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POTENTIAL COLLABORATIVE RESEARCH ON THE DESIGN OF SEISMIC ISOLATED BUILDINGS CONSIDERING MULTIPLE PERFORMANCE OBJECTIVES

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ABSTRACT

In the past 20 years, seismic isolation has see a variety of applications in design of structures to mitigate seismic hazard. In particular, isolation has been seen as a means of achieving enhanced seismic performance objectives, such as those for hospitals, critical emergency response facilities, mass electronic data storage centers, and similar buildings whose functionality following a major seismic event is either critical to the public's welfare or the financial solvency of an organization. While achieving these enhanced performance objectives is a natural (and oftentimes requisite) application of seismic isolation, little attention has been given to the extension of current design practice to isolated buildings which may have more conventional performance objectives. The development of a rational design methodology for isolated buildings requires thorough investigation of the behavior of isolated structures subjected to seismic input of various recurrence intervals, and which are designed to remain elastic only under frequent events. This paper summarizes these investigations, and proposed a consistent probabilistic framework within which any combination of performance objectives may be met. Analytical simulations are presented, the results are summarized. The intent of this work is to allow a building owner to make informed decisions regarding tradeoffs between superstructure performance (drifts, accelerations) and isolation system performance. Within this framework, it is possible to realize the benefits of designing isolated buildings for which the design criteria allows consideration of multiple performance objectives.

Keywords: Seismic isolation, performance-based design, drift, floor spectra

INTRODUCTION

Current provisions governing seismic design of buildings contain implicit performance objectives, which differ for fixed-base and base-isolated buildings. While fixed-base buildings are intended to experience significant inelastic response and damage in a major earthquake, isolated buildings, although designed to a force reduction factor up to $R_I = 2$, remain essentially elastic in the design basis earthquake due to overstrength. The implicit performance objectives assumed in the code are that fixed-base buildings are to be designed for life safety while isolated buildings are designed for continued occupancy. As a result, base-isolation of new buildings has been essentially limited to critical facilities whose operation following a major seismic event is crucial to the public welfare.

A major effort is underway in the U.S. to transform prescriptive design codes to flexible guidelines controlled by owner-defined performance objectives. When this performance-based methodology has matured, a building owner will be able to evaluate base isolation and fixed-base design as potential alternatives to achieve a given performance objective. To be consistent with this framework, design requirements for isolated buildings should be modified to allow the selection of a performance objective and a suitable superstructure strength to achieve acceptable drift and ductility demands, consistent with the approach for conventional fixed-base buildings. This approach may have benefits

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for isolated buildings such as reduced construction costs or enhanced performance at similar cost (e.g. reduced superstructure acceleration, therefore limiting nonstructural damage). However, major changes in the design approach for isolated buildings seem premature since very little research has been conducted to understand the behavior of isolated buildings with inelastic superstructure response.

The purpose of this study is to investigate the design of a multi-story seismic isolated building to meet three discrete performance objectives. The same building is designed as fixed-base, targeting the same performance objectives. The performance of these buildings subjected to an ensemble of ground motions is evaluated, and the feasibility of effectively applying isolation to meet the various performance objectives is investigated.

Statistical Analysis

In this study, the seismic response is characterized by statistical analysis of the results of an ensemble of response history analyses. Because of this, it is necessary to define the descriptive statistics for a set of data. For n observed values x_i, the median and the dispersion are defined in Equation (1) and (2), respectively:

$$\hat{x} = \exp\left[\frac{\Sigma_{i=1}^{n} \ln x_i}{n}\right] \qquad \delta = \left[\frac{\Sigma_{i=1}^{n} (\ln x_i - \ln \hat{x})^2}{n-1}\right]^{1/2} \qquad (1), (2)$$

These definitions are appropriate for data which is lognormally distributed, a common assumption for seismic response parameters (McGuire, 2004). It is noteworthy that, for small values of δ (less than 0.3), the dispersion closely approximates the coefficient of variation, otherwise defined as the ratio of the standard deviation to the mean.

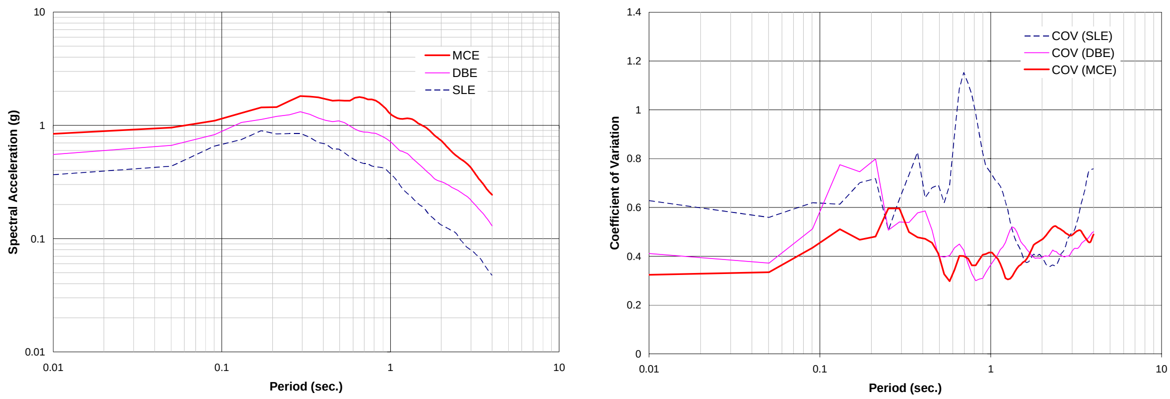
CHARACTERIZATION OF SEISMIC HAZARD

In any study focused on performance-based seismic design, a careful definition of seismic hazard is important since the expectations for seismic performance depend on the severity of the event being considered. For any site, a set of hazard curves may be constructed which relate some spectral parameter (say spectral acceleration at period T_i) to the mean annual frequency (MAF) of occurrence. This MAF may be inverted to represent the return period of the event, T_R. In this study, three return period events are considered, shown below in Table 1.

Table 1: Description of seismic hazard levels considered in this study

Seismic Hazard Level	Return Period
Service Level Event (SLE)	72 years
Design Basis Event (DBE)	475 years
Maximum Considered Event (MCE)	2475 years

As part of the FEMA/SAC Steel Program, suites of acceleration records were developed for a site in Los Angeles corresponding to these three return periods. The suite for each of the three hazard levels above contains 20 records, for a total of 60 records. A complete description of the development of these acceleration records can be found in Somerville et al. (1998.) Figure 1 below shows the (a) median and (b) coefficient of variation of the acceleration response spectra for each of the three hazard levels. These spectra are useful in interpreting statistical results later in this study.



(a) Median (b) Coefficient of variation

Figure 1: Response spectra of 20 ground motion spectra (5% damped) for each of the three hazard levels, described in terms of (a) median and (b) coefficient of variation

SEISMIC PERFORMANCE OBJECTIVES

In the process of performance-based seismic design, the target performance objectives for the facility under consideration must be defined. These performance objectives are typically a function of the type of facility, its value to the owner, its role in public safety, and the risk adversity of the facility stakeholders and decision-makers. It is a challenge to satisfy multiple performance goals which may not be simultaneously met (i.e. limiting both drift and acceleration), and as a result, engineers must often target a specific limit at a specific level of seismic hazard level. US building codes employ an inter-story drift limit at the DBE as a means of controlling earthquake-induced damage. Following this approach, a set of three Seismic Performance Classes have been defined, and are described in Table 2. These definitions roughly follow those of Seismic Use Groups defined in FEMA 450 (BSSC, 2003), however the actual drift limits vary for essential facilities because of the low yield drift being considered in this study (which is indeed valid for braced-frame and other stiff systems.) This issue is elaborated upon in the following section.

Table 2: Description of seismic performance classes considered in this study

Seismic Performance Class	DBE Target Drift Ratio	Ductility Demand	Seismic Performance Description
SPC - I	0.025	10	Basic level of seismic protection implicit in US building codes. Life safety, but no damage protection intended
SPC - II	0.01	4	Enhanced level of seismic protection, operational following DBE, however not all building systems may be functioning
SPC - III	0.005	2	Superior level of seismic protection, fully functional following DBE, appropriate for facilities critical to post-earthquake recovery

DESCRIPTION OF ANALYTICAL MODEL

A generic three-story building has been considered in this study, modeled as a series of lumped masses, interconnected by a nonlinear spring intended to model the stiffness and strength at each story. For the isolated building case, an additional lumped mass is considered at the isolation interface, and an additional nonlinear spring is present to represent the isolation system. Both the fixed-base and base isolated models are shown diagrammatically below in Figure 2. From this figure, it should be clear that m_i represents the mass at the i^{th} story, and R_i represents the resisting force at the i^{th} story, and is generally nonlinear with respect to deformation across the story

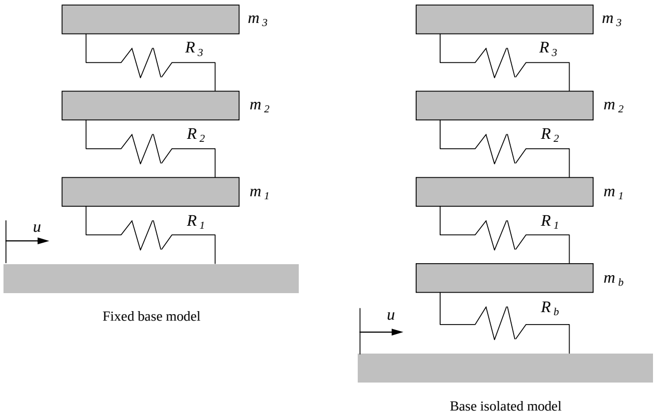


Figure 2: Analytical models for fixed-base (left) and isolated (right) buildings

The masses in the building depend on the gravity loads and the tributary area to frames in the lateral system, and can be considered arbitrarily, however they would likely be nearly identical at each level for typical multistory buildings. The resisting force due to the presence of lateral elements is described by the Bouc-Wen hysteretic constitutive relationship (Wen, 1976). This model is appropriate for both superstructural yielding elements and seismic isolation devices. The time-varying resisting force $R_i(t)$, subjected to an inter-story deformation $u_i(t)$, follows the equation below

$$R_i(t) = \alpha K_{s,i} u_i(t) + (1-\alpha) R_{y,i} z(t) \tag{3}$$

where the hysteretic parameter z follows the differential equation:

$$\dot{z}(t) = \frac{K_s}{R_y}\left[\dot{u}(t) - \gamma\left|\dot{u}(t)\right| z \left|z\right|^{n-1} - \beta \dot{u}(t)\left|z\right|^{n}\right], \; z \in [-1,1] \tag{4}$$

In these equations, K_s is the elastic story stiffness, R_y is the story yield force, and α is the ratio of post-elastic story stiffness to elastic story stiffness. It is convenient to define a normalized story strength, $\bar{R}_{y,i}$, which approximates the spectral acceleration at which yielding in a story occurs. This normalized story strength (for n stories) is defined as:

$$\bar{R}_{y,i} = \frac{R_{y,i}}{\sum_{j=i}^{n} m_j g} \tag{5}$$

Since the objective is the comparison of identical buildings with and without base isolation, the results could be considered applicable for a variety of structural systems. However, to fix ideas, a superstructural lateral system of buckling-restrained braced frames (BRBF) is considered here. The assumption of BRBFs in this study has the advantage of inter-story hysteretic relationships which are simple, stable (i.e. non-degrading in strength or stiffness), and symmetric. Indeed, BRBFs are among the most reliable lateral force-resisting systems available for conventional steel buildings, and therefore provide an interesting basis for comparison in this study.

There are several analytical issues related to the selection of BRBFs for the superstructure. First, the strength and stiffness of a particular story are perfectly coupled. Given a typical story height, h, and a typical bay width, b, equations for both the story stiffness K_s, and the story strength, R_y, may be derived in terms of the brace core-plate area, A_{cp}. From the geometry of a single diagonal braced bay, the story stiffness and story strengths can be described by the equations below:

$$K_s = \frac{E_s A_{cp} b^2}{\beta\left(b^2 + h^2\right)^{3/2}} \qquad R_y = \frac{A_{cp}\sigma_y b}{\left(b^2 + h^2\right)^{1/2}} \qquad (6), (7)$$

Where E_s is the Young's modulus of steel, σ_y is the yield stress of the core-plate material, and β is a modification factor to account for the yielding length of a BRB which is less than the work-point length (i.e. $L_{eff} = \beta L_{wp}$) and therefore increases the effective axial stiffness. By combining (6) and (7) above, we get an equation for the story strength directly in terms of the story stiffness:

$$R_y = \left[\frac{\beta \sigma_y \left(b^2 + h^2 \right)}{E_s b} \right] K_s \tag{8}$$

The term in brackets is a function of either material properties or geometric assumptions, and is therefore constant for both the fixed-base and isolated buildings. This formulation is convenient because the stiffness of each story may be computed considering drift limitations alone, and the strength may then be incorporated for use with nonlinear response-history analysis to compute statistics of engineering demand parameters such as ductility demand, inter-story drift and floor acceleration.

One consequence of selecting BRBFs for this study is the invariant relationship between inter-story drift and ductility demand. Since the yield shear R_y and the initial stiffness K_s are related by the constant shown in Equation 8 above, the yield drift is that same constant, and is independent of strength. Therefore, the ductility demand is simply a linear multiple of the drift. For typical bay widths and story heights, this yield drift ratio is about 0.0025. Therefore a drift demand of 0.025 corresponds to a ductility demand in the brace of 10. This fact has been incorporated in the ductility limits of Table 2 above.

For simplicity, one isolation system has been considered, having a nominal (post-elastic) period of 4 seconds and a yield strength of 0.07 times the supported weight. These parameters are appropriate or modeling either a lead-rubber or friction pendulum isolation system.

ANALYTICAL RESULTS

The demand parameters of principal interest in this study are peak inter-story drift ratios and floor response spectra. These are the demand parameters generally associated with building damage and economic losses. One parameter not considered in this study, but of great importance, is residual story drift. This will be addressed in future work on this topic.

Table 3 below shows the values of normalized story strength (previously defined in Eq. 5) for all structural systems described in this study. These designs are based on achieving the drift limits stipulated in Table 2 above. Also included, for reference, is the fundamental period (T_1) and effective R-factor (R_{eff}), defined as the ratio of the elastic base shear at the fundamental period to the yield base shear.

Table 3: Normalized design story strengths, $\bar{R}_{y,i}$, for both fixed-base and base isolated structures

	Fixed-base			Base isolated		
Floor	SPC-I	SPC-II	SPC-III	SPC-I	SPC-II	SPC-III
3	0.10	0.35	0.83	0.07	0.09	0.17
2	0.08	0.30	0.71	0.06	0.08	0.13
1	0.08	0.29	0.64	0.05	0.10	0.13
T_1	1.09	0.56	0.37	3.65	3.65	3.65
R_{eff}	8.2	3.4	1.8	2.9	1.6	1.2

Inter-story Drift Demands

An important consideration in the design of base isolated buildings is whether meeting a specified drift limit at the DBE will lead to either an increased drift in the MCE or an increased uncertainty in the level of drift under any particular level of hazard. If base isolated buildings are to be designed to conventional performance objectives, it must be established that the performance is not appreciably less desirable than its fixed base counterpart. Figure 3 summarizes the computed *first floor* drift demands from an ensemble of nonlinear response history analyses (NLRHA.) Results from the second and third floors are similar, and have been omitted for the purpose of clarity. Recall that the designation SPC-I, II, or III refers to the target performance level from Table 2 above, and this designation establishes equivalence for the purposes of this comparison. The data of Figure 3 indicates that there is very little statistical difference between the drift demands for the fixed-base and base-isolated buildings.

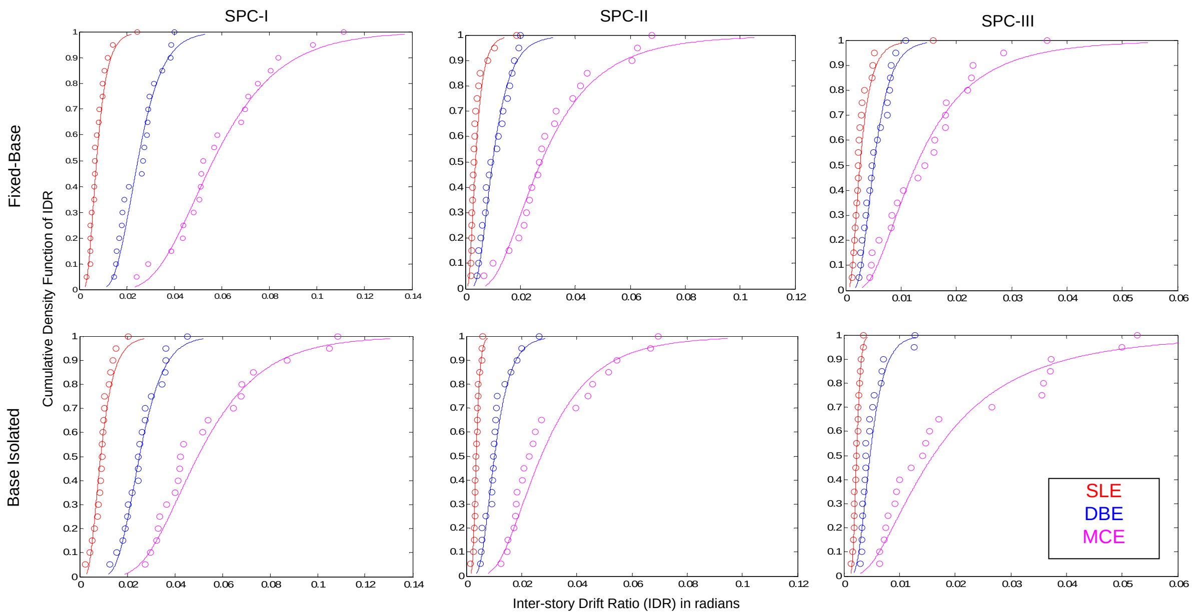


Figure 3: Summary of statistical data for first floor inter-story drift ratios considering three performance levels for both fixed-base and base isolated buildings. NLRHA data shown with open circles, fitted lognormal CDF shown solid

Figure 4 shows the data for all statistical descriptors of drift demand, including *all* floors and *all* hazard levels. Each data point corresponds to a particular ensemble of analyses, and a point lying on the superimposed dotted line indicates that the median or coefficient-of-variation is equal for both the fixed-base and base isolated buildings. This data indicates there is very little systematic difference between the two cases, and therefore no disadvantages are seen with respect to the drift-based performance of the isolated building designed to meet a range of performance goals.

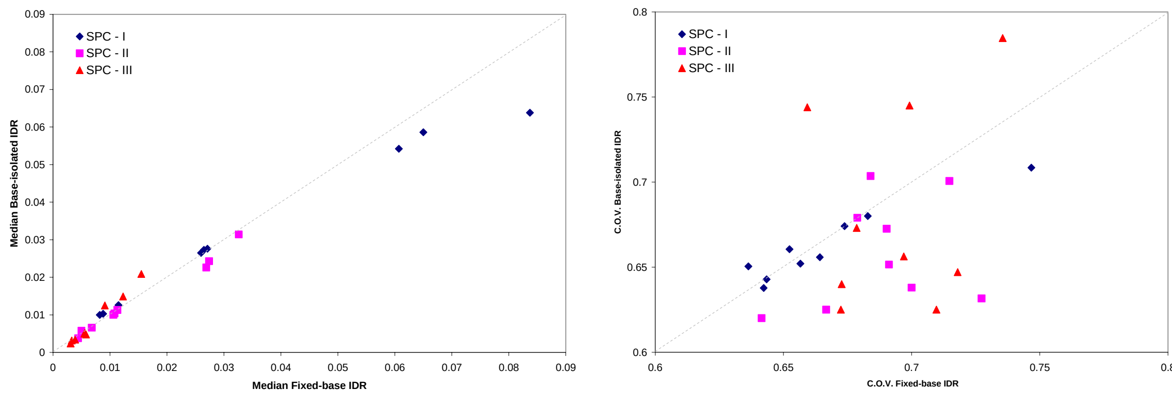
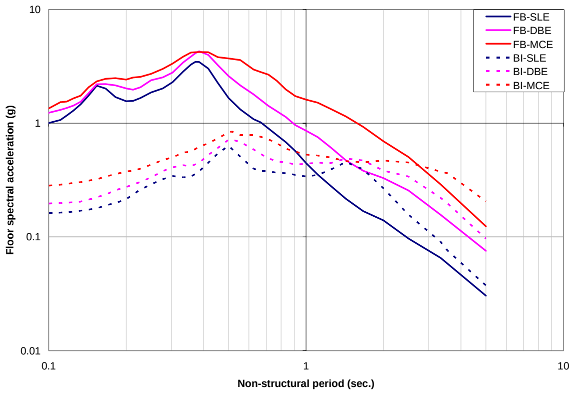


Figure 4: Comparison of inter-story drift ratio (IDR) for fixed-base and base isolated buildings in terms of: median (left) and coefficient of variation (right)

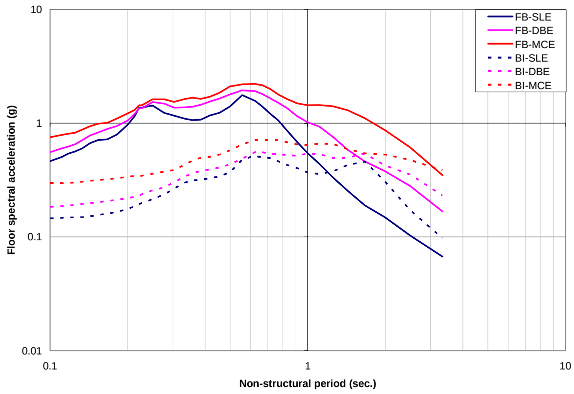
Floor Spectra

A second important demand parameter is the floor acceleration at a particular level. The magnitude and frequency content of a floor acceleration record will impact both the anchorage requirements for fixed equipment and cladding, and will impact the performance of acceleration-sensitive and free-standing equipment and contents.

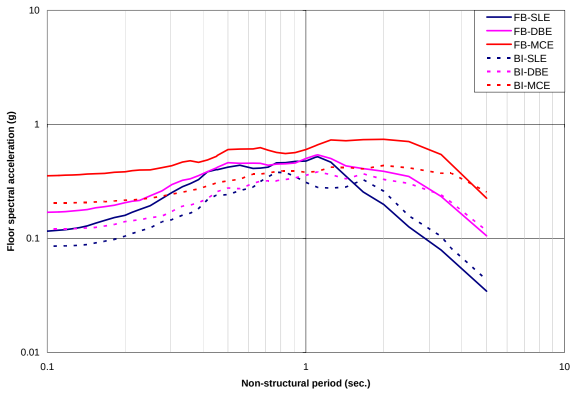
Figure 5 below shows the median *roof* acceleration spectra at the three levels of seismic hazard for all three performance classes, and includes results from both fixed-base and base isolated buildings. For the highest performance objective SPC-III, there is a significant difference between the roof spectral accelerations, particularly in the high-frequency range where most non-structural components are likely to respond. The difference between the two become less significant as the target performance objective decreases. This is an expected result, since the strength of the fixed-base system is decreasing significantly, while the strength of the base isolated building is decreasing modestly.



(a) SPC-III



(b) SPC-II



(c) SPC-I

Figure 5: Comparison of roof acceleration spectra for fixed-base (FB) and base-isolated (BI) buildings for three seismic performance classes: (a) SPC-III, (b) SPC-II, and (c) SPC-I

Combined Performance Measure (CPM)

It is desirable to define some performance index that will provide a means of comparing two structural systems in a meaningful way. A simple index for the assessment of the performance of base isolated buildings relative to fixed-base buildings is described in Ryan et al (2006), and uses a combined performance measure (CPM) defined as:

$$CPM = \left[w_1 \left(\frac{u_{m,iso} - u_{m,fb}}{u_{m,fb}} \right) + w_2 \left(\frac{\ddot{u}^t_{iso} - \ddot{u}^t_{fb}}{\ddot{u}^t_{fb}} \right) \right] \qquad (9)$$

Where w_1 and w_2 are weighting factors, $\left(u_{m,iso}, \ddot{u}^t_{iso}\right)$ and $\left(u_{m,fb}, \ddot{u}^t_{fb}\right)$ are peak structural deformations and accelerations in isolated and fixed-base buildings, respectively. Clearly, a negative CPM indicates the percent reduction in the combined response of the base isolated building relative to the fixed-base building. The weighting factors are each set to 0.5 such that the CPM represents an average between the two demand parameters, but these could be adjusted depending on the building occupancy and required level of function.

Table 4(a) through 4(c) below show the CPMs for each of the three seismic event return periods. Table 4.a includes the portion of the CPM due to peak inter-story drift, Table 4(b) is that resulting from peak acceleration, defined as the average floor spectral acceleration over the frequency range [3 Hz, 10 Hz], and Table 4(c) includes the combined form shown in Equation 9.

Table 4(a) Drift-based portion of CPM

	Seismic Performance Class		
T_R	I	II	III
72-yr	9%	-2%	-12%
475-yr	3%	3%	-7%
2475-yr	-23%	-2%	37%

Table 4(b) Acceleration-based portion of CPM

	Seismic Performance Class		
T_R	I	II	III
72-yr	-34%	-80%	-84%
475-yr	-33%	-78%	-87%
2475-yr	-43%	-71%	-84%

Table 4(c) Total CPM

	Seismic Performance Class		
T_R	I	II	III
72-yr	-12%	-41%	-48%
475-yr	-15%	-38%	-47%
2475-yr	-33%	-37%	-24%

It is clear from these tables above that, where two buildings are each designed to meet a specific drift limit, the drift-based portion of the CPM is relatively small, indicating that the expected drift demand is consistent between isolated and convention buildings with equal displacement performance objectives considering a range of levels of seismic hazard. Of particular interest is that the base isolated building designed to achieve life-safe performance in the 475-yr earthquake exhibits lower drift demands in the 2475-yr event than the similarly designed fixed-base building. The opposite is true for high-performance buildings, indicating that for a very stiff superstructure, the consequence of yielding is more significant for the isolated building that for a fixed-base building. This conclusion is interesting, and, like all of those stated in this paper, deserves closer investigation.

Of primary interest is the results described in Table 4(c), where the CPM is tabulated considering both drift and accelerations. It is clear from this table that, for all seismic performance classes over all considered levels of hazard, the isolated building exhibits superior performance compared to its fixed-base counterpart. This is due to the significantly reduced floor spectra in the isolated building, leading to enhanced performance of non-structural components. Recent work by Miranda et al. (2003), Astrella et al. (2004) and others have indicated that a significant portion of earthquake-induced economic losses are a result of damage to non-structural systems and building contents. This suggests that the weighting factor applied to the acceleration-based portion of the CPM could be increased, but this would depend on the type of facility being considered.

CONCLUSIONS

This study has investigated the design of base isolated buildings to achieve a range of seismic performance objectives. The results of this study are:

1. It is feasible to design base isolated buildings to meet a range of drift limits for a design basis earthquake, with no excessive drift or ductility demands in either the frequent (service-level) or very rare (maximum considered) events. Inelastic behavior in the superstructure of the isolated building was explicitly considered.

2. The median drift demands and associated dispersions are approximately equal between base isolated and fixed-base buildings over a range of performance classes and levels of seismic hazard. This suggests that the uncertainty in the ground motions is the main contributor to the uncertainty in drift demand relative to changes in the natural frequencies and mode shapes between fixed-base and base isolated buildings.

3. There are significant reductions in floor spectral accelerations for all performance classes and all levels of seismic hazard, leading to a CPM that favors base isolation even for a conventional performance objective.

These conclusions indicate promise for the expansion of the application of seismic isolation as a tool for seismic hazard mitigation that encompasses a greater variety of types of facilities, and warrants further investigation by both researchers and design professionals.

FUTURE RESEARCH

In order to further the research motivations described in this paper, a number of observations are made. In the future, there will likely be an interest by building owners to target multiple, independent performance objectives corresponding to different seismic hazard levels (i.e. Functional after an SLE, Immediate Occupancy after a DBE, and Life-Safe after an MCE.) Current design practice (for both conventional and isolated buildings) restricts the designed to target a *particular* level of performance at a *particular* level of seismic hazard. With innovative seismic isolation devices, it may be possible to achieve multiple performance objectives if the response of the device depends on the level of shaking.

The triple pendulum bearing consists of four concave surfaces and three independent pendulum mechanisms. The outer slider consists of concave surfaces on either side of a cylindrical inner slider with a low friction interface on either end. This forms one pendulum mechanism, and defines the properties of the isolation system under low levels of excitation. The outer slider also consists of sliding interfaces between each dish and the major concave surfaces of the bearing. In particular, one sliding surface is in contact with a concave dish of a specific radius of curvature, forming the second pendulum mechanism which defines the primary properties of the isolation system under moderate levels of excitation. The other sliding surface is in contact with another concave dish with a specific radius of curvature, forming the third pendulum mechanism. The friction coefficient of this third sliding interface is sufficiently large to disallow sliding until an extreme level of excitation occurs. The properties of these three pendulum mechanisms may be selected to optimize the performance of the seismic isolated structure considering multiple levels of seismic hazard. A figure describing the triple pendulum bearing is shown below in Figure 6, and the hysteretic behavior is described in Figure 7. Future research efforts are aimed at repeating the studies presented herein, but with the goal of optimizing the performance benefit of the isolated structure through variation of the parameters for the TC-FP bearing.

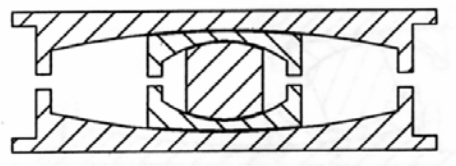


Figure 6: Section through Triple Concave FP bearing

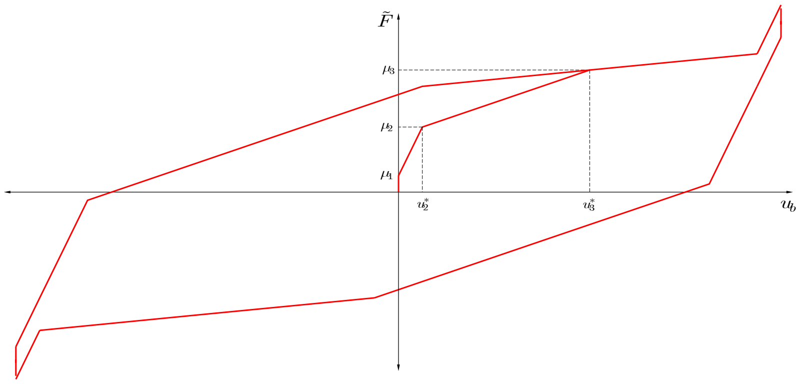


Figure 7: Hysteretic behavior of triple concave friction pendulum bearing, indicating multi-stage force-displacement behavior

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