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ASSESSMENT OF SEISMIC PERFORMANCE OF SEISMICALLY ISOLATED BUILDINGS

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Abstract

Seismically isolated buildings in the United States are currently designed and analyzed by the procedures of the ASCE/SEI 7-16 standard, Chapter 17. The ASCE standard requires that the isolation system be detailed to accommodate the displacement demand calculated in the Risk-Targeted Maximum Considered Earthquake (MCE_R), where this displacement is the average of peak values calculated in seven nonlinear response history analyses. The procedure permits the use of a response modification coefficient (R_I factor) between 1.0 and 2.0 depending on the seismic force-resisting system used. The ASCE/SEI 7-16 standard specifies that the minimum required strength for isolated superstructure and the allowable story drift for the design are determined based on calculations using the MCE_R spectrum.

The authors have investigated the reliability of the ASCE/SEI 7-16 provision by studying an archetypical 6-story perimeter frame seismically isolated buildings designed with special concentrically braced frames (SCBF) and special moment resisting frames (SMF) for a location in California using the minimum criteria of ASCE/SEI 7-16 and also the other enhanced design criteria. Comparable non-isolated structures, also with braced and moment frames, were designed based on the minimum criteria of ASE/SEI 7-16 and studied. This paper summarizes the results of these studies that have been recent published.

The seismic isolation systems considered in this study consist of triple or double friction pendulum isolation bearings. Moat walls for limiting the isolation system displacement and preventing isolator collapse were also considered in some studies. The seismic performance was assessed in terms of the probability of collapse in the MCE_R and in terms of annual exceedance of engineering demand parameters (peak story drift ratio, maximum residual story drift ratio, floor acceleration). The seismic demand parameters used relate to damage of structural components, non-structural components and contents. The spectral shape effects are explicitly considered by utilizing conditional spectra. Conclusions are presented on the required displacement capacity of seismic isolation systems and the strength of the superstructure to meet minimum criteria and to achieve improved performance. (325 words)

Keywords: Seismic isolation, Performance assessment, Isolator displacement capacity, ASCE/SEI 7-16

1. Introduction

Seismically isolated buildings are designed and analyzed according to the minimum requirements of Chapter 17 of ASCE/SEI 7-16 standard [1]. The ASCE standard [1] requires that the isolation system be detailed to accommodate the displacement demand calculated in the Risk-Targeted Maximum Considered Earthquake (MCE_R), where this displacement is the average of peak values calculated in seven nonlinear response history analyses. The procedure permits the use of a response modification coefficient (R_I factor) between 1.0 and 2.0 depending on the seismic force-resisting system used. The forces and drifts for the design of superstructures are based on calculations using the MCE_R spectrum. There are exceptions in which stringent criteria have been employed. Examples are hospitals in California where often project-specific design criteria require the use of an R_I factor of unity for the effects of the MCE_R and larger displacement capacity isolators than the minimum required.

The use of the minimum requirements of the ASCE/SEI 7-16 standard [1] presumably ensures the minimum acceptable level of safety by preserving the lives of the occupants. It is well recognized that these minimum ASCE design requirements do not serve the resiliency objective of avoiding damage in order to maintain facility functionality [2, 3].

Questions may then arise. (a) Is the probability of collapse of seismically isolated structures designed by the minimum design criteria acceptably low? (b) What should be the criteria for design in terms of R_I and isolator displacement capacity to achieve acceptable probability of collapse? (c) What does isolation achieve in terms of performance measures like peak story drift, residual story drift and floor accelerations?

The development of the performance assessment methodologies of FEMA P695 [4] allowed for more rigorous studies of the performance of isolated structures. One of the examples in FEMA P695 [4] involves seismically isolated buildings in which failure of the superstructure was simulated and the isolation system was represented by a generic model together with a displacement-limiting moat wall of various clearances. The structure was a 4-story reinforced concrete building of either a special perimeter moment frame or a special space frame. Concentrating on the code-complaint designs (with R_I=2 in the Design Earthquake or DE based on ASCE/SEI 7-10 [5]), the study demonstrated acceptable collapse margin ratios, which progressively reduced as the moat wall clearance reduced and the space frame was changed from space to perimeter frame.

This paper investigates the reliability of the ASCE/SEI 7-16 provision [1] by concentrating on an archetypical 6-story perimeter frame building that has been previously studied in examples of seismic isolation design and analysis in [6]. Perimeter steel special concentrically braced frames (SCBF) and special moment resisting frames (SMF) for this building are designed for a location in California with an R_I factor of 2.0 and 1.0 in the MCE_R when seismically isolated. The isolation system for these cases consists of triple Friction Pendulum (FP) isolators having a displacement capacity at initiation of stiffening equal to $1.0D_M$ (per minimum requirements of ASCE/SEI 7-16 [1]), and also larger values, where D_M is the displacement demand in the MCE_R (torsion is not accounted for so the displacement considered is D_M without additional displacement due to torsion). The stiffening behavior of the triple FP isolators serves as a displacement restrainer for displacements larger than the assumed capacities. Additionally, double concave (DC) sliding isolators are considered and designed per minimum criteria and without a displacement restrainer, a practice permitted by the ASCE/SEI 7-16 standard [1] (and a common practice in Europe, e.g., [7]). Moreover, representative results are presented for cases in which a moat wall is used at the minimally allowed distance per ASCE/SEI 7-16 standard [1] (displacement capacity D_M) and at larger distances. Non-isolated structures, also with braced and moment frame configurations, are designed and studied.

Summary results are presented on (1) the conditional probability of collapse on the occurrence of the MCE_R of the designed isolated structures and comparable non-isolated structures, and (2) information on the mean annual frequency of exceedance of the peak story drift ratio, the residual story drift ratio and the peak floor and roof acceleration. It is shown that: (a) seismically isolated buildings designed by the minimum criteria of ASCE/SEI 7-16 [1] may have unacceptable probability of collapse in the MCE_R, (b) seismically isolated buildings designed by the minimum criteria of ASCE/SEI 7-16 [1] perform better than the comparable non-isolated buildings in terms of reduced story drift, residual drift and floor acceleration, (c) the probability of collapse is improved by either the use of stiffening isolators of increased displacement capacity or the use of comparable moat walls, (d) reducing the R_I factor may not provide any advantage unless the displacement capacity of the isolators is accordingly increased (as then collapse is due to failure of the isolators) or having a moat wall, (e) designs that meet the minimum criteria of ASCE/SEI 7-16 [1] and without any displacement restrainer (i.e., use of DC isolators without moat wall) have unacceptably high probabilities of collapse.

2. Structures and isolation systems considered

A 6-story archetypical steel buildings are considered. The building was used in examples in the SEAONC Volume 5 Seismic Design Manual [8] and later used in examples of application of the ASCE/SEI 7-16 analysis and design procedures for isolated buildings in [6]. The building's lateral force resisting system consists of four perimeter frames that are configured as SCBF, or SMF. The total seismic weight of the building when seismically isolated is 53670kN. When non-isolated the weight is 45285kN. The building is assumed located on soil class D in San Francisco, CA (Latitude 37.783°, Longitude -122.392°) with MCE_R spectral acceleration values of S_{MS}=1.5g and S_{M1}=0.9g.

The isolation system consists of triple FP isolators (placed below each column) having the geometric and frictional properties determined in [6] but the outer concave plates of the isolators were selected to have different sizes so that the displacement capacities were varied. The considered isolators have displacement capacities of (a) the minimum required by the criteria of ASCE/SEI 7 (capacity D_M at initiation of stiffening) and (b) increased capacities of 1.25D_M and 1.5D_M at initiation of stiffening. Note that the internal construction of the three isolators is the same so that their frictional properties are the same for the same conditions of load and motion. The force-displacement relationship for these isolators is shown in Fig. 1 together with values of the displacement capacity at initiation of stiffening, $D_{Capacity}$, and the ultimate displacement, $D_{Ultimate}$, when the isolator internal parts collapse. The displacement capacity provided was D_M (or a multiple of it) and not D_{TM} (which includes the effects of torsion) as the analysis for the collapse performance assessment was based on two-dimensional representations of the building and torsion was not included.

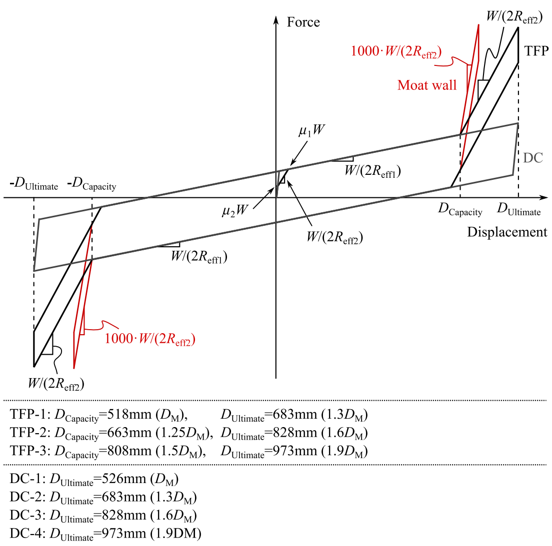

Fig. 1 - Force-displacement relationships of triple FP and DC isolators and effect if moat wall

Two more isolator types were considered without a restrainer ring so that they did not exhibit stiffening behavior. They were designed as DC isolators [9] with the same curvature as the triple FP isolators. Of the two DC isolators, isolator DC-1 has an ultimate displacement capacity equal to D_M, whereas isolator DC-2 has the same concave plate diameter as the smallest triple FP isolator (TFP-1) and thus a larger ultimate displacement capacity than D_M, determined to be $1.25D_M$ (displacement when the inner slider reaches the edge of the concave plate plus half of the contact diameter of 279mm). Isolators DC-1 is permitted by the ASCE/SEI 7-16 standard [1]. Isolators with the characteristics of DC-1 and DC-2 have been used in applications in Europe and South America.

The frictional properties of the triple FP isolators for high speed conditions have been derived in [10, 11] and are listed in Table 1.

Table 1 - Upper and lower bound friction properties of triple FP and double concave isolators

		Interior Isolators		Exterior Isolators	
Isolator Type	Sliding Surface	Outer (μ_1=μ_4 or μ)	Inner (μ_2=μ_3)	Outer (μ_1=μ_4 or μ)	Inner (μ_2=μ_3)
Triple Friction Pendulum	Nominal	0.052	0.017	0.073	0.017
	μ_{max}	1.67	1.29	1.39	1.29
	μ_{min}	0.81	0.85	0.58	0.85
	Upper bound	0.087	0.022	0.101	0.022
	Lower bound	0.042	0.015	0.042	0.015
Double Concave	Upper bound	0.080	NA	0.093	NA
	Lower bound	0.039	NA	0.039	NA

Analyses of the isolated building were conducted per procedures in ASCE/SEI 7-16 [1] for the MCE_R to determine the isolator displacement demands and the base shear force. Response History Analysis (RHA) and Equivalent Lateral Force (ELF) procedures were used. For the RHA, the model of analysis was three-dimensional of which details, including details on the motions used and the scaling procedure, are presented in [6]. Torsion was not considered. That is, the calculation of the isolator displacement demand included the effects of bi-directional excitation but not torsional effects. The frame properties used in the RHA and ELF are those in [1, 2].

The following values were used in design for all structural systems: isolator displacement D_M=526mm and base shear force of 3930kN in the MCE_R (shear force for elastic conditions). Based on these values, frames were designed as follows: (a) per minimum criteria for the MCE_R (per ASCE/SEI 7-16, i.e., R_I=2), (b) for R_I=1 for the MCE_R. Also, comparable non-isolated structures were designed with R=6, Ω_0=2, C_d=5 for the SCBF and with R=8, Ω_0=3, C_d=5.5 for the SMF. Note that for the non-isolated structures the lateral forces were based on the DE (2/3rd of the MCE_R). The section properties for the beams, columns and braces of the designed frames were presented in [10, 11], together with details of the analysis model. The following failure modes were considered:

1) Collapse of the isolators when the lateral displacement exceeds $D_{Ultimate}$ (see Fig. 1 for values). This mode of failure may also occur when moat walls are used as the walls are deformable.
2) Collapse of the structure above the isolators.
3) Story drift ratio exceeding 0.05 for the SCBF [12] and 0.1 for the SMF [13].
4) Instability as detected by termination of the analysis program.

3. Evaluation of structural collapse

Structural collapse evaluation of selected designs with and without moat walls was performed using the procedures described in [10, 11]. The evaluation determined the median collapse spectral acceleration at the fundamental period with the spectral shape effects accounted for based on the approach in [14]. The analysis followed the FEMA P-695 procedures [4] (but for the correction for spectral shape effects) using the set of 44 far-field motions in [4].

The conditional probability of collapse caused by the MCE$_R$ or $P_{COL,MCE}$, is given by:

$$P_{COL,MCE} = \int_0^1 \frac{1}{s\beta_{TOT}\sqrt{2\pi}} \exp\left[-\frac{(\ln s - ACMR)^2}{2\beta_{TOT}^2}\right] ds \tag{1}$$

ACMR is the adjusted collapse margin ratio and β_{TOT} is the total uncertainty, considering uncertainties in record-to-record variability, design requirements, test data and modeling. The detail of the computation of these values are in [10, 11 and 15].

Results are presented in Table 2 for buildings with SCBF and with SMF in terms of probabilities of collapse in the MCE$_R$ computed by total (β_{TOT}) uncertainty. The results are based on analyses using the lower bound friction properties of the isolators as analyses with the upper bound properties resulted in lesser probabilities of collapse.

Table 2 - Results of analysis for buildings (* meets minimum criteria of ASCE/SEI 7-16 [1])

Case	System	SCBF			SMF		
		ACMR	β_{TOT}	$P_{COL,MCE}$	*ACMR*	β_{TOT}	$P_{COL,MCE}$
1*	Non-isolated	2.12	0.538	8.15	3.20	0.448	0.46
2*	R_I=2.0, DC-1	1.23	0.377	29.36	1.08	0.364	41.74
3	R_I=2.0, DC-2	1.47	0.367	14.81	1.26	0.355	26.09
4	R_I=2.0, DC-3	1.67	0.360	7.66	1.41	0.351	16.58
5	R_I=2.0, DC-4	1.87	0.354	3.89	1.55	0.349	10.40
6	R_I=1.0, DC-2	1.45	0.367	15.79	1.24	0.351	26.82
7	R_I=1.0, DC-3	1.64	0.359	8.51	NA	NA	NA
8	R_I=1.0, DC-4	1.83	0.353	4.43	1.55	0.346	10.22
9*	R_I=2.0, TFP-1	1.47	0.357	13.94	1.90	0.389	4.97
10	R_I=2.0, TFP-2	1.59	0.346	8.93	2.09	0.376	2.46
11	R_I=2.0, TFP-3	1.67	0.338	6.38	-	-	-
12	R_I=1.0, TFP-1	1.33	0.356	21.09	1.62	0.371	9.68
13	R_I=1.0, TFP-2	1.53	0.350	11.13	1.82	0.374	5.46
14	R_I=1.0, TFP-3	1.71	0.341	5.87	2.00	0.369	2.98
15*	R_I=2.0, TFP-1 or DC-1 with Moat Wall at D_M	1.53	0.378	13.07	2.82	0.432	0.81
16	R_I=2.0, TFP-2 or DC-2 with Moat Wall at 1.25D_M	1.60	0.359	9.61	2.60	0.400	0.85
17	R_I=2.0, TFP-3 or DC-3 with Moat Wall at 1.50D_M	1.69	0.350	6.76	NA	NA	NA
18	R_I=1.0, TFP-1 or DC-1 with Moat Wall at D_M	1.67	0.395	9.60	3.06	0.442	0.57
19	R_I=1.0, TFP-3 or DC-3with Moat Wall at 1.50D_M	1.85	0.356	4.19	2.77	0.388	0.43

In discussing the results of Table 2, the target reliabilities stipulated in Section 1.3.1.1 of ASCE/SEI 7-16 [1] are used. ASCE/SEI 7-16 [1] requires that conditional probabilities of collapse caused by the MCE_R do not exceed 10% for typical building of risk category I or II, do not exceed 2.5% for essential buildings of risk category IV and do not exceed 5% for other structures (risk category III).

The results in Tables 1 lead to the following observations:

1) For risk categories I and II for which a probability of collapse in the MCE_R of 10% or less is stipulated, the following isolator displacement capacities or moat wall locations are required:

a) For isolation systems without stiffening behavior, an ultimate displacement capacity (that is displacement at isolator collapse) of at least $1.9D_M$ for moment frames and at least $1.6D_M$ for braced frames and regardless of the value of the R_I factor (1 or 2).

b) For moment frames and isolation systems with stiffening behavior, a displacement capacity at initiation of stiffening of at least D_M regardless of the value of the R_I factor (1 or 2).

c) When designing braced frames per ASCE 7-16 [1], a displacement capacity at initiation of stiffening of at least $1.25D_M$ is needed when R_I=2 and at least $1.5D_M$ is needed when R_I=1.

d) For moment frames and isolation systems with moat walls, the moat wall should be placed at a distance of at least D_M for moment frames regardless of the value of the R_I factor (1 or 2).

e) For braced frames and isolation systems with moat walls, and when designing per ASCE 7-16 [1], the moat wall should be placed at a distance of at least D_M when R_I=1 and at a distance of at least $1.25D_M$ when R_I=2.

2) For risk category III for which a probability of collapse in the MCE_R of 5% or less is stipulated, the following isolator displacement capacities or moat wall locations are required:

a) For braced frames and isolation systems without stiffening behavior, an ultimate displacement capacity of at least $1.9D_M$ is required regardless of the value of R_I factor (1 or 2).

b) For cases of moment frames and isolation systems without stiffening behavior designed by either standard, the required ultimate isolator displacement capacity is larger than $1.9D_M$ but detailed studies have not been conducted.

c) For braced frames and isolation systems with stiffening behavior, only the case of R_I=1 designed per ASCE 7-16 [1] and with a displacement capacity at initiation of stiffening of at least $1.5D_M$ provided acceptable probabilities of collapse in the MCE_R. All other cases for braced frames required larger displacement capacity isolators.

d) For moment frames and isolation systems with stiffening behavior, a displacement capacity at initiation of stiffening of at least $1.25D_M$ regardless of the of R_I factor (1 or 2) when designing per ASCE 7-16 [1], a displacement capacity at initiation of stiffening of at least D_M when R_I=2 and at least $1.5D_M$ when R_I=1 are needed.

e) For braced frames and isolation systems with moat walls, the moat wall should be placed at a distance of at least $1.5D_M$ when designing R_I=1 and regardless of standard used in design. For all other cases of braced frames, the probability of collapse exceeded 5%.

f) For moment frames and isolation systems with moat walls, the moat wall should be placed at a distance of at least D_M when designing per ASCE 7-16 [1] and regardless of the value of R_I (1 or 2).

3) For risk category IV for which a probability of collapse in the MCE_R of 2.5% or less is stipulated, the only cases that met the collapse probability criterion were moment frames designed per ASCE 7-16 [1] with R_I=1 or 2 and moat walls placed at distance of at least D_M.

4) The data in Table 2 also show that the non-isolated frames that meet the minimum criteria of ASCE 7-16 [1] have acceptable probabilities of collapse and that the non-isolated SMF has lower than or about the

same probability of collapse in the MCE_R as any of the seismic isolation designs. Kitayama and Constantinou [11, 16] have shown that the non-isolated designs, while having acceptable and even very low probabilities of collapse, have substantially higher probabilities of developing damage in their structural, non-structural systems and contents caused by story drift and floor acceleration in their lifetime than any of the isolated designs. They also had higher probabilities of developing permanent story drift.

4. Probabilistic assessment of seismic response

The seismic performance of the considered isolated and non-isolated buildings was probabilistically assessed in terms of the following engineering demand parameters (EDP): Peak story drift ratio, peak residual story drift ratio and peak floor acceleration. For the evaluation, the mean annual frequency of exceeding specific limits of these EDP was calculated by considering increasing levels of seismic intensity (characterized by the earthquake return period in the range of 43 to 10000 years). Each of the selected EDP is related to the damage of structural and/or nonstructural elements in the buildings. The story drift ratio is generally used as index of structural damage. It is also related to damage to non-structural components that run vertically [17, 18]. The residual story drift ratio is an important indicator in the decision to repair or demolish damaged buildings [18-20]. The peak floor acceleration is related to damage of non-structural components attached to floors (suspended ceilings, lighting fixtures, caster-supported furniture, sprinklers, etc.) [18, 21]. In general, a peak floor acceleration of 0.3*g* indicates very low or no damage to mechanical, electrical, plumbing, suspended ceilings and sprinklers systems, and to building contents [17, 18 and 21]. A maximum story drift ratio of 0.5% indicates the onset of damage for mechanical, electrical and plumbing systems [17] and content damage and loss of use [22], and a 0.5% to 1.0% residual drift ratio indicates that it may be more economical to demolish than to repair [18, 19].

Incremental dynamic analysis (IDA, [23]) is appropriate and useful when assessing the seismic collapse performance of buildings based on FEMA P695 [4]. However, the use of IDA is unsuitable for the assessment of seismic response other than collapse primarily due to fact that it is based on the use of non-frequent strong earthquake motions (for the collapse evaluation of the buildings, same motions are used to represent very low seismic intensities and then scaled up to intensities beyond the MCE_R [4, 11]). For the assessment of seismic response that causes minor or moderate damage under more frequent earthquakes, the selection and scaling of ground motions should be consistent with the seismic hazard as described in the probabilistic seismic performance procedure in NIST [24] and in Lin et al. [25]. This procedure makes use of conditional spectra, with conditional mean and conditional standard deviation that links the seismic hazard information with the selection of ground motions. Also, the multiple stripe analysis technique [26] was selected to conduct the analyses of this study as it allowed the use of different sets of hazard-consistent ground motions at each intensity level. The results of the study are presented in the form of relationships between the selected EDP and the annual frequency of exceeding specific EDP limits. For the analysis, in total 1,200 motions were selected and scaled to represent ten different earthquake return periods and three different building periods (40 motions per return period; 400 motions per building period). The procedure for the selection and scaling of the motions is described in [11, 16].

Figs. 2 to 7 present the mean annual frequency of exceeding limits on the peak story drift ratio, the peak residual story drift ratio, the peak floor acceleration and the peak roof acceleration, respectively. Results are presented for the non-isolated SCBF and SMF and for the isolated SCBF and SMF structures with R_I=1 and 2 (in the MCE_R [1]). The presented results are triple FP isolators of the minimum displacement capacity D_M (TFP-1) and the maximum 1.5D_M (TFP-3). Also, the case with moat walls are included in Figs. 6 and 7 for the SMF with R_I=2 (in the MCE_R [1]), isolator TFP-1 with a moat wall placed at distance D_M and the enhanced design with isolator TFP-2 and a moat wall placed at distance 1.25D_M.

For the generation of results shown in Figs. 2 to 7, the procedures detailed in [11, 16] for probabilistic seismic performance assessment were used.

In the discussion that follows some results will be presented that demonstrate the differences between different structural systems.

The results in Figs. 2 to 7 show that: (a) the mean annual frequency of exceeding any acceleration values at the floor or roof level are lower in the isolated buildings of any design than in the comparable non-isolated buildings, (b) isolated buildings have lower mean annual frequencies of exceeding most but not all values of peak story drift ratio and peak residual story drift ratio than comparable non-isolated buildings, (c) isolated buildings designed by the improved design of R_I=1.0 in the MCE_R [1] and stiffening isolators with displacement capacity at initiation of stiffening equal to $1.5D_M$ have lower mean annual frequencies of exceeding all values of peak story drift ratio and peak residual story drift ratio than comparable non-isolated buildings, and (d) the use of moat walls has generally small effects on the mean annual frequency of exceedance of peak drift ratio and residual drift ratio by comparison to comparable designs with stiffening triple FP isolators without a moat wall. However, the use of moat walls results in higher mean annual frequency of exceedance of high values for the peak floor acceleration due to the pounding effect between building and the wall.

The results in Figs. 2 to 7 show notable differences in behavior between the considered structural systems of braced and moment frames. Particularly the braced frames show lower mean annual frequencies of exceedance of story drift and floor or roof acceleration limits for low seismic intensities characterized in this paper by drift ratio less than 0.5% and acceleration less than 0.3g which are considered representative of the onset of some damage to the structural and non-structural systems, and the building contents [17, 18, 21, 22]. The difference is due to differences in stiffness between the two structural systems (braced frames being about 5 times stiffer than the braced frames based on pushover analysis results in [11, 15]).

The residual drift ratio is used as another index of performance related to the need to repair or demolish a building in its lifetime. Considering the limit of 0.5% on the residual drift ratio, braced frames show a lower annual frequency of exceedance than moment frames, with the difference increasing as the superstructure strength is reduced (R_I factor increased). Again, this behavior is explainable on the basis of the differences in stiffness that result in lower drift and thus lower residual drift in the braced systems.

It is evident that while moment frames exhibit an inherent ability to deform more than braced frames, which results in a reduced probability of collapse in the MCE_R, particularly when isolator failure is prevented (use of moat walls) or delayed (use large displacement capacity stiffening isolators), they have higher probabilities of developing damage in low intensity earthquakes and of being in need to be demolished following an earthquake.

It is noted that the demand hazard curves of braced frames are flat or nearly flat in the range of nearly zero to 0.02 for the residual drift ratio. The same behavior actually occurs for the moment frames for large values of the residual drift ratio beyond 0.02, which are not of any practical value in assessing performance. The difference is again the result of the difference in stiffness. The flatness reflects the fact that the residual story drift ratio is mostly controlled by the collapse probability in the integration of the fragility curve over the slope of the seismic hazard curve.

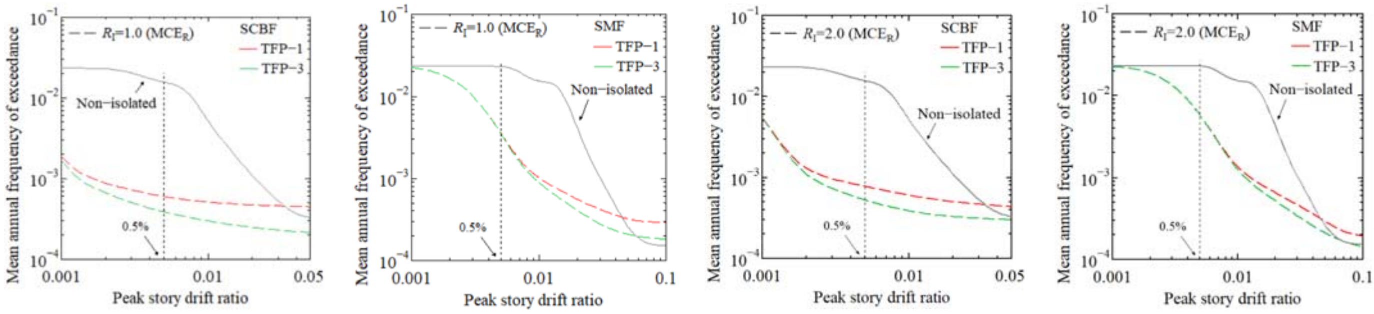

Fig. 2 - Mean annual frequency of exceeding limits on peak story drift ratio

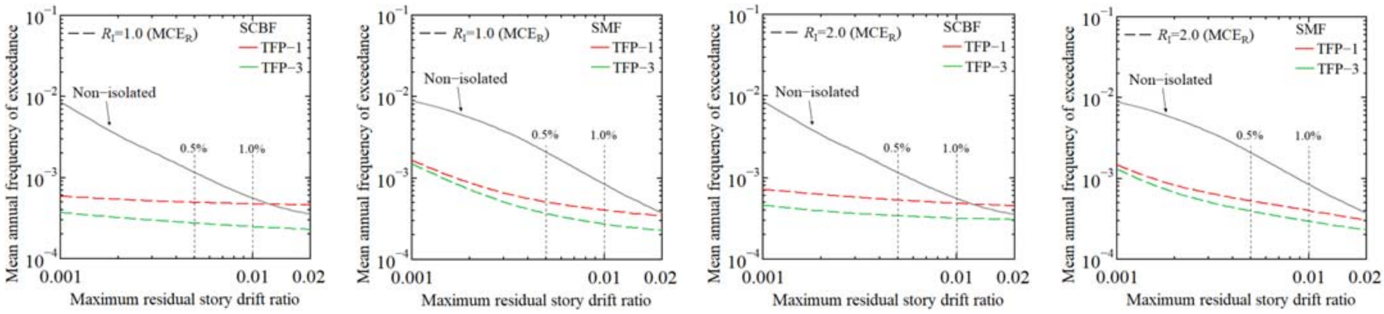

Fig. 3 - Mean annual frequency of exceeding limits on peak residual story drift ratio

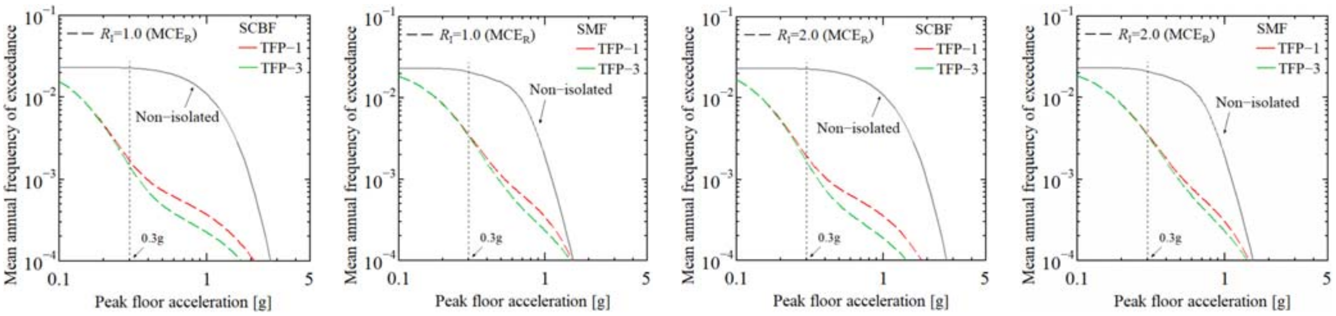

Fig. 4 - Mean annual frequency of exceeding limits on floor acceleration

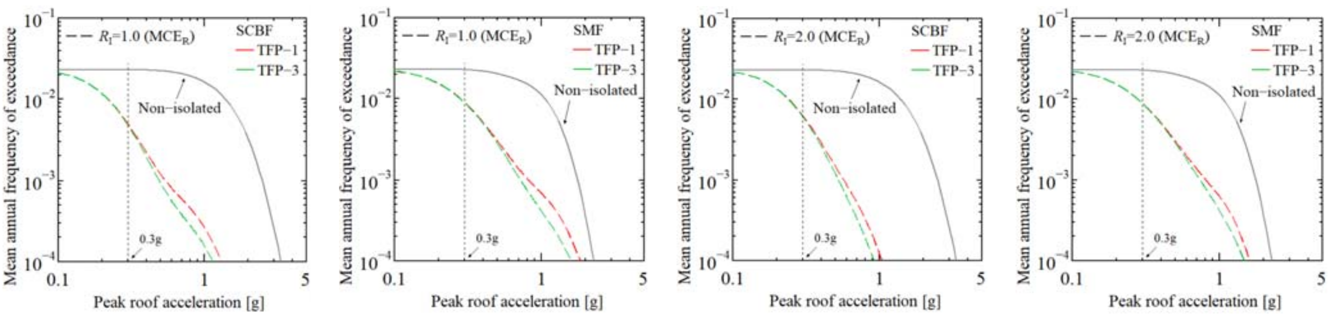

Fig. 5 - Mean annual frequency of exceeding limits on roof acceleration

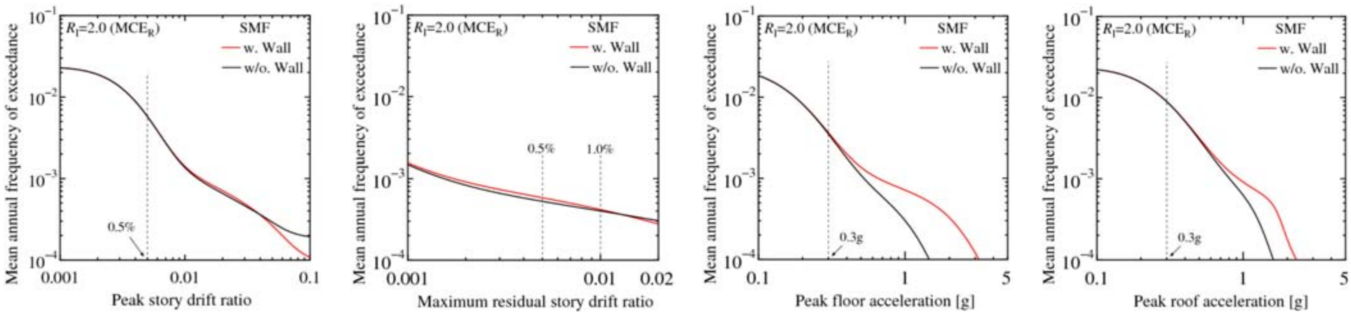

Fig. 6 - Comparisons of mean annual frequency of exceeding limits on drift, residual drift and peak acceleration for SMF designed per minimum criteria of ASCE 7-16 (R_I=2, Isolator TFP-1) without and with moat wall placed at distance D_M

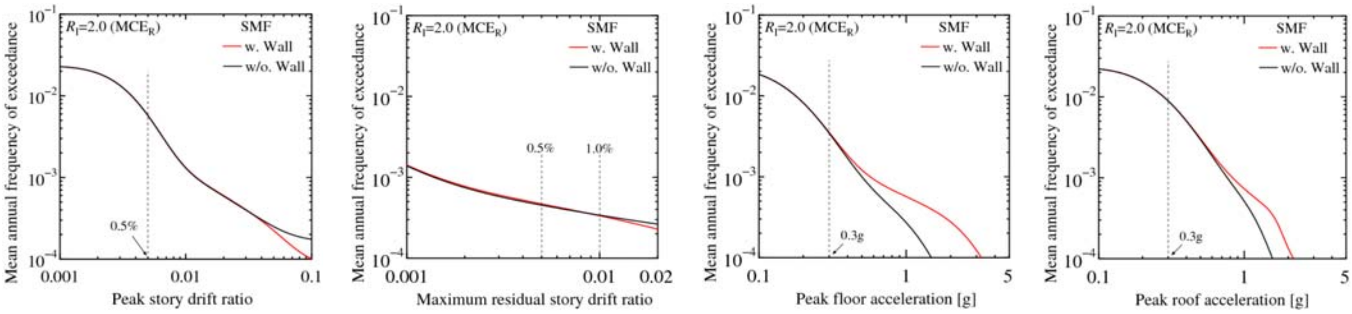

Fig. 7 - Comparisons of mean annual frequency of exceeding limits on drift, residual drift and peak acceleration for SCBF designed per ASCE 7-16 for R_I=2, isolator TFP-2 without and with moat wall placed at distance $1.25D_M$

5. Conclusions

It has been shown that seismically isolated structures designed by the minimum criteria of ASCE/SEI 7-16 [1] may have unacceptable probabilities of collapse in the MCE$_R$, whereas comparable non-isolated structures, also designed by the minimum criteria of ASCE/SEI 7-16 [1], have acceptable probabilities of collapse. Improvement of the collapse performance is achieved by designing for an R_I=1.0 and providing isolators with stiffening behavior and with a displacement capacity at initiation of stiffening equal to $1.5D_M$ and ultimate displacement capacity of $1.9D_M$, where D_M is the displacement capacity in the MCE$_R$ as stipulated by the minimum criteria of ASCE/SEI 7-16 [1]. In general, increasing the strength of seismically isolated structures by itself (by reducing R_I) does not result in improvement of the collapse performance unless the displacement capacity of the isolators is proportionally increased. The reason for this behavior is that by increasing the strength, inelastic action in the superstructure is delayed so that the isolator displacement demand is increased leading to collapse by failure of the isolators.

Moreover, it has been observed that the use of moat walls may improve the collapse performance by comparison to comparable designs with stiffening isolators in cases where collapse is dominated by failure of the isolators due to excessive displacement demand. This is the case when the design utilizes inconsistent combinations of superstructure strength (small R_I, say 1.0) and minimal displacement capacity isolators.

Given that isolated structures of whatever design details mostly had higher probabilities of collapse than comparable non-isolated structures, studies of the performance of the designed isolated and non-isolated structures in terms of peak floor acceleration, peak story drift ratio and peak residual drift ratio were conducted in order to investigate the advantages offered by seismic isolation. These studies and additional results available in [11, 16] showed that isolated structures designed by any design criteria (minimum or improved designs) have lower probabilities than comparable non-isolated structures to develop some form of minor damage to non-structural components and building contents or to have large enough residual story drift to require demolition.

The study also demonstrated the significance of utilizing restraint in the isolation system to limit displacements and prevent collapse of the isolators. The use of stiffening triple FP isolators is one of the options available. The other is the use of moat walls. However, the displacement capacity of the isolators and the location of the moat wall should be consistent with the superstructure design (value of R_I). Designs based on the minimum criteria of ASCE/SEI 7-16 [1] and with a moat wall placed at the MCE_R displacement does not always ensure an acceptable probability of collapse. Moreover, the lack of any restraint, either in the form of stiffening isolators or moat walls, generally results in unacceptable probability of collapse.

The presented studies were limited to examples of perimeter braced and moment 6-story steel frames with sliding isolators at one particular location in California. The study did not consider the effect of vertical ground shaking. Also, there is a need to extend these studies to taller structures and to other structural systems (e.g., concrete space frames) in order to cover a wider range of structures of interest in seismic isolation. Moreover, these studies have been limited to the analysis of two-dimensional representations of isolated structures, whereas three-dimensional representations would have likely resulted in the prediction of higher probabilities of collapse. Similarly, consideration of the vertical ground shaking should result in even higher probabilities of collapse.

Nevertheless, the results of these studies clearly show a need to re-visit the ASCE/SEI 7-16 [1] criteria for the design of seismically isolated structures. Ideally, the specified R_I factor, the minimum displacement capacity of the isolators and the isolator stiffening characteristics should be dependent on the seismic force-resisting system. In the absence of such detailed studies, it justified to require designs with R_I=1.0 and isolators with $D_{Capacity}$=1.5D_M and $D_{Ultimate}$=1.9D_M in order to ensure acceptable collapse performance for important structures. Such a design also offers additional benefits in terms reduction of the probability to develop minor damage to non-structural components and the building contents, and in reducing the probability of having to demolish the building in its lifetime.

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