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COMPARATIVE LIFE CYCLE ANALYSIS OF CONVENTIONAL AND BASE-ISOLATED THEME BUILDINGS

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ABSTRACT

Comparative performance evaluation including life cycle cost evaluation is currently being conducted on a series of conventional and base-isolated case study buildings. This paper presents estimated project costs and direct earthquake losses estimation results for 3-story steel moment frame and braced frame buildings, both conventional and isolated, designed to minimally satisfy U.S. building codes. Seismic isolation appears more effective in reducing the losses in a braced frame building than in a moment frame building, due to both the flexibility of the moment frame and the substantial contribution of collapse losses in the moment frame, as designed. The final results will be useful for engineers to communicate the economic impact of higher performance systems to their clients.

Introduction

Relative to conventional design, seismic isolation systems provide substantial reduction in both drifts and accelerations, thereby reducing both structural and nonstructural damage. As such, seismic isolation is frequently considered for buildings that must remain operational in the design earthquake. However, a typical owner is generally motivated by cost, and the higher initial cost of a seismic isolated building coupled with the difficulty of conveying the performance benefits has prevented seismic isolation from penetrating mainstream design practice in the United States. Fortunately, emerging performance based earthquake engineering (PBEE) procedures allow realistic assessment of probabilistic losses due to earthquakes, and will allow building owners to factor life cycle cost considerations into the decision making process.

Under the PBEE framework developed by the Pacific Earthquake Engineering Research Center (PEER) (Miranda, 2003), performance metrics, or decision variables, such as repair costs and downtime, are determined through a consequence analysis. The assessment problem is deconstructed into four basic stages: hazard analysis, response analysis, damage analysis, and loss analysis. In hazard analysis, ground motions are selected to represent discrete events along the hazard curve. Response analysis involves the creation of high fidelity structural models of the buildings and predicting the relevant structural demands to each ground motion by response history analysis. In the damage analysis, fragility functions are defined that relate structural

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response values to physical damage in the structural and nonstructural components. Finally, in the loss analysis, repair actions with associated costs and repair times are defined. According to the total probability theorem, the four stages are combined by integration to determine the expected losses in a given event or over the lifetime of the building.

In this paper, the cost and performance of conventional and base-isolated 3-story code-designed steel braced frame and moment frame buildings is evaluated. Initial project costs have been estimated for each building with the help of a cost consultant. Each step in the loss estimation procedure has been performed. The general ground motion selection, modeling procedure, and structural analysis results are reported in a companion paper (Erduran, 2010). In this paper, repair costs are estimated for different scenarios considering both collapse and non-collapse losses. The scenario losses are integrated over the hazard curve to arrive at an expected annual loss for each building.

Initial Design and Construction Cost Estimates

Four three-story steel buildings were designed by Forell-Elsesser Engineers, Inc. for use in this study: a conventional special concentric braced frame (SCBF), an isolated ordinary braced frame (OCBF), a conventional special moment resisting frame (SMRF), and an isolated intermediate moment resisting frame (IMRF). The design of the buildings for a high seismicity Los Angeles, California location is discussed in a companion paper (Erduran, 2010). Design and construction costs for the buildings were estimated with the help of Peter Morris of Davis Langdon, a professional cost estimator. The total cost of assembled structural elements, including materials and labor, was based on an assumed cost per unit weight or volume of raw materials using mid-2008 market values. For instance, concrete was priced at $350/cubic yard and steel was priced at $4000/ton. The cost of moment and brace connections, including materials and labor, was estimated from representative connection details. Unit costs were also assumed for assembled nonstructural components; floor slabs, exterior walls, interior partitions, windows, roofing, and ceilings were all priced using a unit cost per sq. ft. Reasonable quantities for architectural elements were proposed based on Morris's professional experience.

The total project costs for each building are listed in Table 1. Total building and site costs have been predicated on the assumption of a clean site with no site acquisition fee. The recommended budget for each project is about 50% higher (Table 1) and includes the following surcharges: general conditions (9%), contractor's overhead and profit (5%), contingency for development of design (10%), soft cost package (20-21%). The only difference in the assumed surcharges is an increased design fee for the isolated building (2% versus 1% for the conventional building), which is reflected in the soft cost package. As can be seen, the cost premium for seismic isolation, not including a design surcharge, is estimated to be 11.6% for a braced frame building and 7.7% for a moment frame building, which is substantial.

The total costs are broken down into general categories in Table 1, while Table 2 identifies the major costs contributing to the cost of the isolation layer. The most substantial contributors to the cost of the isolation layer are the devices ($525K) and the additional floor system at the base ($710K). These additional costs are somewhat offset by reduced superstructure framing costs in the isolated building as a result of reduced section sizes; more so in the moment frame building since a braced frame is a very inexpensive lateral system.

Table 1: Initial cost comparison for the buildings

	Conventional SCBF	Isolated OCBF	Percent Increase for Isolation	Conventional SMRF	Isolated IMRF	Percent Increase for Isolation
Foundation	$265K	$331K	24.9%	$363K	$331K	-8.8%
Structural Framing	$1,387K	$1,193K	-14.0%	$2,162K	$1,506K	-30.3%
Isolation Layer	-	$1,973K	NA	-	$1,973K	NA
Nonstructural Elements	$6,793K	$6,793K	0%	$6,793K	$6,793K	0%
Utilities	$7,485K	$7,485K	0%	$7,485K	$7,485K	0%
Total Building and Site Cost	**$15,931K**	**$17,776K**	**11.6%**	**$16,803K**	**$18,089K**	**7.7%**
Recommended Budget	**$24,067K**	**$27,081K**	**12.5%**	**$25,385K**	**$27,554K**	**8.5%**

Table 2: Breakdown of added costs due to seismic isolation

	Total Cost	Unit Cost ($/sf footprint area)	Unit Cost ($/sf total area)
Excavation	$156K	6.48	2.16
Retaining Wall and Moat Cover	$218K	9.06	3.02
Isolator Pedestals	$29K	1.19	0.40
Isolation Devices	$525K	21.82	7.27
Level 1 Framing and Floor Slab	$710K	29.52	9.84
Flexible Connections	$115K	4.78	1.59
Crawlspace Drainage/Lighting	$120K	5.00	1.67
Suspended Elevator Shafts	$100K	4.16	1.39
Total Isolation Layer	**$1,973K**	**82**	**27.33**
Differential Costs	*Braced Frame / Moment Frame*		
Foundation (without excavation)	$66K / -$32K	2.74 / -1.31	0.91 / -0.44
Structural Framing	-$194K / -$656K	-8.06 / -27.25	2.69 / -9.08
Total Cost Premium	**$1845K / $1285K**	**76.67 / 53.40**	**25.56 / 17.80**

The absolute cost of the seismic isolation layer, per square foot (sf) of footprint area, tends to be similar for all applications. The relative cost of seismic isolation, as a percentage of the total cost, may be higher in this study than for typical U.S. isolation applications because: (1) the relative premium is greater for a short building than a tall building, and (2) the relative premium is greater for standard classes of buildings (office, residential) than for buildings with expensive contents (hospitals, emergency response). A cost premium of 8-12% is a huge deterrent for most owners, and strategies to reduce this cost should be investigated seriously.

Structural Analysis of Theme Buildings

A detailed 3 dimensional (3D) nonlinear model for dynamic analysis was developed for each building in the structural analysis program OpenSees. For loss estimation, motions were selected to represent 9 discrete events along the hazard curve, and each building was analyzed by nonlinear response history analysis to each set of motions. A brief presentation of the analysis results interpreted for loss estimation follows. The ground motion selection, model development, and analysis results are presented in more detail in a companion paper (Erduran, 2010).

Seismic Response Distribution Functions

Lognormal distributions for observed seismic responses were developed from the RHA results for each ground motion scenario. Joint distribution functions were determined for the correlated responses by computing the mean, standard deviation, and correlation matrix of the natural log of the response vectors (ATC, 2007).

Representative response distribution functions for story drift and acceleration are shown in Fig. 1. Distributions for first story drift [Fig. 1(a)-(c)] and roof acceleration [Fig. 1(d) and (e)] are plotted for each building for the 72 year, 475 year and 2475 year earthquake scenarios. In each event, drifts are reduced in the OCBF relative to the SCBF and in the IMRF relative to the SMRF. The drift reduction is more substantial for the braced frame building because of the inherent flexibility of moment frame systems. In the 2475 year event, the IMRF has a lower expected drift than the SMRF but a larger dispersion, which means that a small probability of observing very large drifts in the IMRF exists. Accelerations are greatly reduced in both isolated buildings relative to the conventions ones, and their dispersions are small, such that accelerations can be predicted with high confidence.

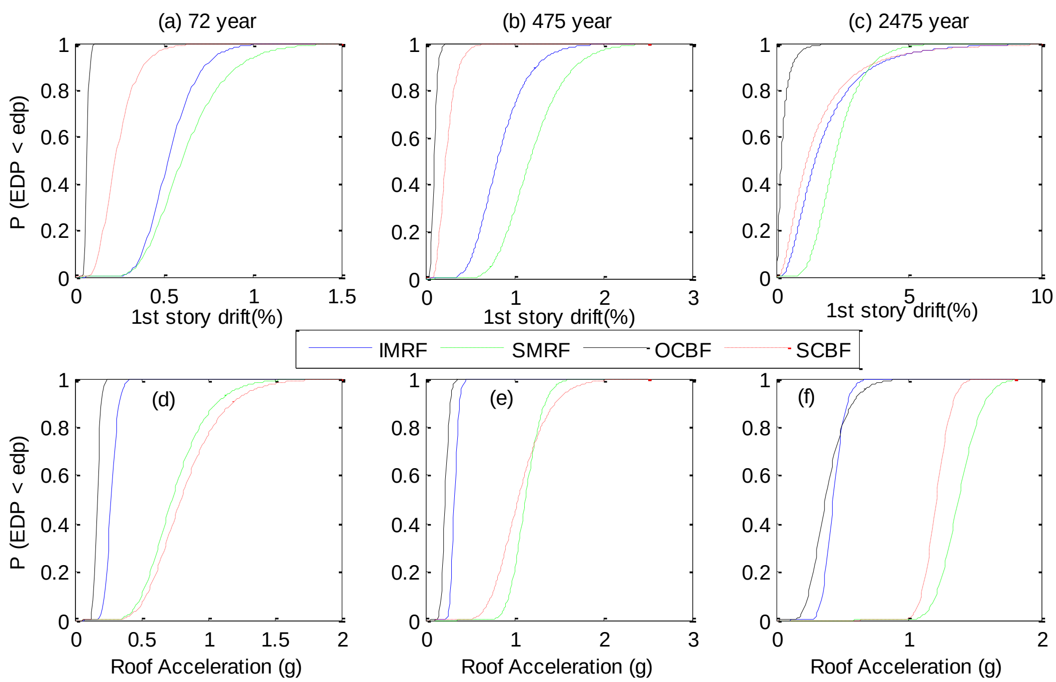

Figure 1. Fitted cumulative distribution functions for (a)-(c) 1st story drift and (d)-(f) roof acceleration in 72 year, 475 year and 2475 year events, respectively.

Collapse Fragility

A collapse fragility was generated for each building using Incremental Dynamic Analysis (IDA) of the building model to the set of 20 large intensity ground motions used for the largest scenario earthquakes (Erduran, 2010). The collapse fragility is a cumulative distribution function describing the probability of collapse versus ground motion intensity, taken to be PGA. The collapse intensity of each motion was determined by plotting the PGA, which is scaled incrementally, against measured peak interstory drift. The collapse limit was interpreted as the intensity at which the response measure increases without bound for small intensity increments. For IDA, the building models were simplified to 2D models with concentrated plasticity elements using a stiffness and strength degrading Clough model for the moment-rotation relation. The model parameters were assigned using guidelines developed by Lignos (2007) for different connection types using a database of cyclic tests of beam-column connections. The distribution function was determined by fitting the observed data to a lognormal distribution.

The collapse fragilities of the buildings are plotted in Fig. 1. The SMRF and OCBF are predicted to collapse at higher ground motion intensity than the SCBF and IMRF. The IMRF, though subjected to lower spectral accelerations than the SMRF, is the weakest of all the buildings and only about 40% as strong as the SMRF (Erduran, 2010). Since isolated buildings accumulate ductility quickly after yielding, the collapse behavior of the IMRF is not surprising.

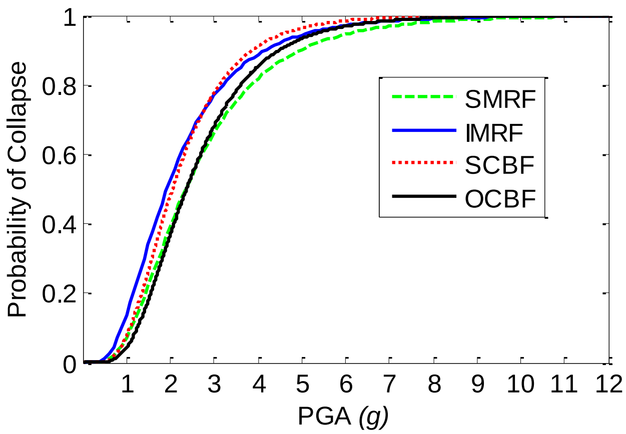

Figure 2. Collapse fragility as a function of PGA for the four theme buildings.

Damage and Loss Analysis Procedure

Given seismic demands, probabilistic descriptions of component damage and likely repair costs are needed to carry out the loss estimation. The following terminology is utilized:

- *Fragility Functions*: Fragilities relate computed demands such as story drifts to physical descriptions of damage or damage states, and are expressed as probabilistic distributions.
- *Consequence Functions*: Consequence functions relate predicted damage states (DS) to repair actions/repair costs, and are expressed as probabilistic distributions.
- *Performance Group (PG) Quantities*: Performance groups assemble structural and non-structural components into groups that can be described by the same demand parameters and fragility functions.

Fragility and consequence functions were gathered from various sources and represent the best available information. Table 3 summarizes the component fragility and consequence functions that were used in the study. Damage states and repair actions are described for each performance group. The fragility and consequence functions are represented as lognormal distributions, with given median (x_m) and dispersion (β) values. Whenever possible, fragility functions were selected from sources that documented their development.

We developed fragilities separately for RBS and WUF-W moment connections and for braces from available test data. Quality data was available for moment connections via tests conducted as part of the SAC steel program. However, an identical loading protocol was used for each test, and the observed variance in the test results is likely to be less than in the field. For the braces, a simple strategy was adopted to simply replace the brace once buckling occurs. Repair costs for RBS and WUF-W connections were obtained from ATC-58 (ATC, 2007), while repair costs for pre-Northridge connections was estimated from post-earthquake studies of building repair (Gates, 1995). The median replacement cost for a brace was simply estimated as 4 to 5 times the cost of materials and installation in a new building. Repair costs for nonstructural elements and contents were generally evaluated by combining relative repair costs, denoted as a fraction of the replacement cost (Krawinkler, 2005), with the unit replacement cost, evaluated from RSMeans (2008).

Table 3: Performance groups, fragility and consequence functions used in analysis

Perfor-mance Group	Fragility Functions				Consequence Functions			Source
	EDP^	**Damage Description**	**x_m**^	**β**^	**Repair Action**	**x_m^ ($)**	**β**^	
RBS Connections (SMRF bldg)	IDR* (%)	Flange and web buckling	2.2	.22	Heat straightening	8000 each	.30	From data in Engel-hardt 2000, Gilton 2000, and Ricles 2002.
		Beam lateral torsional buckling	3.6	.16	Heat straightening; replacement	15000 each	.30	
		Tearing/fracture through beam flanges	5.6	.17	Replace large portion of beam with shoring	60000 each	.40	
WUF-W Connections (IMRF bldg)	IDR (%)	Beam flange buckles; panel zone yielding	2.5	.22	Add stiffener plate on web	8000 each	.30	From data in Ricles 2002.
		Severe local buckling; weld cracking	3.7	.14	Back gouge and reweld repair	15000 each	.30	
		Beam bottom flange fracture	5.5	.09	Replace large portion of beam with shoring	60000 each	.40	
Pre Northridge Connections (SCBF and OCBF bldgs)	IDR (%)	Inspection	1.2	.40	None	1500 each	.35	Deierlein 2009
		Fracture of lower beam flange, fracture may spread to column	1.7	.40	Gouge out and reweld, repair column as needed	9000 or 23000 each	.35	
		Fracture of upper beam flange; fracture may spread to column	2.5	.40	Gouge out and reweld; remove floor slab, repair column as needed	16000 or 30000 each	.35	
		Ductile fracture at weld access hole spreading through beam flange	3.0	.40	Replace large portion of beam with shoring	20000 each	.35	
Braces (SCBF and OCBF bldgs)	$\delta_{brace}/\delta_{cr}$	Brace buckles			Replace brace			
		Large inelastic cycling			Replace brace and gusset plate			

Table 3: Performance groups, fragility and consequence functions used in analysis (cont.)

Performance Group	Fragility Functions				Consequence Functions			Source
	EDP^	**Damage Description**	**x_m^**	**β^**	**Repair Action**	**x_m^ ($)**	**β^**	
Aluminum Framed Windows	IDR (%)	Minor damage	1.6	.29	Realignment	70/ pane	.20	Kra-winkler 2005
		Cracking without fallout	3.2	.29	Replace glass panel	348/ pane	.20	
		Panel falls out	3.6	.27	Replace glass panel	696/ pane	.20	
2-sided Interior Partitions	IDR (%)	Small cracks	.39	.17	Patch	.67/sf	.20	Porter 2001, Mitrani-Reiser 2007
		Extensive cracking; crushing	.85	.23	Replace	3.90/ sf		
Interior Finish (Opposite Exterior Wall)	IDR (%)	Small cracks	.39	.17	Patch	.42/sf	.20	Porter 2001, Mitrani-Reiser 2007
		Extensive cracking; crushing	.85	.23	Replace	2.48/ sf	.20	
Suspended Acoustical Tile Ceilings	PFA* (g)	Wires exposed, some panels fall	.27	.40	Fix wires, replace fallen panels	.23/sf	.20	Kra-winkler 2005
		Main runners & tee bars damaged	.65	.50	Replace bars and fallen panels	.95/sf	.20	
		Grid tilts; near collapse	1.28	.55	Replace ceiling and panels	3.16/sf	.20	
Traction Elevators	PGA* (g)	Failure	.41	.28	Inspection and repair	55000 each	.20	
Automatic Sprinklers (braced)	PFA (g)	Fracture	32	1.4	Replace	1000/ 12 lf	.50	Mitrani-Reiser 2007
Servers and Network Equip.	PFA (g)	Overturning; Inoperable	.8	.50	Repair	50000 each	.40	ATC 2007
Desktop Computers	PFA (g)	Falling; Inoperable	1.2	.60	Repair/replace	3000 each	.40	ATC 2007

* IDR = story drift, PFA = peak floor acceleration, PGA = peak ground acceleration
^ EDP = engineering demand parameter, x_m = median EDP for fragility or median repair cost for consequence, β = associated dispersion

Loss estimation was carried out using a Matlab code developed by the authors for this purpose. The code uses a Monte Carlo simulation technique to sample from the distribution functions for seismic response, seismic fragility, and consequence functions. Correlated demand vectors for seismic response are generated by passing random variables sampled from a uniform distribution through a linear transformation based on the mean and correlated standard deviation (ATC, 2007). The simulation process includes a decision tree to evaluate if collapse occurs, and computes losses independently for collapse and non-collapse. For each simulation, the PGA is sampled from the associated distribution for the given scenario, and the probability of collapse *p* at that PGA is evaluated from the collapse fragility function (Fig. 2). If the building is declared to have collapsed, evaluated by random sampling with *p* percent chance, the total replacement cost is estimated from the consequence function for collapse. If the building does not collapse, the total repair cost is estimated by summing the repair costs for observed damage states in individual performance groups, which are sampled from the fragility functions. The damage states for pre-Northridge connections, applied to the braced frame buildings, are not ordered; thus a probabilistic decision tree was added to the sampling routine for this performance group.

Probabilistic Repair Costs

For each scenario earthquake, repair cost distributions were generated separately for both collapse and non-collapse simulations. The probability of collapse was simply the number of observed collapse simulations ÷ total number of simulations. The cumulative distribution function (CDF) of repair cost for each scenario is evaluated by combining collapse (C) and non-collapse (NC) repair costs as follows:

$$CDF_{total} = CDF(C) \cdot P(C) + CDF(NC) \cdot (1 - P(C)) \quad (1)$$

where P(C) = probability of collapse. Combining the two CDFs in this manner leads to a two-tiered total CDF, such as shown in Fig. 3 for representative earthquake scenarios. The CDF is essentially flat over an extended cost range between the two tiers. For example, the 2500 year CDF in Fig. 3(a) can be interpreted as ~85% probability of repair cost ranging from 0-$3 million (non-collapse), and ~15% probability of repair cost ranging from $20-$35 million (collapse).

An obstacle encountered in the simulation process was that for low intensity scenarios, the total repair cost was computed to be zero in many individual Monte Carlo simulations. Since a lognormal distribution cannot be fit to data containing zeros, the data was required to be altered in some fashion that leads to a plausible distribution. Replacing the zeros by arbitrary small numbers produces a distribution with a low expected value but a very large dispersion such that the tails of the distribution lead to unrealistically large losses. The solution adopted was to replace the zeros by 1/1000 to 1/100 of the maximum repair cost observed over the simulations. This led to plausible repair cost distributions, which are reflected in the tiered CDFs in Fig. 3.

Table 4 summarizes the median expected repair costs in each building for each scenario earthquake. The total repair cost is computed from Eq. (1), wherein repair costs for both collapse

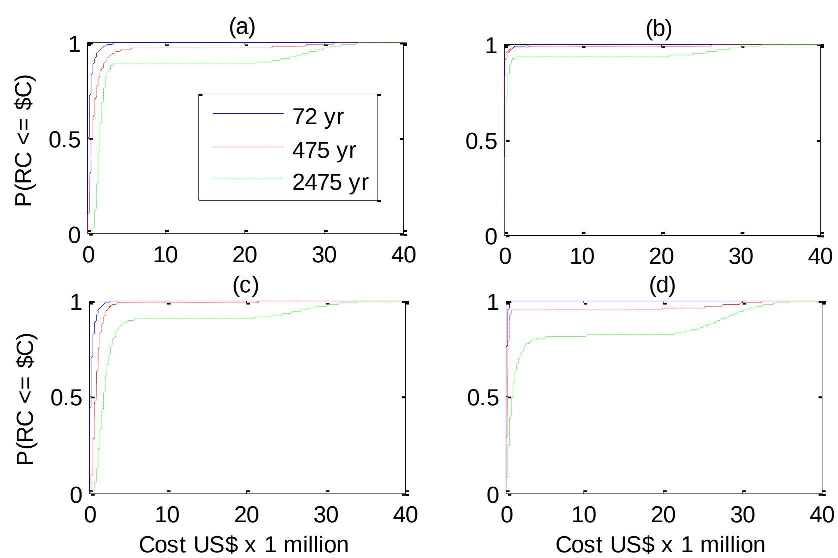

Figure 3. CDF for P(Total Repair Cost <= $C/IM) in 72, 475 and 2475 year scenario earthquakes for (a) SCBF, (b) OCBF, (c) SMRF and (d) IMRF.

and non-collapse, and the probability of collapse are given. All buildings except the OCBF experience nominal damage in frequent events, because damage in interior partitions is initiated at low drift levels; low level damage in the moment frame is greater because of its relative flexibility. In moderate to design level earthquakes, non-collapse losses dominate and the repair cost is significantly less in isolated buildings. In very rare earthquakes, collapse losses are significant in all buildings but the OCBF and the repair costs in the other three are comparable.

The losses for individual earthquake scenarios were integrated numerically with the hazard curve to derive a loss curve for each building according to the following:

$$\text{ARC} = \sum_i \tfrac{1}{2}(MAF(i+1) - MAF(i-1)) \cdot (1 - CDF(i)) \quad (2)$$

where ARC = annual repair cost, and MAF(i) is the mean annual frequency of exceedance of the i^{th} scenario earthquake. The area under the loss curve is integrated to get the expected annual loss, which is reported to be $48.7K, $12.9K, $29.9K, and $27.9K for the SCBF, OCBF, SMRF, and IMRF buildings, respectively. From this perspective, the benefit of isolation is obvious for the braced frame buildings but less apparent for the moment frame buildings, as designed.

Table 4: Collapse and not-collapse losses of buildings to different earthquake scenarios

Scenario	**Braced Frame Buildings**					**Moment Frame Buildings**				
	Building	**Prob. of Collapse**	**Median Loss ($K)**			**Building**	**Prob. of Collapse**	**Expected Losses ($K)**		
			NC	**C**	**Total**			**NC**	**C**	**Total**
10 year	SCBF	0	60.4	0	60.4	SMRF	0	4.2	0	4.2
	OCBF	0	0	0	0	IMRF	0	17.4	0	17.4
40 year	SCBF	0	111.9	0	111.9	SMRF	0	131.7	0	131.7
	OCBF	0	0.7	0	0.7	IMRF	0	99.1	0	99.1
72 year	SCBF	0	225.9	0	225.9	SMRF	0	263.5	0	263.5
	OCBF	0	6.9	0	6.9	IMRF	0.004	136.9	27914	137.3
200 year	SCBF	0.013	465.2	27470	471.9	SMRF	0.004	622.7	28492	624.5
	OCBF	0.005	24.2	29978	24.5	IMRF	0.016	197.4	24984	199.5
475 year	SCBF	0.030	672.1	29102	695.2	SMRF	0.013	917.6	24848	925.5
	OCBF	0.018	61.1	26860	62.9	IMRF	0.046	283.9	28653	292.3
975 year	SCBF	0.056	940.6	28376	990.6	SMRF	0.041	1249.0	27765	1283.0
	OCBF	0.040	87.8	27952	92.7	IMRF	0.093	376.7	28321	401.4
1500 year	SCBF	0.064	1331.7	28290	1381.3	SMRF	0.043	1520.7	26897	1561.4
	OCBF	0.038	155.4	28884	161.3	IMRF	0.100	501.1	28485	550.0
2475 year	SCBF	0.115	1599.2	28057	1683.2	SMRF	0.096	1844.9	27657	1962.3
	OCBF	0.068	251.8	27703	269.9	IMRF	0.183	708.2	28023	910.0
5000 year	SCBF	0.164	1850.7	27265	1999.5	SMRF	0.130	2409.7	28419	2646.1
	OCBF	0.126	426.8	28142	493.8	IMRF	0.219	1114.2	28307	1645.2

Conclusion

Loss estimation results suggests that annual losses in an isolated braced frame will be about ¼ of those in a conventional braced frame, but annual losses in an isolated and conventional moment frame will be about the same. Stiffening and strengthening the isolated moment frame building, which was designed to code minimum, should be considered to reduce collapse losses and achieve the desired performance. The total economic impact of an earthquake could be much greater than predicted here when downtime and profit loss are included.

Acknowledgment

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References

Applied Technology Council, 2007. *ATC-58: Guidelines for Seismic Performance Assessment of Buildings* 35% Complete Draft, Washington, DC.

Deierlein, G., Victorsson, V., 2009. ATC 58 SPP: Fragility curves for components of steel SMF systems, 90% draft. Prepared for ATC-58 Project.

Engelhardt, M. D., Fry, G., Jones, S., Venti, M., and Holliday, S., 2000. Behavior and Design of Radius Cut Reduced Beam Section Connections, *SAC/BD-00/17*, SAC Joint Venture.

Gates, W. E. and Morden, M., 1995. Lessons from inspection, evaluation, repair and construction of welded steel moment frames following the Northridge Earthquake, *Survey and Assessment of Damage to Buildings Affected by the Northridge Earthquake of January 17, 1994*, SAC Rep. 95-06.

Gilton, C., Chi, B., Uang, C. M., 2000. Cyclic Response of RBS Moment Connections: Weak- Axis Configuration and Deep Column Effects, *SAC/BD-00/23*, SAC Joint Venture.

Krawinkler, H. (ed.), 2005. Van Nuys Hotel Building Testbed Report: Exercising Seismic Performance Assessment, *PEER Rep. No. 2005-11*, Pacific Earthquake Engineering Research Center, University of California, Berkeley.

Lignos, D. G. and Krawinkler, H., 2007. A database in support of modeling component deterioration for collapse prediction of steel frame structures", ASCE Structures Congress, Long Beach, CA.

Miranda, E., and Aslani, H., 2003. Probabilistic response assessment for building specific loss estimation, *PEER Rep. No. 2003-03*, Pacific Earthquake Engineering Research Center.

Mitrani-Reiser, J., 2007. *An Ounce of Prevention: Probabilistic Loss Estimation for Performance-Based Earthquake Engineering*, PhD Dissertation, Caltech.

Porter, K. A, Kiremidjian, A.S., Legrue, J. S., 2001. Assembly-based vulnerability of buildings and its use in performance evaluation, *Earthquake Spectra*, **17**, 291-312, EERI.

Ricles, J. M., Mao, C., Lu, L.-W., and Fisher, J. W., 2002. Development and Evaluation of Improved Details for Ductile Welded Unreinforced Flange Connections, *SAC/BD-00/24*, SAC Joint Venture.

RS Means Co., 2008. *Assemblies Cost Data*, 34th Edition, Kingston, MA.

Ryan, K. L., Erduran, E., Sayani, P. J., and Dao, N. D., 2010. Comparative seismic response of code designed conventional and base-isolated buildings to scenario events, *Proc. 9th U.S. National/10th Canadian Conference on Earthquake Engineering*, Paper No. 1617, Toronto, Canada.