Detailed Seismic Loss Estimation for a Tall Building in Japan

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SUMMARY:
Seismic loss analysis is becoming an increasingly routine aspect of financial and social decision-making related to buildings at risk of earthquake-induced damage whether it be at the design stage or at a later time in the building's lifespan. As these analyses become more commonplace and are used for critical decision-making, it is important to have rigorous and consistent methods for assessing damage to both common and unique building types. Additionally, in practice, these methods need to be computationally and financially tractable so that they can be applied within limiting budget and schedule constraints. Moreover, the methods must also be commensurate with the quality and extent of the data made available by the client, which, in the authors' experience is generally limited. The rigor of the analysis also needs to be commensurate with the particular objective of the analysis, for example insurance purchase, development of optimal physical risk mitigation options, operational considerations, etc. Thus, the authors view the application of seismic loss analysis as a balance between practical feasibility and the incorporation of the latest research and recommendations that have been shown to most strongly influence the loss estimates such that the results are reliable, defensible, and allow the client to utilize them for the purposes set forth in the study objectives. With this in mind, this paper discusses a recently completed detailed building-specific seismic loss estimation study that considered a tall, 24-story steel-framed building in Osaka, Japan. The paper describes the methodology applied to this study, which balances detail and comprehensiveness with computational and financial efficiency and includes a discussion on the benefits and limitations of the methodology. The method combines different analysis techniques to provide a manageable, computationally efficient way of computing seismic performance in terms of quantitative metrics, namely monetary loss.

Keywords: earthquake loss estimation; seismic performance assessment; Japanese seismic design; tall steel-framed building;

1. INTRODUCTION

The vast majority of seismic loss estimation studies that are performed on portfolios of buildings, typically for (re)insurance purposes, utilize commercially available catastrophe loss estimation software. The software provides defensible industry average loss estimates for "typical" buildings distinguished by broad characteristics such as construction material, structural type, height, occupancy, date of construction, among others. Because these commercial models seek to capture industry average losses, the models are better suited to estimate losses for classes of buildings that dominate the industry exposure in a region, e.g. low-to-mid height buildings with common structural systems and materials. This approach is reasonable for large portfolios which are expected to reflect industry exposure averages, however, for small portfolios or for portfolios dominated by highly protected risks or unique buildings and other structures, this approach can result in markedly different loss estimates than if the losses were estimated in a more building-specific manner. The building studied in this paper is part of one such small portfolio of buildings and other structures located throughout Japan. Based on results from commercially available catastrophe loss estimation software, this building was found to disproportionately dominate the seismic risk of the entire portfolio. This combined with the owner's specific objective of developing a well-informed catastrophe risk mitigation program for this building motivated a building-specific loss assessment to drive improved understanding, reliability, and defensibility in the seismic loss estimates for the building.

There are several methods that are currently available for loss estimation of tall buildings. Each method varies in complexity and the level of detail at which the individual building characteristics are captured. At the simpler end of the spectrum, regional loss estimation methods have been developed to compute losses for large portfolios of buildings. These methods consider mean building characteristics and relate ground motion intensity to building damage considering average structural response for the given building characteristics. These methods are efficient and easy-to-use but may overlook individual buildings' specific characteristics that may strongly affect the estimated damage for atypical buildings. One of the most widely used regional loss estimation methods, which are available in the public domain, is the HAZUS methodology and software (Whitman et al. 1997, NIBS, 1997). Proprietary methodology and software are also commercially available from various companies.

At the other end of the spectrum, component-based loss estimation methods attempt to compute losses by estimating the earthquake damage to each component in a building. Because of the extensive details that these methods attempt to capture, the data requirements and computational demands are high. For example, these methods require explicit structural modeling to relate ground motion intensity to structural demand parameters which is then related to component damage. In recent years, the Pacific Earthquake Engineering Research (PEER) center and the Applied Technology Council (ATC) have developed a component-based loss estimation method that is widely considered as the future of performance-based earthquake engineering (ATC-58).

In this study, the proposed approach aims to balance complexity with efficiency by drawing on different aspects of both the PEER/ATC-58 and HAZUS methods. The approach includes incremental dynamic analysis (or IDA; Vamvatsikos, 2002) to probabilistically predict collapse and determine structural demand parameters, such as story drift and floor accelerations, in non-collapse scenarios. The structural demand parameters are supplemented with fragility function information provided by the HAZUS methodology to estimate damage and corresponding monetary loss; the fragility function is modified to reflect building and region specific characteristics. The site-specific earthquake hazard is probabilistically captured using a stochastic catalog of simulated seismic events previously developed by research staff at AIR Worldwide.

This study presents an approach that attempts to balance the computationally intensive nature of structure-specific loss estimation of tall buildings with the practical constraints on resources that are encountered in industry applications, while enhancing the reliability of the estimated losses. The approach draws heavily on previous research on performance based earthquake engineering and is applied to a real case-study building in Osaka, Japan. The resulting loss estimates are presented to illustrate the approach's use and to discuss the practical challenges faced during its application.

2. DESCRIPTION OF THE CASE-STUDY BUILDING

The building presented in this paper is a 90m, 24-story steel-framed building constructed in 1994. The lateral load resisting systems in both primary directions comprise of steel moment frames and steel braced frames. The primary lateral load resistance of the building is derived from the steel moment frames; the incorporation of one bay withbraces increases the first-mode elastic stiffness only by approximately 15% and the first-mode strength only by approximately 10%. The columns are square cold formed steel tubes, the beams are Japanese H-section (i.e. wide flange) and the braces are a combination of square tubes and H-sections. All beam-column connections in both directions are moment-resisting, as is common practice in Japan.
Seismic design provisions in the United States (e.g. the Uniform Building Code) and Japan (e.g. the BCJ) both require that buildings respond elastically for small, frequent earthquakes but allow for an inelastic response during large, infrequent earthquakes so long as global stability is maintained. Moreover, both provisions prescribe seismic load reductions to account for structural ductility (Nakashima, 2000). Despite these similarities, there are a few differences in design methodologies

between the two regions that result in some important differences in performance expectations between Japanese and United States buildings. Consequently, the publicly available loss estimation methodologies, which are primarily developed for US designs, need to be modified to be applied to the Japanese designs.

Because this paper is focused on a tall steel-framed building with steel moment frames as the primary load-resisting system, the differences described below focus on steel-framed, long-period (i.e. T > 0.7s) structures for which the design (i.e. the proportioning of structural members) is controlled by prescribed drift limits.

In Japan, the elastic interstory drifts are restricted to less than 0.5% under the reduced design forces. In the US, the drift limit is imposed on the inelastic drift expected under the non-reduced design forces, which, for tall structures with standard occupancies, is equal to 2.0%. Because Japan places a limit on the elastic drift and the US places a limit on the inelastic drift, a direct comparison between the two provisions cannot be carried out. However, Nakashima (2000), invoking the constant displacement hypothesis, estimates that Japanese drift-controlled moment-frame buildings are ~15% stiffer and stronger than comparable buildings in the US designed to the same non-reduced lateral forces. Other important differences in the design provisions and practice are (1) the slenderness limits which tend to be less restrictive in Japan than in the United States where building ductility is more relied upon, (2) typical system redundancy which is typically greater in Japan, and (3) typical beam depths which tend to be shallower than is common in the United States (Nakashima, 2000). Japanese moment frames typically have full moment-resisting connections at all beam-to-column connections (Nakashima, 2000) as opposed to US moment frame design in which moment resisting frames are typically planar frames located on the perimeter of the building with beams and columns made from wide-flange cross-sections. Both Japanese and US designs typically prescribe yielding in beams before columns.

It is commonly observed that moment frames in the US typically yield at an interstory drift ratio of 1.0-1.2% (Aschheim, 2002) regardless of the strength and stiffness of the moment frame structure, which is not necessarily the case for moment frames in Japan. Publicly available structural fragilities (e.g. HAZUS) for moment frames in the US, which relate building drift to a probability of a particular damage state, reflect this expected behavior for US designed moment frames. For this reason, US-based structural fragilities needed to be modified to be applied to the case-study building.

3. SEISMIC HAZARD AND STRUCTURAL RESPONSE ANALYSIS

The probabilistic site-specific seismic hazard curve for the building is shown in **Error! Reference source not found.**, which is developed using a time-dependent 10,000-year stochastic catalog of earthquake events based on internal AIR research, recent scientific studies, and the recommendations of Japan’s Headquarters for Earthquake Research and Promotion (HERP). Access to such stochastic catalogs may not be readily available for all risk assessments. In such cases, depending on the scope of the study, one can use published information for the building location such as probabilistic maps developed by agencies such as the United States Geological Survey or regional equivalents to develop a site-specific hazard curve or conduct a probabilistic seismic hazard analysis (PSHA) for the site. The reliability of the site-specific hazard curve directly impacts the reliability of the overall loss estimation.

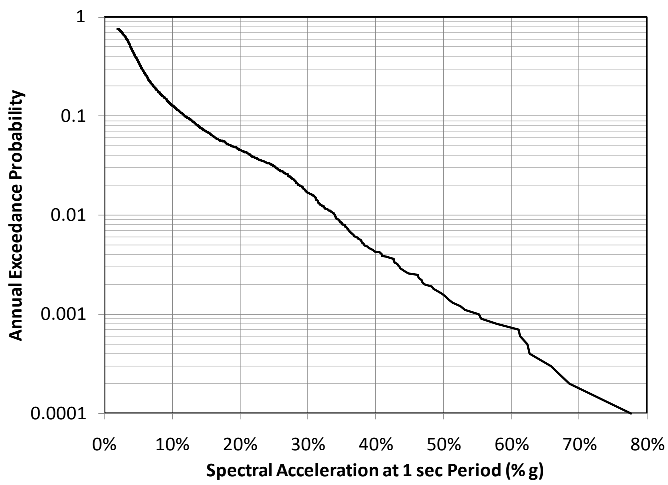

Figure 1. Probabilistic hazard curve for the case study building.

Incremental dynamic analysis (IDA) is a common tool for assessing the vulnerability of buildings to earthquake ground-shaking for the purposes of probabilistic risk analysis. While an IDA based on three dimensional structural models of the building is the most comprehensive route to assess the expected performance of a building, IDA using three dimensional models and employing a large suite of ground motions that have to be analysed at multiple scaled intensities can be resource intensive especially for tall buildings where the models can have many thousands of degrees of freedom. To circumvent this problem, this paper proposes a methodology that attempts to combine useful information from a detailed three dimensional model with the low computational demands of a “stick” model. In this context, the term “stick” model refers to a model that is constricted to two lateral degrees of freedom per story.

The first step in the proposed methodology is to create a three dimensional structural model, which, for the case-study building, includes over 43,000 degrees of freedom. For this particular building model, perfectly plastic hinges (i.e. no hardening) were defined at the ends of all beams and columns and in braces wherein the braces can develop full yield capacity in tension and one-half of the full-yield capacity in compression;the floor diaphragms were modelled as rigid. The elastic-perfectly plastic hinge behavior assigned to the beams and columns has been shown to be appropriate for wide-flange cross-sections. Applying these hinge models to the box cross-section columns in a Japanese building, is questionable. However, as noted previously, Japanese buildings are designed such that inelasticity occurs first in beams rather than columns so this design philosophy somewhat mitigates the inaccuracy of the hinge model. The modeling of the braces is similarly simplistic, however, as noted in the previous section, the primary lateral load resistance is derived from the moment frames as opposed to the braced frames. The complexity of the model needs to be commensurate with the range of expected behaviour for the building, i.e., if the expected response is not overly inelastic or mechanism formation is expected then simplification in the element modelling is appropriate.

In the second step, nonlinear static pushovers are conducted on the three-dimensional model. The pushovers are continued until the onset of numerical convergence issues. For this case study, the convergence of each pushover analysis failed at a building drift of just over 1.0%. The maximum story drift under this condition was about 1.3% which occurred near mid-height of the building. The applied load pattern was the distribution of lateral forces per the Japanese Design Code (BCJ), as shown in Figure 2 which also compares this shape with the first three mode shapes in the longitudinal direction of the three-dimensional model. The figure shows that the lateral force distribution does not correspond to any single mode shape.

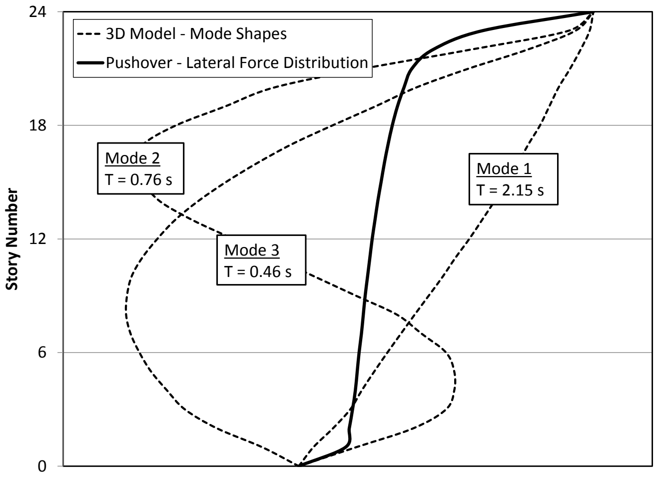

Figure 2. Comparison between the first three mode shapes in the longitudinal direction of the three-dimensional model and the lateral force pattern for the pushover analysis.

During the pushover analysis, the shear force and inter-story displacement for each story is recorded, which is used to develop the force (story shear) -displacement (interstory drift) relationship for the stories in the stick model. The stick model has two degrees of freedom corresponding to the lateral displacement in two primary directions at each story, and thus, for the case study building, the stick model has 48 degrees of freedom. Thus, for this building, the degrees of freedom in the model reduce from 43,000 in the three dimensional model to 48 in the stick model. Story shear stiffness at interstory displacements greater than those calculated in the pushover analysis are assumed to be zero. This is a notable approximation because the average shear stiffness for each story at the final step of the pushover analyses was 32% of the initial stiffness for that story (the maximum and minimum were 20% and 60%).

After defining the dynamic characteristics of the stick model, a set of ground motions is selected to represent a range of potential ground shaking at the site. For this particular study, the suite of 10% in 50 year ground motions time histories for firm soils in the Los Angeles region developed as part of the SAC Joint Venture Steel Project Phase 2 were applied (SAC Joint Venture Steel Project Phase 2, 1997). This suite of motions includes 10 sets of ground motion pairs which provide consistent ground motions for two perpendicular directions. Each ground motion is then scaled to a consistent intensity of spectral acceleration at 1-sec period, and, in total, eight intensities are considered (5.0%, 7.5%, 10%, 25%, 50%, 75%, 100% and 150% of gravity). The ground motions were scaled at the 1-sec period rather than at the first period of the structure because the probabilistic hazard was only available for peak ground acceleration and at spectral accelerations corresponding to periods of 0.3 and 1.0 seconds. The 1.0 second spectral acceleration was the most appropriate considering the first mode period of the structure.

The stick model is then analyzed using nonlinear dynamic analysis for each pair of scaled ground motions. The maximum interstory drift and absolute acceleration is recorded for each story and primary direction of the building. Figure 3 shows the median value of the maximum interstory drift and floor acceleration for each scaled ground motion intensity.

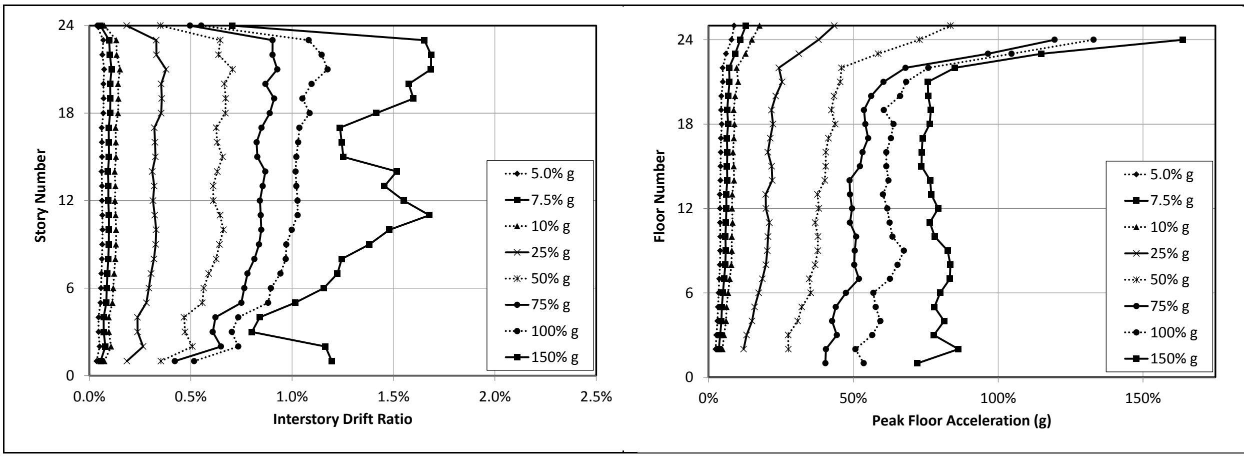

Figure 3. Distribution of median interstory drift (left) and median absolute floor acceleration (right) from IDAs.

Conducting the IDA on a stick model rather than a detailed three-dimensional model clearly has some limitations. The elastic dynamic characteristics of the stick model are based on the lateral story shear stiffness of the three dimensional model with applied load pattern per the distribution of lateral forces defined by the Japanese seismic code. It is well known that the dynamic response of tall buildings include significant contributions from higher modes and thus a comparison of the modal characteristics of the three dimensional and stick models is important. Figure 4 compares the first three mode shapes in one direction for the two models; the stick model seems to reasonably represent the first three modal characteristics of the three dimensional model. This comparison is encouraging for the elastic dynamic characteristics, however, it is possible that the inelastic dynamic characteristics differ between the two models and this is a notable limitation of the methodology. Moreover, the stick model cannot consider the effect of interaction due to simultaneous loading in both primary directions.

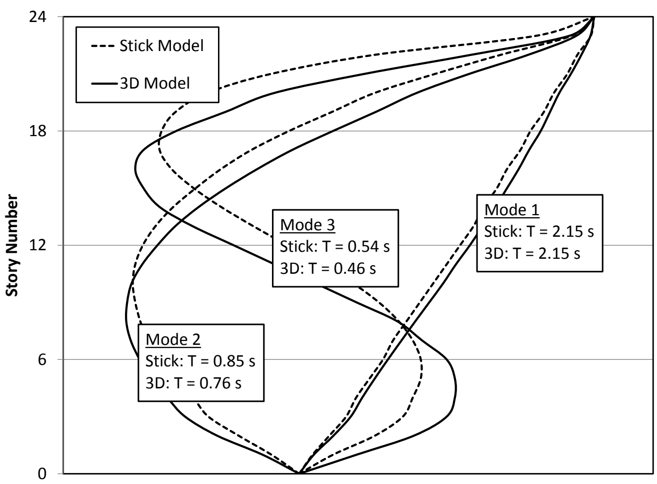

Figure 4. First three elastic mode shapes and corresponding periods for the 3D model and the stick model.

4. LOSS ESTIMATION APPROACH AND LOSS RESULTS

The loss estimation approach implemented in this study is primarily based on the simplified story-based building-specific loss estimation methods proposed by Zareian and Krawinkler (2009) and Ramirez and Miranda (2009). To develop the building-specific relationship that connects ground motion intensity to economic monetary loss (i.e. the building-specific damage function), the overriding equation (Aslani, 2005) was used:

$$E[L_T|IM] = E[L_T|NC, IM]P(NC|IM) + E[L_T|C]P(C|IM) \tag{1}$$

where $E[L_T|NC, IM]$ is the expected loss in the building given that collapse has not occurred for ground motions with an intensity level of *IM*, $E[L_T|C]$ is the expected loss given that the building has collapsed, $P(NC|IM)$ is the probability that the building does not collapse when subjected to a given ground motion intensity, *IM*, and $P(C|IM)$ is the probability that the structure collapses conditioned on *IM* (which is complimentary to $P(NC|IM)$, that is, $P(NC|IM) = 1 - P(C|IM)$).

Equation (1) relies on the ability compute the probability of the building collapse, $P(C|IM)$. Recent studies in nonlinear structural simulation combined with recent advances in computing ability, have improved methods and techniques that quantify this parameter (e.g., Haselton et. Al. 2007, Liel et al. 2008). In this study, $P(C|IM)$ is determined using the results of the IDAs by establishing a collapse criteria based on maximum interstory drift. For each ground motion record used in the IDAs, the building was assumed to have collapsed if the interstory drift in any story exceeded 10%. At this level of deformation, it is assumed that the building will not able to recover to a stable position and side-sway collapse will initiate. It is recognized that estimating the probability of collapse has many limitations. The dynamics of collapse can be very difficult to capture as a structure can be re-centered and re-stabilized by inertial effects and by changing directions of the ground motion excitation. Further, the PEER Tall buildings guidelines suggest that story drifts greater than 4.5% should not be trusted (PEER, 2011). Although estimating collapse has many challenges, what was implemented in this study is consistent with what is widely considered the best methods of predicting collapse.

The losses conditioned on non-collapse, $E[L_T|NC, IM]$ in equation (1), can be computed using a story-based approach as documented in detail by Ramirez and Miranda (2009). This is accomplished by grouping individual component losses per story and pre-computing estimated damage using assumed cost distributions of the total story value. This can be done using the following equations:

$$E[L_T|NC, IM] = \sum_{i=1}^{NS} \sum_{k=1}^{N=3} E\left[L_{i,k}|NC, IM\right] \tag{2}$$

$$E\left[L_{i,k}|NC, IM\right] = \int_0^\infty E\left[L_{i,k}|NC, EDP_k\right]|dP(EDP_k > edp_k|NC, IM)| \tag{3}$$

where $E\left[L_{i,k}|NC, IM\right]$ is the expected loss at the *i*th story and the *k*th component category (e.g. drift-sensitive, structural components, etc.) conditioned on non-collapse and seismic intensity level, *IM*, $E\left[L_{i,k}|NC, EDP_k\right]$ is the expected loss at the *i*th story and the *k*th component category conditioned on non-collapse and the *EDP* associated with the *k*th component category, and $P(EDP_k > edp_k|NC, IM)$ is the probability of EDP_k exceeding the value of edp_k conditioned on non-collapse and a given level of seismic intensity, *IM*. In this study, the value for $E\left[L_{i,k}|NC, EDP_k\right]$ is obtained from story damage functions and the probability distributions for $P(EDP_k > edp_k|NC, IM)$ are computed from the IDAs.

The story-based approach defined in equations (2) and (3) requires that the replacement value of the entire building be distributed among each story and each type of building component in the structure. For the case-study building, a complete inventory of components was not available so assumptions on how the replacement value was distributed among its stories and components had to be made. Although it was recognized that the 1st story and roof may have different values than the typical floors in between, it was assumed the total value was uniformly distributed across all the stories. Since tall buildings have many stories, even if the difference in value of the 1st story and roof is significant, it represents a small percentage of the overall value and consequently will have a small influence on the total loss results.

Consistent with an approach demonstrated by Zareian and Krawinkler (2009) and Ramirez and Miranda (2009), components were grouped into broad categories. In this case, three broad categories of components were considered: (1) drift-sensitive structural components (2) drift-sensitive non-structural components, and (3) acceleration-sensitive components. Each story's value was distributed into these three categories using value data provided by the client and available construction cost data (RS Means), resulting in the following value breakdown:

Drift-sensitive, structural components	20%
Drift-sensitive, non-structural components	40%
Acceleration-sensitive, non-structural components	40%

While the assumed grouping of components is simple, it is important to note that detailed cost distribution inventory data for individual buildings is rarely available. Even within the context of insurance-related risk assessment consulting, it has been the authors' experience that the owner is unable to provide building inventory value broken down by individual components. For this particular project, the client's documentation of the structural design was impressively extensive and the client was willing to fund a state-of-the-art loss analysis. However, even in this case, the client was unable to provide the cost data to conduct a component-based loss analysis.

Story-based loss estimation requires story damage functions that relate engineering demand parameters to the monetary loss of the entire story. Previous work has been done to develop these types of functions (Ramirez and Miranda, 2009), however, these functions are dependent on occupancy and building usage. The functions reported by Ramirez and Miranda (2009) were developed using construction cost distributions for typical commercial office buildings. Unfortunately, these could not be applied to the case-study building since it is not an office building, and thus it was decided to derive custom story damage functions based on the building-level fragility functions provided by HAZUS. For some components, the HAZUS functions were applied directly, while for others the functions were adjusted to account for the differences in the building design between Japan and the US, and the unique structural response determined from the nonlinear analysis of the three dimensional model.

The fragility function for drift-sensitive, structural components used in this study was based on the original function for high-rise steel braced-frame buildings (HAZUS code S2H) designed to a "high-code" level. These were then modified based on the story force-drift relationships computed from the nonlinear static analysis of the 3D model built for the 24-story building. These modifications were necessary because of the differences between design standards and philosophies in Japan and the US, and because of the unique structural characteristics of tall buildings. For comparison, the assumed building height and fundamental period of the HAZUS generic structure for S2H are 48 meters and 1.77sec, respectively, whereas the corresponding values for the case study building are 90m and 2.15sec. The selected statistical parameters for the drift-sensitive structural components and the original HAZUS functions for each damage state are listed in Table 1. Figure 5 (left) shows the resulting story damage functions when the fragility functions and the damage ratio are integrated.

Table 1. Median and standard deviation of the HAZUS structural fragility and the customized structural fragility for the case-study building.

	HAZUS		**Customized**	
Damage State	**Median (interstory drift)**	**Beta**	**Median (interstory drift)**	**Beta**
Slight	0.25%	0.63	0.45%	0.40
Moderate	0.50%	0.63	0.90%	0.40
Extensive	1.50%	0.64	1.80%	0.40
Complete	4.00%	0.71	3.60%	0.40

Damage to the drift-sensitive and acceleration-sensitive non-structural components was estimated directly using fragility functions that were based on the building-level relationships published by HAZUS. These functions, however, were not modified because it was not expected that there would be any substantial differences between the vulnerability of non-structural components in a tall building and the vulnerability of these types of components in any other building. Additionally, there were no significant differences expected between non-structural components used in Japan and those used in the US. These functions are illustrated in Figure 5 and their parameters are tabulated in Table 2.

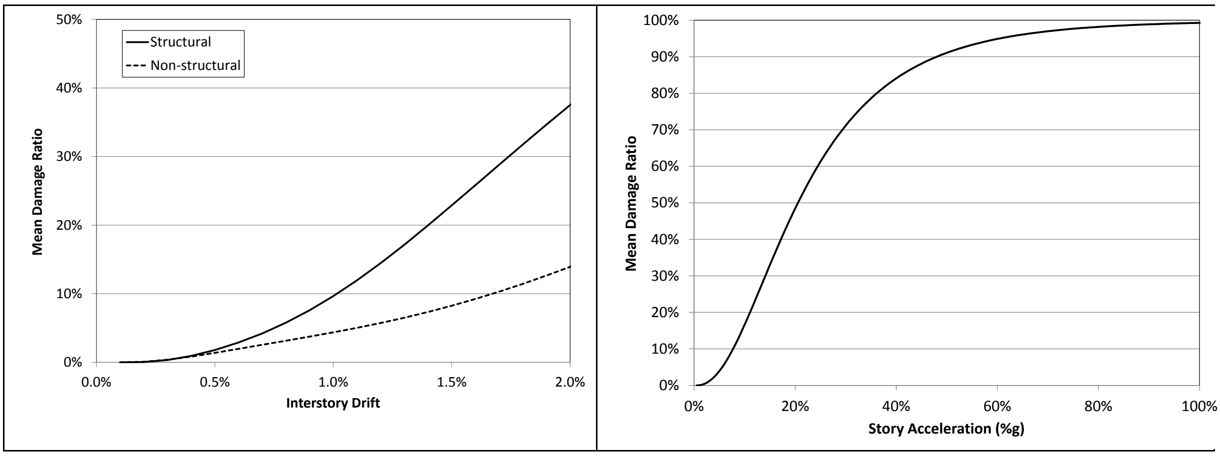

Figure 5. Story damage function for estimating structural and non-structural drift-sensitive damage (left) and non-structural acceleration sensitive damage (right) of the case-study building.

Table 2. Median and standard deviation of the HAZUS non-structural drift-sensitive and acceleration-sensitive fragilities that were applied to the case-study building.

	HAZUS - Drift Sensitive		**HAZUS - Acceleration Sensitive**	
	Median (interstory drift)	**Beta**	**Median (%g)**	**Beta**
Slight	0.40%	0.50	3.1%	0.60
Moderate	0.80%	0.50	6.1%	0.60
Extensive	2.50%	0.50	12%	0.60
Complete	5.00%	0.50	24%	0.60

The resulting estimated repair cost losses for the case-study building using the methodology described above are presented in Figure 6, which shows the overall probabilistic mean loss estimates for the building at specific return periods. All loss results have been normalized by the building total replacement value. The results indicate that a loss of about 1.7% of the replacement value can be expected to be incurred or exceeded, on average, once every 100 years; conversely, there is a 1% chance of incurring or exceeding the particular loss value every year. The corresponding mean loss for a 500-year return period, or 0.4% probability of exceedance in a year, is about 3.2%. The Average Annual Loss (AAL) associated with the analysis is 0.099%.

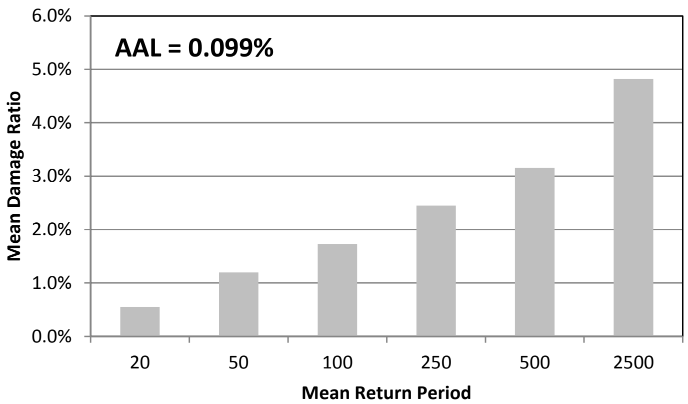

Figure 6. Expected loss results at selected mean return periods for the case-study building normalized by the replacement value of the building. AAL = Average Annual Loss.

5. CONCLUSIONS

A practical method to evaluate the seismic risk associated with individual tall buildings has been presented using a case-study building. The method combines different analysis techniques to provide a manageable, computationally efficient way of computing the seismic performance in terms of quantitative metrics, namely monetary loss. The method follows PEER's overall framework as adopted by ATC-58, with a few key adjustments that address some of the practical challenges faced by a building-specific, component-based loss estimation methods. The structural characteristics of a full 3D model is captured using a more simple "stick" model by matching their dynamic characteristics through nonlinear pushover analyses. This stick model, which is much more computationally efficient, can be used to compute the probabilistic engineering demand parameters required for loss estimation by using incremental dynamic analysis. Story-based loss estimation instead of component-based loss estimation is used to increase efficiency further by grouping components into seismic sensitivities and relying assumptions on the cost distribution of the building's component inventory.

Efficiency rarely comes without trade-offs, and this method is no exception. For instance, the stick model cannot completely capture all aspects of behaviour of a tall building, and the method cannot provide disaggregated loss estimates by component as it uses a story-based method. However, the focus is to develop probabilistic loss analysis as a computationally and resource friendly tool to promote its use and customize the analysis to be commensurate with the overall objectives of a study; thus, reasonable assumptions and an understanding of the same are necessary and appropriate. Overall, such methods and application offer many promising advantages to the engineer to effectively evaluate the earthquake risk of tall buildings.

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