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A CASE STUDY COST-BENEFIT ANALYSIS ON THE USE OF BASE ISOLATION IN A LOW-RISE OFFICE BUILDING

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ABSTRACT

Base isolation has been shown to be highly effective in mitigating earthquake damage in buildings. However, in the United States, implementation has essentially remained limited to key historic buildings and critical facilities such as hospitals. This paper presents a case study life cycle analysis of a conventional and base isolated braced steel office building. It is found that the overall performance of the base isolated building is far superior to the convetional building, but that the expected losses in the isolated building increase markedly if structural pounding occurs. The benefit-cost ratio for incorporating base isolation is found to be highly sensitive to the ability of the business to quickly and effectively migrate its business functions after a large earthquake.

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ABSTRACT

Base isolation has been shown to be highly effective in mitigating earthquake damage in buildings, but in New Zealand and the United States, implementation has remained limited to key historic buildings and critical facilities such as hospitals. This paper presents a case study life cycle analysis of a conventional and base isolated braced steel office building. It is found that the overall performance of the base isolated building is far superior to the convetional building, but that the expected losses in the isolated building increase markedly if structural pounding occurs. The benefit-cost ratio for incorporating base isolation is found to be highly sensitive to the ability of the business to quickly and effectively migrate its business functions after a large earthquake.

Introduction

Base isolation provides a proven and effective means of protecting structures from the damaging effects of horizontal ground motion. However, first cost increases and uncertainty about future benefits act as strong disincentives for building developers [1]. As such, implementation of base isolation in the United States has been largely limited to critical facilities such as hospitals, for which post-earthquake function is essential [2]. If the implementation of base isolation is to be extended to a wider class of structures (office buildings, apartment complexes, shopping malls and so on), the earthquake engineering community must be able to identify when base isolation represents appropriate and cost-effective earthquake risk management.

Expected value decision theory suggests that base isolation is appropriate and cost-effective when the expected dollar value of benefits from base isolation over the building life cycle outweighs the expected initial cost increase. Several previous studies have attempted to demonstrate the expected life-cycle benefits of base isolation in buildings. Early studies used a loss estimation approach based on the Modified Mercalli Intensity intensity scale [3, 4]. A series of later studies [e.g., 5, 6] estimated damage and life cycle costs using a damage index approach. Recently, two studies [7, 8] used the FEMA P-58 methodology [9] and Performance Assessment

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Calculation Tool (PACT) software to assess comparative financial performance of fixed base and isolated building typologies [10]. In these studies and others, the expected life cycle benefits are found to be significant and often outweigh the first cost increase where one is provided. However, the cost effectiveness of base isolation is found to be highly sensitive to the level of seismic hazard, the structural design details, the analysis time period and the discount rate, as well as whether or not earthquake insurance and business downtime are considered in the analysis.

The Network for Earthquake Engineering Simulation Tools for Isolation and Protective Systems (NEES TIPS) project was initiated in 2008 as a collaborative effort between researchers in the US and Japan to create and promote tools that will facilitate adoption of isolation and protective systems [11]. This paper presents a case study cost-benefit analysis on the use of base isolation in a low rise office building, that is an extension on previous NEES TIPS studies on comparative structural response [12, 13] and moat wall pounding [14].

Case Study Buildings

The three-story isolated and conventional office buildings investigated in this study are shown in Figure 1. They were designed by Forrell/Elsesser Engineers Inc. of San Fransisco for a location outside of Los Angeles, CA (34.50N, 118.2W). The site was assumed to be class D (Vs30 =270 m/s) with short period and one second spectral accelerations S_s=2.2g and S_1=0.74g. The conventional building was designed as a Special Concentrically Braced Frame (SCBF) with force reduction factor R=6 and the isolated building was designed as an Ordinary Concentrically Braced Frame with force reduction factor R=1. The design of both buildings was force controlled. The interstory height and bay width were 15 ft and 30 ft respectively. The isolation system was designed for an effective period and effective damping ratio of $[T_D, \beta_D]$ = [2.85 s, 21%] in the Design Basis Earthquake (DBE) event and $[T_M, \beta_M]$ = [3.10 s, 15%] in the Maximum Considered Earthquake (MCE) event. Readers interested in more detail about the two designs, including section sizes and isolator properties, are referred to [13]. The fundamental period of the conventional building T_1 was 0.41 s and the pre-yield and post-yield periods for the isolation system were 0.79 s and 3.55 s.

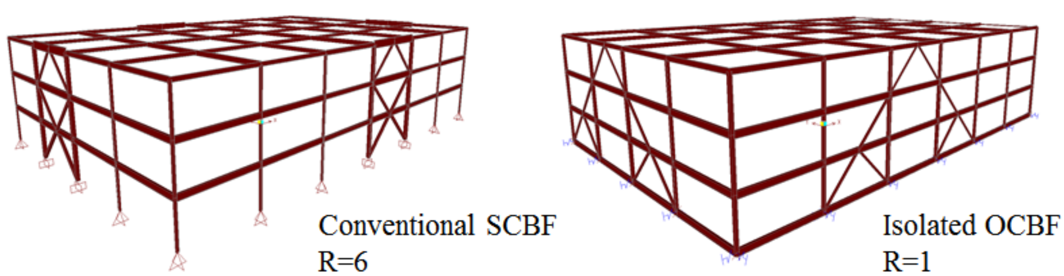

Figure 1. Three-dimensional renderings of the case study buildings

Detailed three-dimensional nonlinear finite element models for each structure were built in OpenSees, including variants of the isolated building with and without a moat wall. When included, the moat wall interaction model was based off [14], and used a seismic gap of 30.0 in, which was the MCE design displacement for the isolated building including torsion. The superstructure modeling was based on the use of fiber sections with steel stress-strain properties given by a Giuffré-Menegotto-Pinto model with a strain hardening ratio of 3%. Brace models were calibrated to match the experimental response reported by [15]. More information on the building modeling is available from [13].

The probabilistic life cycle cost analysis is based on response assessment of the as-designed hypothetical building, and thus uses site specific hazard spectra to determine the ground shaking in lieu of code design spectra. On the site class D soil, large differences existed between the MCE code spectra (to which the building had been designed) and the MCE hazard spectra (to which MCE ground motions are scaled). As a result, during response assessment a building on site class D in the long period range would be subjected to motions significantly larger than those for which it had been designed, while the opposite would be true in the short period range. These differences would have obscured a consistent estimate of comparative MCE performance. Furthermore, a class D soil with Vs30 =270 m/s is considered relatively soft in the context of base isolated building design, and so is not representative of 'most typical' design practice. To address these concerns, response assessment was conducted assuming the buildings was located on on site class C (Vs30 = 540 m/s) instead of site class D (Vs30 =270 m/s). As a result, the conventional building can be considered code compliant, while the isolated building can be considered to slightly exceed code requirements with an effective R = 0.87 on site class C soil.

Ground Motions

Ten discrete hazard levels were selected with corresponding intensities that provided even coverage of the intensity range of interest. The hazard data is summarized in Figure 2. Recorded ground motions were selected from the NGA database in four bins, one bin for the 1/10, 1/40 and 1/72 hazard levels, one bin for the 1/125 and 1/475 hazard levels, one bin for the 1/475 and 1/975 hazard levels, and a final bin for the 1/2475 and 1/4975 hazard levels, with different ground motions for the isolated and conventional buildings in the top two bins. Twenty bidirectional ground motions were selected for each bin. Scale factors for individual records typically ranged between 0.5 and 2.5.

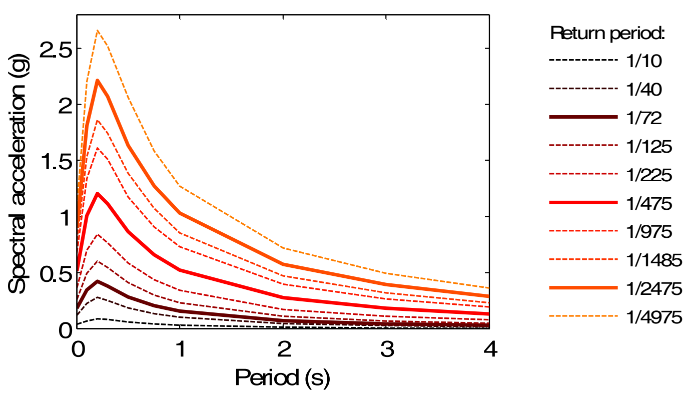

Figure 2. Uniform hazard spectra for the ten selected hazard levels.

Recent studies have emphasized that lower dispersion structural demand predictions can be achieved when there is a closer match between the response spectral shape of ground motions and the shape of the target hazard spectra [9, 16]. This is particularly true for base isolated buildings, because the isolation systems have an effective linear period that varies with displacement, and so are sensitive to spectral content over a large period range. Therefore, quality of spectral fit was adopted as a critical ground motion selection criterion in this study. At the lower hazard levels, a large number of earthquake records were available that fit the target spectrum well over the whole period range. However, at the higher hazard levels, very few earthquake records could be identified that closely fit the target spectra over the whole period range. To address this problem, at the five highest hazard levels, different ground motions were selected for the base isolated and conventional buildings. Specifically, the base isolated building records were selected according to the closeness of fit to a target conditional mean spectrum constructed at 3.0 s period, over a period range of $0.5T_D$ (1.425 s) to $1.25T_D$ (3.875 s) [17], and the conventional building records were selected according to the closeness of fit to a conditional mean spectrum constructed at 0.5 s period, over a period range of $0.2T_1$ (0.082 s) to $2.0T_1$ (0.82 s) [9]. The 0.5 s period was selected to be slightly higher than the fundamental period (0.41 s) to account for period elongation upon yielding and buckling of braces, which is expected at the higher hazard levels. The selection of independent records for the conventional and isolated buildings led to records with far better spectral matches over the period ranges of interest, which ultimately created more accurate (lower dispersion) demand predictions. The final bins contained motions that were roughly representative of the magnitudes and distances that contribute most to overall seismic hazard, as well as motions with and without near fault effects.

Structural Response

For each building and each hazard level, the 20 scaled ground motions from the corresponding bin were applied with x- and y-components aligned to the global x- and y-axes in the structural model, and then again at a 90˚ orientation. Median response profiles of peak vector floor acceleration and peak x-direction interstory drift over height are shown in Figure 3 for the conventional SCBF building, the isolated OCBF building with a moat wall and the isolated OCBF building without a moat wall. As expected, the decoupling action of base isolation typically allows for significant reductions in both peak vector floor acceleration and peak x-direction interstory drift.

Pounding of the base isolated building against the surrounding moat wall occurred at the 1/1485 hazard level (1 simulation out of 40), the 1/2475 hazard level (13 simulations out of 40) and the 1/4975 hazard level (31 simulations out of 40). Pounding typically caused superstructure peak floor accelerations to increase to between 0.5 g and 1.5 g on the upper stories and 0.5 g and 2.0 g on the first story, with 5 simulations at the 1/4975 hazard level out of 40 producing first floor accelerations in excess of 3.5 g. Pounding-induced amplification of peak floor acceleration at the 1/4975 year hazard level is clearly apparent in Figure 3. Pounding-induced amplification of peak interstory drift demand was observed, but was found to be highly sensitive to the approach velocity and torsional displacement at the time of impact. The superior 1/4975 year performance of the isolated building with no moat wall is achieved at the expense of high displacements (as high as 50-60 inches). This performance should be considered as a “best case scenario” on account of potential for displacement-induced isolator failiure.

Simulations with high transient interstory drift were observed in both the conventional building (up to 5.7%) and the isolated building with a moat wall (up to 3.6%). However, none of the simulations predicted a full structural collapse, which may be attributed to the tight spectral fit of selected ground motions (such that few ground motions had spectral content significantly exceeding the MCE design spectra) and the fact that post-peak strength degradation was not incorporated into the OpenSees models. Collapse probabilities were not included in further financial analysis. However, probabilities of the building receiving an unsafe placard or requiring replacement on account of excessive residual drifts were included. Unsafe placard probabilities were assessed using the default PACT method [9], whereby an unsafe placard is issued if a threshold number of components in the structural system reach a prescribed damage state. Replacement probabilities were assessed using a residual drift-based repair-replace fragility curve with a median of 1.0% and a lognormal dispersion of 0.3. A simple pilot study showed that structural collapse was unlikely to have a significant impact on the financial loss analysis because its influence was likely to be enveloped by the influence of residual-drift enforced replacement. In other words, in those simulations that estimated drifts high enough to put the building at risk of structural collapse, the building was found to require replacement in any case, which has a comparable cost consequence.

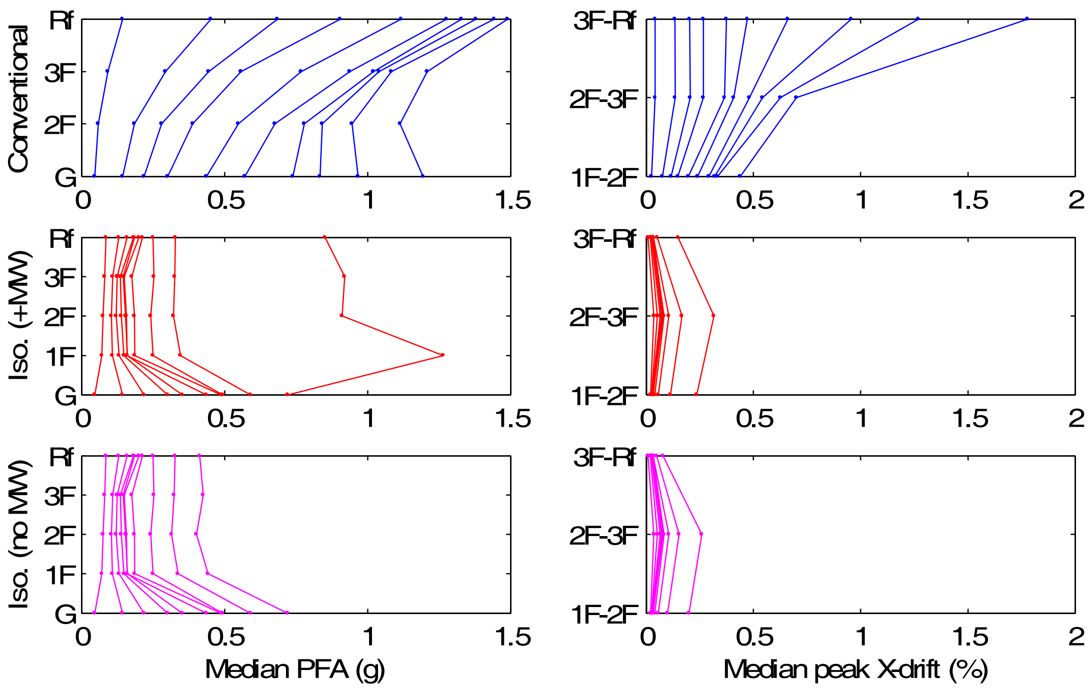

Figure 3. Profiles of median values of Peak Floor Acceleration (PFA) and peak X-direction story drift, for each of the ten hazard levels plotted from left to right in order of increasing hazard intensity.

Damageable Component Inventory, Fragility and Cost data

The damageable component inventory included structural components (OCBF and SCBF braces, pre-Northridge moment connections and gravity connections), non-structural components (monolithic exterior glazing, fully fixed interior partition walls, a suspended ceiling system segmented into areas between 250 ft^2 and 1000 ft^2, two traction elevators, electrical and plumbing distribution, a fire sprinkler system, roof-mounted HVAC equipment and a transformer) and contents (desktop computers and workstation desks). The quanities of each inventory item were based on the normative quantities suggested in Appendix F of FEMA P-58-1 [9]. Whenever possible, the fragility, repair cost and repair time data for each component was assumed to take on the default values suggested in the PACT fragility specification manager [10]. User input was required in some instances; for example the median cost of desktop computers and workstation desks was taken as $1800 and $1000, respectively. Lastly, the fragility models for the OCBF and SCBF braces were slightly modified. Specifically, the drift demand input for both OCBF and SCBF braces was set to a negligible value when the model predicted that stresses in the brace (at any point) had not reached within 2% of the brace yield stress. This modification was made to prevent erroneous realizations of damage state 1 (initiation of buckling and some yielding) predicted by the default PACT fragility when the model predicted that the brace had not buckled and was not close to yielding.

Loss Analysis

Demand parameters from nonlinear time history analyses, seismic hazard data and component inventories were assembled and input into PACT software [10]. Performance evaluations were then conducted for each building. These involved the generation of 300 Monte Carlo performance realizations at each hazard level and integration over the intensity range of interest to determine key expected annual values [9]. Table 1 shows the key expected value outputs for each building, where expected annual repair times are the average value of parallel and series repair strategies over the building floors. The superior performance of the isolated building types is highlighted by significant reductions in expected annual repair cost and repair time, and decreased likelihoods of receiving an unsafe placard or requiring replacement.

Table 1. Key outputs from the PACT analysis. Expected annual repair times are based on the average of parallel and series repair strategies over the building floors.

Value	**Conventional SCBF**	**Isolated OCBF (with moat wall)**	**Isolated OCBF (no moat wall)**
Expected annual repair cost	$20,500	$2,030	$156
Expected annual repair time	1.26 days	0.061 days	0.018 days
Unsafe placard return period	230 years	19000 years	N/A
Replacement return period	7800 years	46000 years	N/A

Earthquakes of all different hazard levels contributed relatively evenly to the overall expected annual loss in the conventional SCBF building – apart from the 1/10 hazard level, all contributions were between 7% and 15%. However, in the isolated building with a moat wall, expected annual loss was dominated by the 1/4975 hazard level, with an 86% overall contribution. This suggests an important role of structural pounding in modeling the financial performance of base isolated structures.

Interior partitions, ceiling systems, fire sprinkler systems, desktop computers and roof-mounted HVAC equipment all contributed significantly to overall losses in the conventional SCBF building, with steel connections and steel braces becoming more influential at the higher hazard levels. Similar contribution patterns were observed in the isolated OCBF building when structural pounding occurred. However, when structural pounding did not occur, the minor financial losses observed in the isolated buildings were dominated by the contribution of minor damage to interior partitions. In particular, these damage realizations were generated from the extreme lower tail of the partition damage state 1 fragility curve, where exceedence probabilities were at most 8%. Accordingly, partition losses are expected to be more significant if the isolated building superstructure is designed for R > 1 (generally permitted by code) or has a more flexible lateral load resisting system compared to the building examined here [8].

Repair cost-intensity relationships for each building type are shown in Figure 4. In the conventional SCBF, expected repair costs increase almost linearly with spectral acceleration after a threshold spectral acceleration of about 0.2g is reached. Conversely, in the isolated OCBF building, expected losses remain minimal until spectral accelerations well above DBE level. Losses in the isolated building become particularly significant at the 1/4975 hazard level, where response is dominated by moat wall pounding (see losses in Figure 4b for $S_A(3.0s) > 0.392$ g).

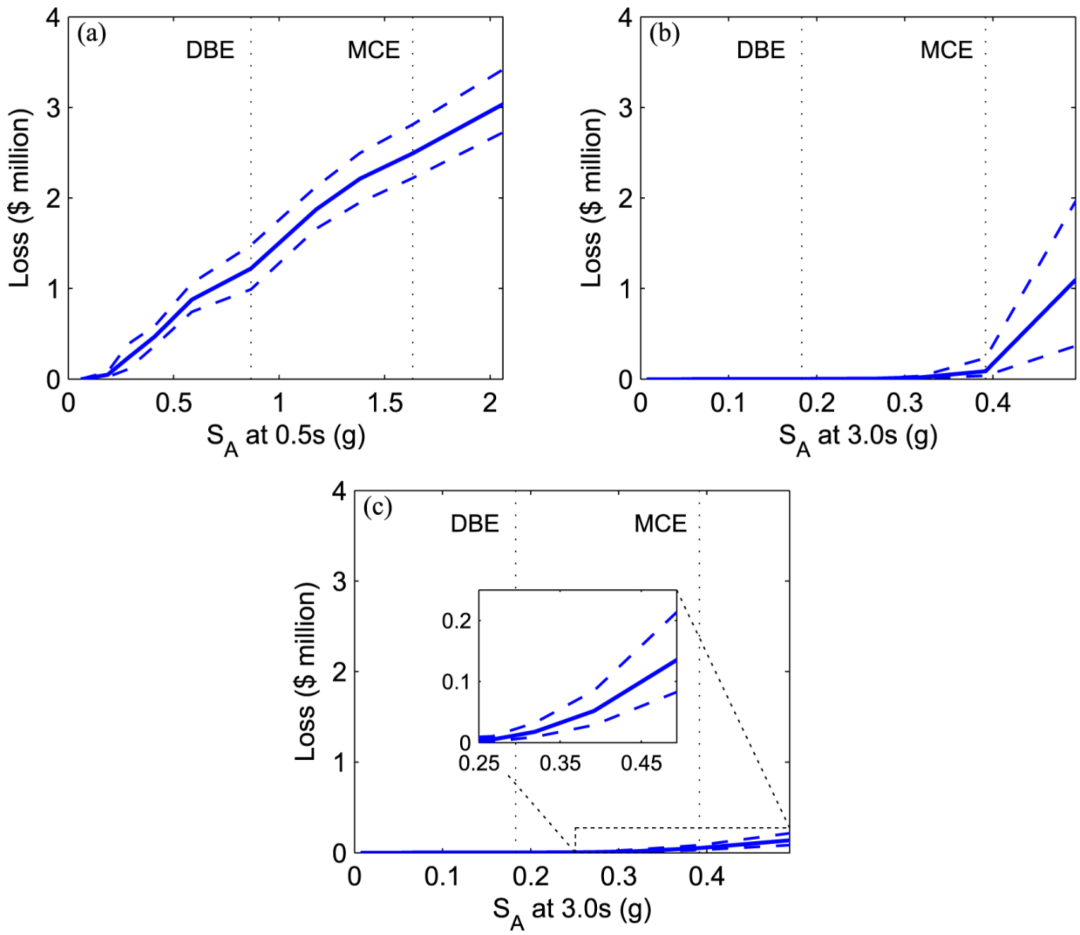

Figure 4. Median and ±1σ repair costs vs. spectral acceleration for (a) the conventional SCBF, (b) the isolated OCBF with a moat wall and (c) the isolated OCBF without a moat wall, given building replacement is not required.

Business interruption losses were calculated for each Monte Carlo realization based on a decision tree approach. If the building had received an unsafe placard and required replacement, then the total time to reoccupancy was assigned as a random variate that ranged between 2 and 4 years [18]. If the building received an unsafe placard but did not require replacement, business downtime was set equal to the repair time estimated from PACT plus a planning period immediately after the earthquake that was assumed to be a random variate of a lognormal distribution with a median of three months and a dispersion of 0.4 [18, 19]. Lastly, if the building did not receive an unsafe tag, then business downtime was assumed to equal the repair time from PACT but only for those damage occurrences expected to cause a loss of occupancy (loss of occupancy criteria were based on Table 4 from [20]). In all cases, business interruption loss was taken as downtime multiplied by the building income, assumed to be $60,000 per day (Table 15.15 from [19]). Expected annual repair times are based on the average of parallel and series repair strategies over the building floors. The resulting increase in expected annual loss is shown in Table 2. This increase should be considered a "worst case scenario" in that it neglects both insurance and business relocation.

Table 2. Key outputs from the PACT analysis.

Value	**Conventional SCBF**	**Isolated OCBF (with moat wall)**	**Isolated OCBF (no moat wall)**
Expected annual loss (no business interruption)	$20,500	$2,030	$156
Expected annual loss (with business interruption)	$160,000	$6,000	$200

Expected Cost-Benefit Analysis

The difference between the expected annual loss in the base isolated and conventional structures (ΔEAL) may be interpreted as an "expected annual benefit" for using base isolation. The expected benefit over some arbitrary time period $E[B(t_{\max})]$ may be calculated from ΔEAL using the following equation:

$$\mathrm{E}[B(t_{max})] = \begin{cases} \Delta\mathrm{EAL}\; t_{max} & r = 0 \\ \Delta\mathrm{EAL}\left(\dfrac{1 - e^{-rt_{max}}}{r}\right) & r > 0 \end{cases} \qquad (1)$$

where $t_{\max}$ is the time period and r is the continuously compounding, real Discount Rate (DR). The DR discounts future cash flows to an equivalent present value and typically ranges between 2% and 7% for earthquake engineering purposes [21].

The raw first cost of the conventional SCBF structure is expected to be about $24.1 million, based on a professional cost estimate. Assuming that the first cost increase to incorporate isolation is 5% of the building first cost, the time period is 50 years and the discount rate is 4%, then the expected benefit-cost ratio for investment in base isolation (including a moat wall) is 3.1 when business downtime is included and 0.4 if business downtime is not included. This suggests that the appropriateness of isolation in the current study is influenced strongly by the ability of the business to transfer risk of business interruption loss. Figure 5 shows how the expected benefit-cost ratios change with time period, discount rate and first cost increase to incorporate isolation, assuming the time period may be anywhere between 20 and 150 years, the discount rate may be anywhere between 2% and 7% and the first cost increase might be anywhere between 3% and 10%. Changing assumptions regarding any of these three parameters can cause significant changes in the benefit-cost ratio.

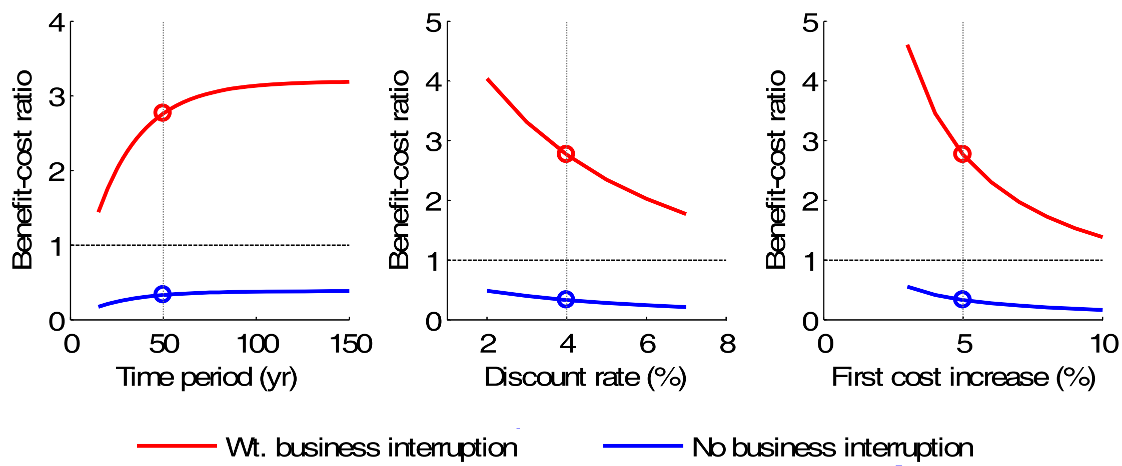

Figure 5. Sensitivity of benefit-cost ratio results to changes in time period, discount rate and first cost to incorporate isolation.

Conclusions

The FEMA P-58 methodology has been applied to a conventional low-rise, braced steel frame building and an equivalent isolated building with and without a moat wall. Overall, the performance of the isolated building models was far superior to the conventional building model; however, this performance degraded somewhat in the unlikely event of structural pounding against the building's moat wall. The expected financial benefits provided by the isolation system over the building life cycle were found to be significant, but highly sensitive to assumptions about business interruption. An investment in seismic isolation for the current building typologies may be easily justified when the business has a high income and/or the business is unable to quickly and effectively migrate its business function to an offsite location.

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