity so that an accurate representation of bearing response was obtained over the range of inputs. A comparison between the analytical behavior of a type E bearing during the Joshua Tree Fire Station analysis and the force-displacement relationship measured in prototype tests of the same bearing type is shown in Figure 6. The damping in the isolation system model was assumed purely hysteretic, except in the Upland correlative analysis described below. Modal damping of 3 percent was assumed for all superstructure modes.

Analysis Results

Table 3 summarizes the results of analyses for the unscaled Upland record as well as the scaled Desert Hot Springs, Joshua Tree Fire Station, and Caleta de Campos records. The first portion of the analytical study involved correlating the numerical model with the building responses recorded during the Upland earthquake. This was accomplished by using linear elastic elements for the bearings with stiffnesses 35 percent larger than the 2 percent strain stiffnesses reported by [Tarics, 1984], and incorporating an equivalent viscous damping ratio of 18 percent for the isolation system. The increased stiffnesses are appropriate because the displacement response of the bearings at the center of the building was somewhat less than 2 percent strain, although bearings along the east and west walls of the building did deform to approximately 2 percent strain due to torsion. Figure 7 shows that the recorded and calculated transverse roof accelerations compare well.

Figure 8 shows the distribution of accelerations (normalized with respect to PGA) for each of the analyses, as well as for three of the recorded events (Upland, Whittier, and Palm Springs). Figure 9 presents the same results in terms of absolute accelerations. It should be noted that an acceleration profile in a fixed-base building would show increasing accelerations up the height of the building. However, only the profiles derived from recordings of the Palm Springs and Whittier earthquakes display such a trend — these were two of the smallest events recorded at the site, each with a PGA of less than 0.02g. All of the other profiles display significant reductions of acceleration from the foundation to the basement level, although some amplification is evident from the basement to the roof. The absolute acceleration profiles from the Joshua Tree Fire Station and Caleta de Campos analyses show significant increases in the top two floors due to higher-mode effects in the superstructure. The normalized accelerations tend to decrease with increasing ground motion intensity — the scaled Desert Hot Springs record leads to a normalized acceleration at the roof of 1.26, while the scaled Caleta de Campos record gives a normalized roof acceleration of only 0.76.

Conclusions

This study has examined the response of an existing seismically-isolated building to earthquakes with a wide range of intensities. The results from previously recorded minor earthquakes and the time-history analysis using the scaled Desert Hot Springs motion (PGA $\approx$ 0.09g) indicate that for earthquake motions with small PGAs the isolation system is only partially effective in limiting the accelerations transmitted to the superstructure. In these minor events the upper floors of the building exhibit amplified accelerations, which could result in damage to nonstructural components. However, as the PGA of the input motion increases, the isolation effect becomes more pronounced, and the roof-ground acceleration amplification ratio is reduced, although there may still be some amplification between the basement and the roof level due to higher-mode effects.

Several recommendations follow from the results of this study. First, severe events such as the DBE (and MCE) should continue to form the basis for the design of the superstructure and the bearings. If requirements are placed on the performance of equipment and other non-structural components in more frequent minor events, then additional checks on the response of the isolated system should be made for lower intensities of shaking. Second, a triangular force distribution is more appropriate than a rectangular distribution for designing the superstructure because it takes into account higher-mode effects which are present even in large events, unless the stiffness of the superstructure is very high. Finally, the characteristics of the ground motion expected at the site must be thoroughly evaluated with respect to PGA, frequency content, and duration of strong motion, because larger magnitude earthquakes typically have more energy in the long period range of the spectrum (beyond about 1 second), as well as longer strong motion durations. The analyses presented have incorporated these effects and indicate that the best performance for a range of input intensities is achieved by an isolation system which has a relatively low effective yield point in combination with increasing stiffness at large displacements.

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Table 1: FCLJC Transverse Direction (N/S) Peak Accelerations Recorded in Past Earthquakes

EARTHQUAKE	DATE	MAG. (M_L)	DIST. (miles)	ACCELERATION (g)				
				Free-Field	Below Brgs†	Above Brgs†	2nd Floor†	Roof†
REDLANDS	10.2.85	4.8	19	0.045	0.032	0.013	0.018	0.029
PALM SPRINGS	7.8.86	5.9	56	0.021	0.014	0.018	0.021	0.041
WHITTIER	10.1.87	6.1	29	0.046	0.019	0.023	0.025	0.049
UPLAND	2.28.90	5.5	7.5	0.240	0.142	0.054	0.071	0.156
SIERRA MADRE*	6.28.91	5.8	27	0.04	0.02	0.03	0.04	0.08
LANDERS*	6.28.92	7.5 (M_S)	82	0.12	0.10	0.09	0.08	0.18
BIG BEAR*	6.28.92	6.5	44	0.04	0.03	0.04	0.04	0.07

† acceleration recorded at the center of building
* peak accelerations determined from uncorrected data

Table 2: Earthquake Records Used for Analyses

EARTHQUAKE	DATE	MAGNITUDE	STATION	DIST. (miles)	COMP.	UNSCALED PGA (g)
UPLAND	2.28.90	M_L=5.5	FCLJC (foundation)	7.5	N00E N90E	0.141 0.108
PALM SPRINGS	7.8.86	M_L=5.9	Desert Hot Springs	7.5	N00E N90E	0.300 0.269
LANDERS	6.28.92	M_S=7.5	Joshua Tree Fire Station	9	N360E N90E	0.273 0.284
MICHOACAN	9.19.85	M_S=8.1	Caleta de Campos	14	N00E N90E	0.141 0.140

Table 3: Summary of Analytical Results

EARTHQUAKE RECORD	SCALE FACTOR	SCALED PGA (g)	$\ddot{v}_{rf,\,max}$ (g)	$\frac{\ddot{v}_{rf,\,max}}{PGA}$	$V_{b,max}$ (%W)	D_{max} (In.)	γ_{max} (%)
UPLAND	1.0	0.14	0.13	0.94	6.1	0.26	2
DESERT HOT SPRINGS	0.3	0.09	0.11	1.26	8.2	1.04	9
JOSHUA TREE FIRE STATION	1.15	0.33	0.34	1.03	18.4	5.42	45
CALETA DE CAMPOS	3.0	0.42	0.32	0.76	24.7	10.95	92

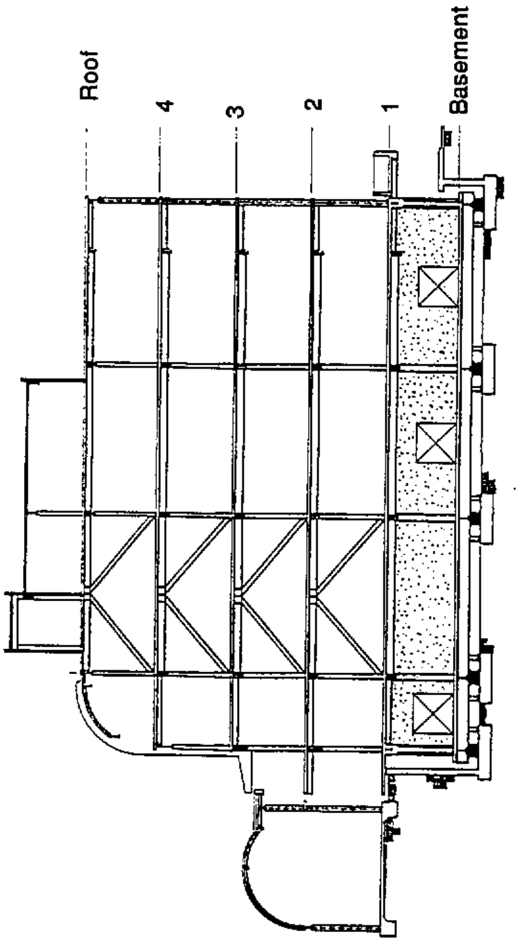

Figure 1: FCLJC North/South Cross-Section

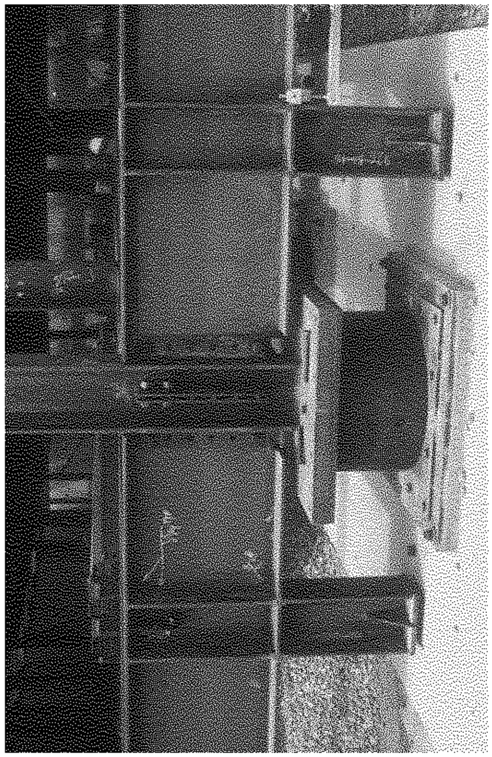

Figure 2: View of Isolation Bearing During Construction

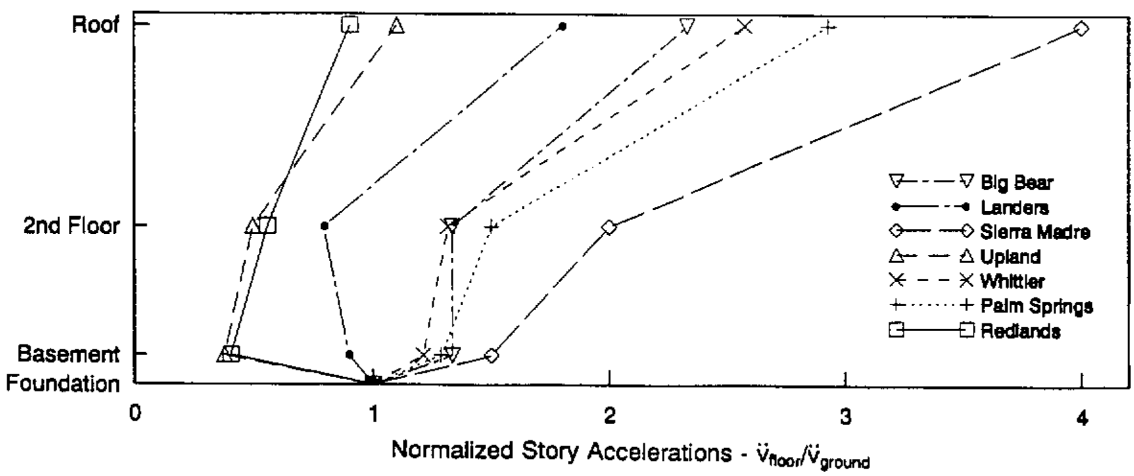

Figure 3: Peak Acceleration Responses in Past Earthquakes

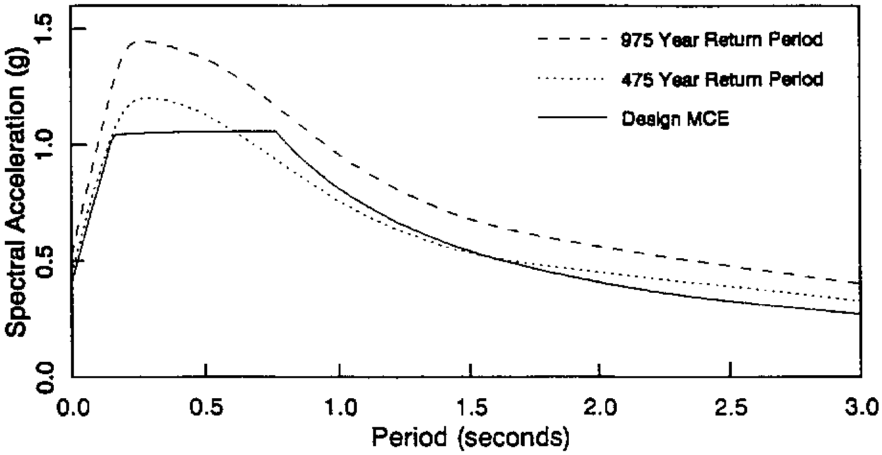

Figure 4: Probabilistic Site-Specific Response Spectra

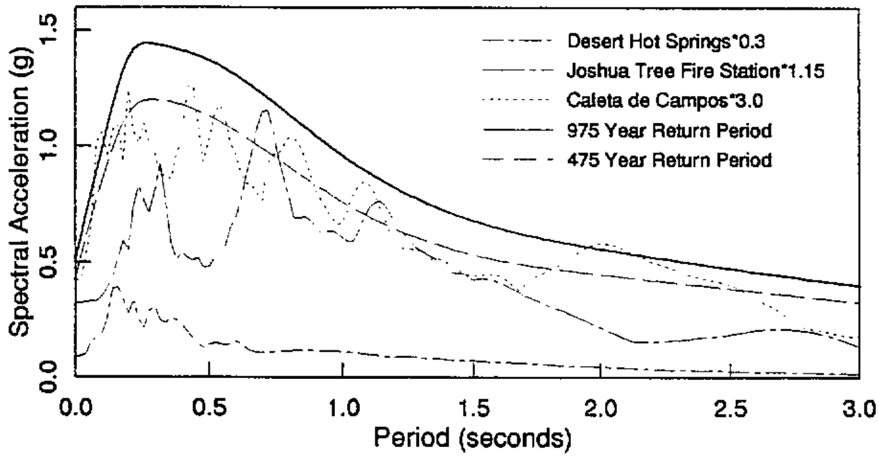

Figure 5: Response Spectra for Site-Specific Ground Motions

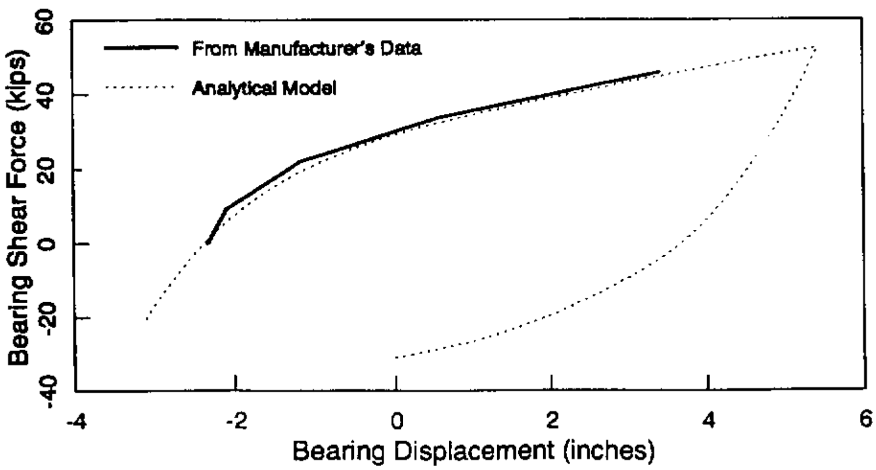

Figure 6: Comparison of Bearing Analytical Model and Actual Force-Displacement Curve (From Manufacturer's Test Data, [Tarics, 1984].

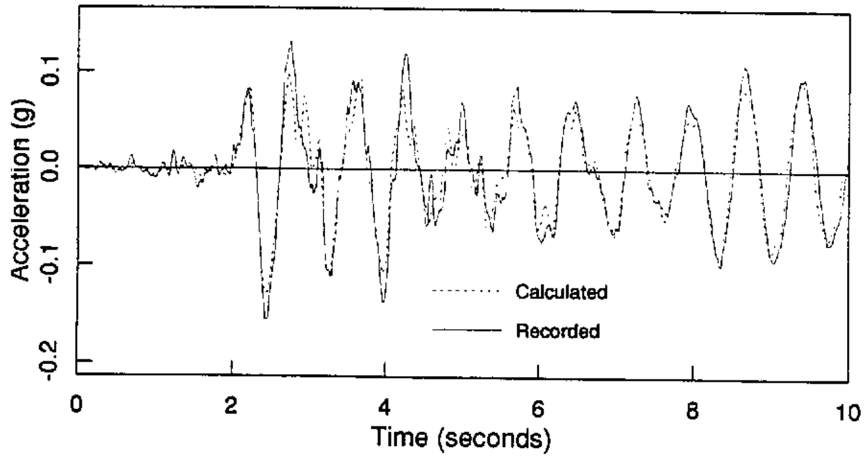

Figure 7: Comparison of Recorded and Calculated Roof Transverse Accelerations for Upland

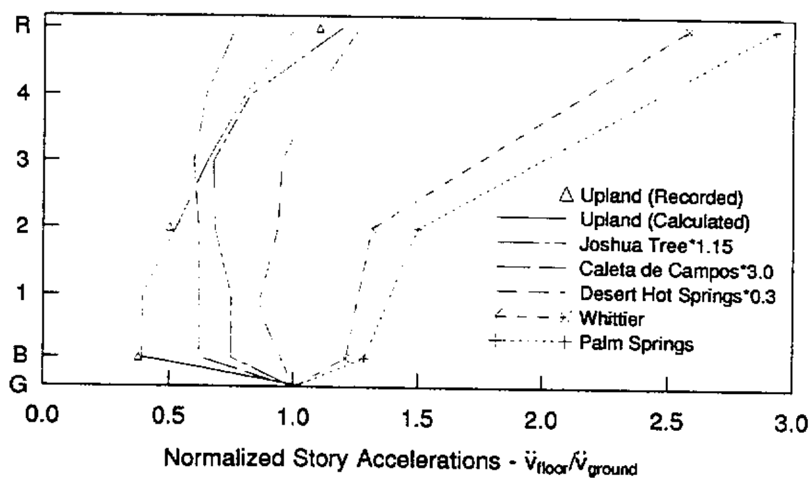

Figure 8: Normalized Accelerations - Calculated and Recorded

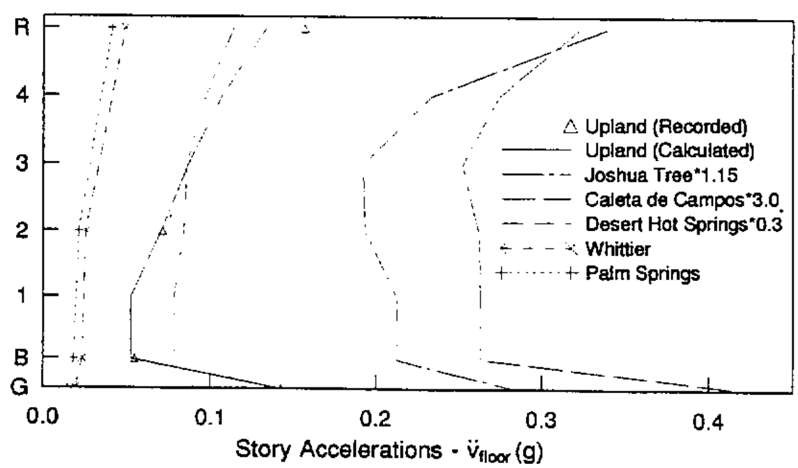

Figure 9: Absolute Accelerations - Calculated and Recorded