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Effects of Building Features on Seismic Performance

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Effects of Building Features on Seismic Performance

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Effects of Building Features on Seismic Performance

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Often building decision-makers make decisions regarding the seismic risk to existing buildings, sometimes considering the effect of specific building features, such as geometric irregularities. Alternatives for quantifying the effect of a building feature on vulnerability are to use expert opinion, analytical methods, or empirical data from past earthquakes to quantify the effect. This work presents an analytical methodology to quantify the effect of a readily observable building feature on the seismic performance of a building class using 2nd-generation performance-based earthquake engineering (PBEE-2). Such a methodology does not appear to have been developed. The methodology begins by creating index buildings designed to span the observed range of values of the features with the greatest effect on vulnerability, to reflect variability within the building class. One set is designed with the feature of interest and another without. Each building is analyzed for collapse probability using PBEE-2 procedures. The effect of the feature is estimated as the ratio of the weighted-average vulnerability of the set with the feature to the weighted-average vulnerability of the set without the feature. I will call this ratio a seismic performance modification factor (SPM), a scalar value that varies with shaking severity. Illustrating the methodology, I quantify the effect of plan irregularity on the vulnerability of tall, steel-moment-frame buildings. It was impractical to fully illustrate the methodology, since doing so requires PBEE-2 models of several buildings, so I used Muto and Krishnan's (2011) models as proxies, and evaluated their PGV-based fragility functions. At Sa(1.0 sec, 5%) = 0.533g (2/3 MCE shaking for an area of Los Angeles County), I found a SPM = -0.488 (DSa = 0.31), compared to -0.50 from FEMA 154 (ATC 2002a). Considering the simplifications, these two values are surprisingly close, but perhaps a coincidence. However, comparing the result to what the FEMA 154 (ATC 2002a) authors produced by expert opinion shows that the methodology can result in a reasonable SPM for use in practice.

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1. Introduction

1.1 Background

Often insurance companies, investors, and building owners make decisions regarding the seismic risk to existing buildings. One example of a common decision is to determine if after a rapid visual screening, a building's safety is questionable enough that it should undergo detailed seismic evaluation. This decision (and others) often involves evaluating the seismic vulnerability of the building.

There are several methods for evaluating the seismic vulnerability of a building: empirical relationships, analytical methods, and expert judgment. Martel (1936) offers one of the earliest methods to estimate damage and loss based on building features. His method is considered an empirical method, which in general involves gathering observations about existing buildings (specifically building damage or values of monetary loss due to earthquakes), performing a regression analysis of the data, and estimating a ratio of repair cost to replacement cost (damage ratio) as a function of seismic excitation. If the expected value of loss at a specific level of excitation exceeds a tolerable threshold, then it may be worthwhile to perform a more-detailed, building-specific analysis.

Analytical methods require selection of ground-motion time histories and estimation of their frequencies of occurrence, producing the hazard analysis for the site. A detailed mathematical model of the structure is subjected to the seismic hazard through a structural analysis, producing a probabilistic estimate of the structural response of the building and its components. The response is put into fragility functions, which estimate the probability of the damage falling into specified damage states at a specified excitation, or vulnerability functions, which relate damage or loss to seismic excitation. Loss is calculated in terms of monetary value, risk to life-safety, or a damage ratio. One of the earliest analytical methods was developed by Czarnecki et al. (1973), followed by Kustu (1982), who built on Czarnecki's method. Analytical methods can be prohibitively costly, especially for the decision situation suggested above.

If sufficient empirical data are unavailable and analytical methods are not time or cost effective, a Delphi process can be used to gather expert judgment about the future seismic performance of the building, and again determine if the risk is high enough to warrant a more detailed seismic evaluation. A Delphi process relies on a panel of experts to provide judgments such as the probability of damage falling into certain damage levels. ATC-13 (1985) is an example of a seismic assessment that makes use of expert opinion in the development of vulnerability functions.

These three approaches—empirical, analytical, and expert opinion—have all been used in the past to help make decisions regarding seismic risk to buildings.

Sometimes decision-makers want to make decisions considering specific building features, sometimes called secondary features, such as building geometric irregularities, in addition to the broader class of the building (such as the category of construction material, lateral force resisting system, height category and design era). Commercial seismic risk (e.g., ST-RISK) software use such secondary features, which are believed to strongly influence seismic performance. It may be therefore valuable to be able to estimate the effect on the building's performance from features during the rapid visual screening.

A challenge to using empirical methods for this type of analysis (using secondary features) is that they do not offer many useful correlations between specific features and damage or loss, because the available data commonly do not resolve the presence or absence of these features. Analytical methods can offer reasonable estimates of damage and loss based on building features; however, they require significant time and effort – too much to be used in rapid visual screening. Expert judgment can also be used, but relying on opinion is seen as unscientific, non-repeatable, unverifiable, etc..

Some authors have combined methods into tools that can be used relatively quickly to make decisions regarding seismic risk. One example of these tools is FEMA 154 (ATC 2002a), which combines the use of analytical methods and expert judgment. FEMA 154 (ATC 2002a) employs a 1-page building survey form that estimates if further detailed seismic analysis is needed. A basic structural score is assigned to the building classification (based on material and lateral force resisting system) and seismic hazard. Then the surveyor selects modifiers for height category, vertical and plan irregularities, era of construction, and site soil classification, that are summed with the basic score to form a final score. A final score below a specified cut-off value indicates a need for further evaluation. Modifiers for some features, such as height, age, and the basic scores are determined analytically. However, modifiers for vertical and plan irregularities (e.g. soft story conditions where the ground floor is weaker or more flexible than the stories above) are determined based on the judgment of the authors. Other tools, such as ATC-50 (2002), also combine methods.

The present discussion focuses on FEMA 154 (ATC 2002a), but is more generally applicable to ATC-50 (2002) and commercial catastrophe risk models (e.g., ST-RISK) that currently use secondary features to adjust vulnerability functions for generic building classes, and potentially to other models such as HAZUS-MH (NIBS and FEMA 2003) that do not use secondary features but could in the future.

1.2 Objective

This research will develop an analytical methodology to estimate the effect of a readily observable building feature on the seismic risk of a class of U.S. buildings. To the author's knowledge, no such methodology currently exists that (1) uses an analytical method to reflect the effect of a readily observable building feature, (2) operates on an entire class of structures and (3) employs the state of the art in 2nd-generation performance based earthquake engineering (PBEE-2). An illustration of the methodology is provided that considers a building classification and a building feature. The illustration is not intended to address all features or all building types, though the method should be broadly applicable to many building types and features.

1.3 Organization

This chapter has introduced loss estimation methods and the motivation for this research. Chapter 2 provides a review of key literature on earthquake loss estimation, building inventories, and other related topics. Chapter 3 outlines the proposed analytical methodology to estimate the effects of a readily observable building feature on seismic damage and loss. Chapter 4 illustrates how to estimate the effect one observable building feature has on the seismic performance of a building class. In Chapter 5, conclusions are offered about the effectiveness of the presented methods, results and implications of the illustration, and possible future uses of the research. Chapter 6 contains a bibliography of references.

2. Review of Relevant Literature

This chapter reviews the literature of several topics related to the present research. Because the present research is concerned with quantifying the effects of building features on damage and loss, Section 2.1 provides a brief overview of damage- and loss-estimation methodologies, especially as they relate to classes of buildings and features of those buildings. Since the likelihood of observing various building features within a building category may be relevant, Section 2.2 reviews research related to building-inventory methods. To prepare for an illustrative case study, Section 2.3 reviews literature that sheds light on the features that appear to most strongly affect the performance of steel-moment-frame buildings. Section 2.4 investigates related methodologies. Section 2.5 reviews other relevant literature on the topics of moment matching, building models, and risk reduction.

2.1 Damage and Loss Estimation Literature

Damage and loss estimation is a fairly general topic, and where to begin can be daunting. Porter (2002) performs a "Survey of Surveys" that provides an overview of available literature. Porter reviews past methods of designing and conducting surveys to gather earthquake damage data as well as methods for disseminating the data for research purposes. This survey covers topics such as seismicity and geotechnical engineering, socio-economic impacts of earthquakes, building performance, and many other topics, with surveys spanning from 1936 to 2002. Of particular interest in this study are articles that deal with gathering post-earthquake data based on readily observable features of buildings (or of bridges in the case of Basoz and Kiremidjian (1998)), and using the data to define easily understandable damage states or other metrics of damage or loss. Articles that fall into this category are the HAZUS-MH Technical Manual (NIBS and FEMA 1999), ATC-50 (ATC 2002), ATC-21 ([FEMA 154] ATC 1988), ATC-38 (2000), Martel (1936), Rutherford and Chekene (1990), Lizundia et al. (1993), and Basoz and Kiremidjian (1998). These will form the initial framework of the literature review.

Martel (1936) offers a rigorous analysis of damage and loss of earthquake-induced building damage considering readily observable building features. He investigates the effect on damage as a function of building size, openness, and adjacency on damage, for unreinforced masonry bearing-wall buildings affected by the 1933 Long Beach, California, earthquake. The author offers aggregate observations, such as that the average damage for all buildings amounted to 20% of their replacement cost (i.e., 20% damage factor). He also quantifies the effects of building attributes. For example, loss tends to decrease as the number of stories increases, from 23% among buildings with one story down to 12% among buildings with four stories. The attribute that seems to matter most is openness. Buildings with only a single open side (a storefront) have on average 14% damage. For two-side open garages and stores, the damage is 35% and 26% respectively. Adjacency shows a general trend of lesser damage to buildings with two or more sides adjacent to other buildings. Category of building use does not appear to have much influence on percent damage.

Steinbrugge et al. (1969) perform another study of damage and loss estimation for a variety of structure types. The authors approach damage estimation for the purpose of studying alternative earthquake insurance methods. They estimate the total economic loss from the 1952 Bakersfield, CA earthquake, based on building permit applications. They look at different estimates of loss (market value, replacement cost, and cost to rebuild better) for each construction classification (concrete, steel, wood, and frame type). Relationships between earthquake frequency and intensity are also investigated.

Blume (1969) uses a different method to advance loss estimation. He proposes an engineering intensity (EI) scale for ground motions. At the time there were several scales available, such as the Modified Mercalli Intensity scale and Gutenberg-Richter. The proposed EI scale is based on 5% damped response spectra and a standard 10 x 9 matrix with columns representing period bands and rows representing bands of damped elastic spectral velocity response level, Sv. Acceleration or relative displacement can also be readily assigned with the use of 4-way logarithmic paper. The EI scale can be reported by period column as 9-digit, 3-digit, or 1-digit numbers, or by all three in a standard format. The author proposes that it can be used for buildings of all periods, types, ages, and localities. Damage is estimated as a function of building response using empirical data or judgment.

Czarnecki (1973) offers perhaps the first analytical, building-specific method for estimation of earthquake damage to tall buildings based on component energy absorption compared to maximum component absorption of which the component was capable. Czarnecki illustrates his method using building models subjected to ground motions from the 1971 San Fernando earthquake and compares his results with actual damages reported. To do this he creates stress-strain diagrams. The damage ratio is computed as the ratio of elastic energy absorbed to the elastic energy at yield. Once the energy absorption of a component (structural or non-structural) reaches the maximum energy absorption capacity, it is considered failed. Czarnecki derives a computer program to apply his model to buildings with known damage ratios and compares the results. It can predict general trends but may have great error for any specific case.

Writing for the U.S. Department of Commerce, Culver et al. (1975) appear to present the first public, multi-hazard loss estimation software, along with two other survey and life-safety evaluation methodologies for existing buildings subject to natural hazards. First, the Field Evaluation Method includes a detailed data-collection form to gather information based on observable or researchable features such as building and story height, adjacency, and age. The form supplements these with numerical ratings related to frame type, symmetry and quantity of resisting elements, and building condition, producing a building sub-rating. Finally, the evaluation form takes the sub-ratings and computes a basic structural rating which is then divided by an intensity-level factor (correlated to Modified Mercalli Intensity Scale). This so-called capacity ratio determines a building rating in qualitative terms such as good, fair, and poor, and indicates whether further analysis is needed. The authors denote the second method as the Approximate Analytical Evaluation Method. Structural analysis is used to evaluate stress in critical building elements, which is divided by the code stress limit. This is the "damage ratio," similar to Czarnecki's, which results in a building rating similar to the field method. Finally, the authors' Detailed Analysis Evaluation method requires a computer program that uses response spectra to compute floor accelerations, velocities, and inter-story drift. Structural damage is defined as the percent of members that fail in a given floor, meaning that the inter-story drift ratio causes the member to exceed its yield stress. Hence, ductility demand becomes the expression of damage. Each method can be individually employed. The authors suggest that using all three methods provides a more comprehensive view of the damage. These procedures do not indicate what level of damage is acceptable, since that is dependent on building use. The building ratings do not (explicitly) relate to life safety.

Whitman et al. (1977) correlate earthquake damage with strong ground motions, using several different methods, and data from the 1971 San Fernando Earthquake. They correlate ground-motion parameters, such as average peak ground acceleration, with a mean damage ratio (ratio of repair cost to replacement cost) for all structure types. The authors also estimate the elastic base shear ratio, which is related to structural type and represents the ratio of base shear a structure would experience if it remained elastic in a given earthquake to the base shear it experiences. Some general trends appear to correlate increased damage with increased intensity, but with a lot of scatter about the regression line. The authors also select 34 buildings in the area with known fundamental periods and an average ground motion for the specified zone, and plot the damage ratio against the average response spectrum of the zone for steel and concrete buildings and against the elastic base shear ratio. The same general trends of increased damage with increased intensity again appear but with a lot of residual scatter about the regression line. This methodology is illustrated using data from buildings built before 1933 and after 1947 that are more than 5 stories tall.

Blejwas and Bresler (1979) offer another analytical study of damageability. Damageability refers here to the level of damage in existing buildings when exposed to environmental excitation in a natural hazard. The authors perform a structural analysis using the selected hazard to determine component forces and displacements. Damageability has been split into different levels with an upper and lower limit, and each component response is assigned to a level of damageability. Once the damageability level of each component is known, it is multiplied by an importance factor and summed with all the others to determine the level of damageability for the entire building and normalized so that 0 indicates no damage. The last step is to relate damageability to a potential earthquake using an inelastic response spectrum. The authors analyze a model three-story structure to demonstrate the method.

Hassselman et al. (1980) present a computer program used to estimate earthquake and wind damage to highrise buildings on a story-by-story basis. They develop damageability models that relate a building median damage ratio to average interstory drift of the building. Damageability is the potential of a building to suffer damage from a natural hazard. Damage ratios are evaluated as the ratio of estimated repair cost to replacement cost of the building. The methodology combines empirical data with judgement using a Bayesian statistical updator.

In 1982, Kustu et al. offered a method to estimate building damage as a function of damage to individual building components, along the lines Czarnecki (1973) proposed but using empirical, probabilistic relationships between the structural response to which components are subjected and the damage they experience. These so-called fragility functions are in the form of parametric cumulative probability distribution functions, which give the probability of a component of a particular type reaching or exceeding given damage states, as a function of the value of the local structural response parameter. For example, damage states for a reinforced concrete beam could be: DS(0), no damage, DS(1), beam has cracked but reinforcement has not yielded, DS(2), beam is more severely cracked and some reinforcement has yielded, DS(3), beam has reached ultimate capacity and cannot carry a vertical load. The work advances the component approach for predicting damage by offering empirical fragility functions for various components.

ATC-13 (ATC 1985) offers a method to estimate societal-level damage and repair costs in California earthquakes. It operates by estimating damage and loss for individual buildings and summing by facility classification and then by all buildings. Facility classifications are defined by societal function (e.g. commercial retail) and structure type. Structure type is defined by construction material (wood, steel, etc.), lateral force resisting system (shearwall, moment frame, etc.), and height category (low, mid, or high rise). Building damage is estimated using damage probability matrices (DPMs), which relate damage factor (repair cost as a fraction of replacement cost new) to the Modified Mercalli Intensity scale, by structure type. The DPMs are developed through a modified Delphi process. (The modification has to do with experts weighting their expertise and confidence in particular estimates, and the authors using these self-assigned weights to combine judgments from multiple experts into weighted-average quantities in the DMP). That is, the authors solicit the expert judgment of a panel of experienced structural engineers. Experts are given a series of questionnaires and asked to estimate damage factors for each ground motion intensity level, collateral losses (ground failure, fire, etc.), and losses due to repair time, along with a confidence and experience level for their own estimates. Losses are estimated by multiplying the damage factor by the estimated replacement cost for the structure type. A final damage ratio is defined as the number of buildings damaged to the total number of buildings, again for similar structures subject to the same shaking intensity. Detailed building features do not appear in the methodology.

Scawthorn (1986) provides a basis for a rapid visual screening method for seismic hazard, which is described shortly (FEMA 154, ATC 1988, 2002a). Scawthorn presents an overview of empirical and analytical methods, and use of expert judgment, as well as a methodology for an advanced analytical method. He also discusses the rapid assessment of seismic vulnerability methods available at the time. Expert judgment is commonly used (ATC-13,-14) to estimate seismic performance for use in a rapid assessment. Experts can often quickly identify potential vulnerabilities such as: architectural and structural configuration, foundations, history of the building (age, changes, damage, maintenance), and detailing. The author suggests that different surveyors who examine a building to rapidly assess seismic hazard may come to different conclusions, but a set of expert judgments can be encoded into a checksheet so that the surveyor's judgment will become less of a variable. The checksheet is later developed by the authors of FEMA 154 (ATC 1988) (scores based on expert judgment and analytical methods). He offers some thoughts on requirements for an advanced analytical method.

Steinbrugge and Algermissen (1990) present a method to estimate earthquake-related monetary losses for single-family dwellings (in terms of dollars normalized by building value). Losses are quantified in terms of an insurance risk parameter called probable maximum loss (i.e. repair costs in excess of insurance deductible, in something approaching worst-case shaking). Building values are quantified both in terms of market value and insured value. The authors report earthquake damage to this building class using insurance data and field surveys. They categorize single-family dwellings that have experienced any of a few California earthquakes by age, number of stories, and type of floor (wood or concrete slab-on-grade). The authors find that newer dwellings experienced less damage and dwellings with concrete floors performed better than those with wood floors. The authors calculate losses in terms of insured losses in excess of deductible and market-value. The losses in terms of excess of deductible are lower than those in terms of market-value. Probable maximum losses are determined using judgment and showed a difference considering age, but not floor type, except in pre-1940 buildings. This method gives trends for this specific site and could be used for other sites or earthquakes.

Lizundia et al. (1993) analyze the results of a survey of damage to unreinforced-masonry (URM) buildings affected by the 1989 Loma Prieta earthquake, in part to examine the loss-reduction benefits of seismic retrofit. They relate earthquake building damage to ground motion and soil conditions using a multivariate regression analysis. They express the results using damage probability matrices. Soil conditions and story height appear to be the most statistically influential in estimation of damage ratios (assumed repair cost divided by replacement value). Most methods at the time considered damage ratios based on the assumption that any damage would be repaired, but this article proposes course-of-action probability matrices that influence the damage ratio. (In addition, the authors do not find strong evidence that common seismic retrofit measures for URM buildings generally reduced losses.)

FEMA 356 (ATC 2000) updates an earlier study (FEMA 273, Building Seismic Safety Council 1997) into mandatory (code) language. The authors propose a nationally applicable standard for seismic rehabilitation of buildings. The owners or other stakeholders choose structural and nonstructural performance objectives, which form a target building performance level. Seismic hazard is characterized by an acceleration response spectrum or acceleration time histories. The report provides general requirements for data collection, analysis procedures, methods, and strategies for design of seismic rehabilitation projects. It also lays out assumptions, limitations, and procedures for four different analysis methods that result in forces and deformations, and acceptable limits for each. Also included are procedures for the design of seismic isolation and energy dissipation systems. The methods operate at the single-building level, rather than building categories. The authors of FEMA 356 (ATC 2000) recognize FEMA 154 (ATC 1988, 2002a) as a tool for determining whether a building needs more-detailed seismic evaluation. To that extent it operates on categories of building and acknowledges the effects of readily-observable features. It acknowledges the risk-reduction benefits of building regularity, or the reduction in the amount of plan and vertical irregularities, especially torsion.

Porter et al. (2001) advance analytical methods to estimate damage and loss for individual buildings. The authors propose a methodology for a building specific, assembly-based, seismic vulnerability analysis. Similar to Czarnecki (1973) and Kustu et al. (1982), the authors expand the library of component fragility functions, and using Monte Carlo simulation, propagate performance uncertainties resulting from ground-motion time history, uncertain structural characteristics, uncertain component capacity, and uncertain repair costs. A simulation approach takes ground acceleration time history and applies it to a nonlinear dynamic structural analysis to determine structural response. The response is applied to assembly fragility functions to estimate damage to each structural and nonstructural building component. Repair costs and loss-of-use costs are determined probabilistically. The authors also propose a framework for gathering post-earthquake assembly damage statistics. This methodology evolved into or closely mirrored the second-generation performance-based earthquake engineering methodology developed by the Pacific Earthquake Engineering Research Center (PEER) and described by Porter (2003), among other PEER authors.

NIBS and FEMA (2003) describe procedures employed within the HAZUS-MH Advanced Engineering Building Module (AEBM) to estimate damage and loss for individual buildings. Like the PEER methodology, this component-based analytical approach employs hazard, structural, damage, and loss analysis, though at a more aggregated or approximate level. Structural analysis is performed by a single-degree-of-freedom non-linear pushover methodology called the capacity spectrum method. Damage is estimated using fragility functions that represent the building as three aggregated components capable of entering any of 4 qualitatively defined damage states, each with an associated deterministic damage factor or repair duration. By applying the theorem of total probability, one can estimate the expected value of loss. Loss is measured in terms of repair cost, life-safety impacts, and loss of functionality ("dollars, deaths, and downtime").

FEMA 154 (ATC 1988, 2002a) is a FEMA manual for the rapid visual screening process: a 1-page field-investigation form used to quickly assess whether a building is within an acceptable level of risk to life safety in an earthquake, or if it should be evaluated further. Risk to life safety is defined as the probability of collapse in an earthquake with a 2% probability of exceedance in 50 years. A basic structural score is assigned to the building classification (based on material and lateral force resisting system) and seismic hazard. Then the inspector selects modifiers for height category, vertical and plan irregularities, era of construction, and site soil classification, that are summed with the basic score to form a final score. If the final score is below a specified cut-off value, it indicates a need for further evaluation. Basic structural scores and some score modifiers are determined using the (largely analytical) HAZUS methodology. Vertical and plan irregularity modifiers are based on expert judgment. The score and modifiers represent collapse probability and multiplicative adjustments to that probability in the logarithmic domain, hence the use of sums to arrive at a final score.

ATC-50 (ATC 2002) encodes a rapid visual screening methodology in another building survey form to help decision-makers estimate the expected seismic performance of an existing building. The authors develop a standardized procedure and 4-page building-inspection form for seismic risk evaluations of single-family wood-frame dwellings. An inspector identifies structural deficiencies by assigning penalty points for features such as vertical and plan irregularities, mass irregularities, materials, soils, and some nonstructural elements. The penalty points are summed and subtracted from 100 to form a structural score. Regional deficiencies such as earthquake shaking potential, soil conditions, fault locations, and tsunami potential are also assigned penalty points that are summed to produce a seismic hazard score. The relevant features and penalty values were determined through expert judgment of structural engineers. A seismic performance grade (A, B, C, or D) is assigned based on the structural and seismic scores. The grade is indicative of an expected level of damage defined on the form.

Porter (2003) summarizes the developing analytical damage- and loss-estimation methodology of the Pacific Earthquake Engineering Research (PEER) Center, which he calls second-generation performance based earthquake engineering. The methodology includes four stages of analysis: hazard, structural, damage, and loss. Hazard analysis characterizes seismic hazard at the site, using ground motion time-histories whose intensity measure (IM) is appropriate to varying hazard levels. Structural analysis is usually a non-linear time history analysis subjected to ground motions of a given IM, resulting in estimates of structural response. Damage analysis uses the structural response as input to component fragility functions to determine probabilistic damage. Finally, a loss estimate applies construction contracting data and principles to physical damage to estimate repair costs, operability, repair duration, and potential for casualties. These are all determined probabilistically to estimate the frequency with which a performance measure will exceed various levels for a given design at a given location. The methodology is building-specific as opposed to applying to classes of buildings. It documents the PEER methodology at an early stage.

Krawinkler et al. (2005) establish a PEER Methodology Testbed of the Van Nuys Hotel Building in Southern California. These testbeds are real facilities to which PEER performance based earthquake engineering assessment methodologies are applied, then the methodology is assessed, in this case for buildings. The performance metrics of this study are financial losses due to damage and probability of collapse. The PEER methodology involves 4 processes: hazard analysis, structural analysis, damage analysis, and loss analysis. The testbed structure is an example of how to apply the procedures.

2.2 Building Inventory Literature

The methodology includes requires the use of a building inventory. To properly conduct a building survey, some knowledge is needed about the important components of a building inventory. The following literature provides insight into what components are considered important.

The authors of ATC-13 (ATC 1985) offer a building-inventory methodology. They propose using population- and housing- census data, along with economic-census data, as proxies for building area, supplemented by tax-assessor data. These sources provide data in various forms (e.g. number of employees or square footage). The information desired is the engineering facility classification, replacement value, location, value of contents, and number of occupants. ATC 13 authors suggest mathematical relationships between available proxy data and the desired information, such as square footage of building area per building occupant.

Jones et al. (1987) offer an inventory technique and illustrate it using the building stock of Wichita, Kansas, estimating the inventory in terms of type and use of structure, height, age, and area. Buildings are classified as residential or non-residential and then as a subcategory of those two classifications (e.g. single-family residence, mobile home, school, church, etc.). Data on number of buildings, square footage, and replacement costs for the entire country are gathered using tax-assessor data, while field surveys, and satellite photos are used to verify information. The data are standardized from total for population to a number per capita and were comparable to studies of Ithaca and New York City. The method appears to be fairly accurate. The authors find fairly consistent negatively exponential relationships between distance from the center of population and number of buildings of each type, area, and replacement cost.

Jones (1997) offers simple mathematical expressions to estimate the quantities and characteristics of a building stock using building height, floor area, and population density. Pre- or post-disaster planning can be improved through estimates of building stock in a specific area. Jones performs a regression analysis on 5 cities' building inventories and compares the results to his mathematical estimations; the estimated building stocks are close to the regression analysis. Using Jones' methods, replacement costs can be quickly estimated by building classification after a natural disaster.

French and Olshansky (2000) develop an inventory methodology and database for mid-American essential facilities (hospitals, police and fire stations, and schools). They consider buildings in 93 counties in the central United States that fall into zones that have experienced horizontal accelerations of 0.2g, 0.15g, 0.1g and have populations of 40,000 or more. The authors consider all hospitals, perform a field survey of 20% of essential facilities, and conduct telephone surveys for another 20% of facilities to obtain a representative sample. Facility characteristics of interest are structural type and material, age, height, and soils. The surveys also gather information about the facility shape and vertical or plan irregularities.

The broader HAZUS-MH methodology (NIBS and FEMA 2003) includes a building inventory used to estimate societal damage and loss. Buildings fall into one of 33 occupancy categories and one of 36 model building types, as well as a seismic design level and building quality specification. Occupancy categories are based on use (e.g. residential, commercial, industrial, etc.), while model building types are wood, steel, concrete, masonry, and mobile homes, and further characterized by lateral force resisting system (frame, shearwall, etc.) and height category (high-rise, mid-rise, or low-rise). Seismic design levels are categorized as high-code, moderate-code, low-code, and pre-code, generally referring to the code's seismic design force and detailing requirements. Construction quality is superior, ordinary, or inferior.

Jaiswal et al. (2010) present a global inventory at the country level for loss estimation and risk analysis as a part of the Prompt Assessment of Global Earthquakes for Response (PAGER) for the U.S. Geologic Survey. The inventory process involves three phases: data acquisition and quality rating, data aggregation, and data assignment for missing entries. The building distribution characterizes structures by material, lateral force resisting system, and occupancy type (residential, non-residential, urban, and rural). They synthesize data from many sources; each data source is given a quality rating (low, medium, or high) based on the survey type, survey objective, and expertise level of those taking the survey. Data from each country are entered in a single database and missing country distributions are estimated based on surrounding countries and their quality rating.

2.3 Welded Steel-Moment-Frame Building Important Features

For the illustration of the methodology, the building class chosen is steel-moment-resisting-frame buildings. It is necessary to understand which features have the greatest influence on the seismic performance on this class of buildings. The following literature contains insight into what some authors considered these features to be.

FEMA 310 (ATC 1998) is a handbook for seismic evaluation of buildings. The authors revised FEMA 178 into this prestandard, and the American Society of Civil Engineers has since turned it into a national consensus-based standard (ASCE 31-02 2002). Any building type can be evaluated and each building is given a Life Safety or Immediate Occupancy Performance

Level. An inspector uses the basic checklist to check for features that might affect the seismic performance of the building. For steel-moment-frame buildings, the observable features considered important to seismic risk are: load path, mezzanines, weak and soft story, horizontal geometry, vertical irregularity, mass irregularity, torsional irregularity, redundancy, connection type, plan irregularity, and steel or concrete damage. There are more checks, but the rest are not readily observable, they are design and construction checks. There are 3 tiers of evaluation: Tier 1- Screening Phase, Tier 2- Evaluation Phase (linear analysis), Tier 3- Detailed Evaluation Phase.

FEMA 350 (SAC 2000a) is the first in a series of reports by the SAC Joint Venture, which investigates performance problems with welded, steel moment-frame connections discovered after the 1994 Northridge earthquake. Many steel high-rise buildings experienced brittle failure in the beam-to-column joint. The authors offer updated design guidelines for steel moment-frame buildings. They also offer prequalified connection designs to replace the old designs. Prequalified connections include reduced beam sections, welded flange plates, welded unreinforced flange plates with bolted or unbolted webs, and others. The authors find that important building characteristics (i.e., features that seem to have strongly affected performance in the earthquake) are plan and vertical irregularities, connection type, height, and bay redundancy.

FEMA 351 (SAC 2000b) contains recommendations for methods to evaluate probable seismic performance of existing welded steel moment-frame (WSMF) buildings in the future, and retrofit them for improved performance. Connection type is determined to be the contributing characteristic to damage from the 1994 Northridge earthquake. Age is a factor only regarding pre- or post- Northridge eras. Height is considered in the analysis as it affects the interstory drift ratios. The authors offer prequalified connection data and connection qualification guidelines. Loss estimation combines empirical methods and expert judgment to arrive at loss as a function of damaged moment connections (as a fraction of total number of connections in building), connection restoration costs (fraction of building replacement cost), and nonstructural repair costs (fraction of building replacement costs). This document contains information that specifically updates FEMA-273 (written prior to the 1994 Northridge earthquake) with regard to the upgrade of WSMF buildings.

FEMA 352 (SAC 2000c) offers recommendations for performing post-earthquake building inspections (to detect damage in welded steel moment-frame buildings), evaluating the levels of damage and safety in damaged buildings, and repairing damaged buildings. If a damaging earthquake occurs and exceeds specified ground motion levels, then a preliminary evaluation occurs and the building will receive a green (no damage), yellow (damage), or red (unsafe) rating. Buildings suspected of having experienced potentially damaging ground motions should undergo more detailed inspection or evaluation. The preliminary post-earthquake building evaluation is used to identify building characteristics that might indicate vulnerability: age (may indicate specific vulnerabilities such as soft story or weld strength), height, vertical and horizontal irregularities, architectural elements, and connection type.

Bonowitz and Maison (2003) describe the basis for the rapid loss estimation method laid out in FEMA 351. The method relates overall building performance measures (e.g., repair costs) to seismic demand parameters (e.g., peak ground acceleration) based on data from the 1994 Northridge earthquake. The authors synthesize a database of building and damage information from surveys performed after the earthquake. Building inventory data includes the number of stories, gross floor area, and number of welded moment connections. They focus on damage patterns within the welded moment connections, but also investigate the effects of features such as height and load path redundancy. They create scatter plots to observe trends between demand and damage, but trends indicate weak correlations at best. They suggest the large variability is because the damage is site specific and the excitation used is the nearest recorded ground motion, which may not be accurate for the site and post-earthquake building inspections are not always complete. Also every building is uniquely configured (e.g., member sizes, layout, building codes) and construction may vary within the building category.

Kircher (2003) provides the basis for the procedures developed in FEMA 351 (SAC 2000b). Default values of damage and loss are provided for 3-story, 9-story, and 20-story SMRF buildings designed for Los Angeles, Seattle, and Boston seismic criteria with pre-Northridge, post-Northridge, or damaged pre-Northridge connection conditions. The authors estimate losses due to welded steel moment connections.

To keep track of the features that authors considered important to the seismic performance of steel-moment-resisting-frame buildings, Table 1 was created.

Table 1 - Features Considered Important to Seismic Performance of SMRF Buildings

	FEMA 154 (ATC 2002a)	FEMA 350 (ATC 2000a)	FEMA 351 (ATC 2000b)	FEMA 352 (ATC 2000c)	Bonowitz and Maison (2003)	Kircher (2003)	FEMA 310 (FEMA 1998)
Plan Irregularity	x	x		x			x
Soft Story/Vertical Irregularity	x	x		x			x
Number of Stories	x	x	x	x	x	x	x
Redundancy		x			x		x
Connection Type		x	x	x	x	x	x
Floor Area					x		x

*age is also important, but only to construction era

Important features are plan irregularity, vertical irregularity, number of stories, redundancy, connection type, and floor area (only mentioned by two authors).

2.4 Related Methodologies

Porter (2006) offers a methodology for combining empirical and analytical methods into an estimation of vulnerability for a subset of a building class, where the performance of the class as a whole can be estimated using empirical methods, but sufficient empirical data do not exist to distinguish the effect of a feature that defines the subset. Porter (2006) assumes that vulnerability functions derived empirically have greater credibility than those derived analytically, at least among some constituencies such as insurers. The methodology is applied to estimate the effect of certain seismic retrofit measures on the vulnerability of a building class, specifically pre-1979 single-family wood-frame dwellings. Empirical data are used to estimate the vulnerability of the building class, but not enough data are available to estimate vulnerability based on retrofit conditions. He develops 6 index buildings representative of pre-1979 single-family wood-frame dwellings and uses component-based methods to calculate probabilistic vulnerability in terms of expected annualized loss for dwellings that have undergone a retrofit and for the average of all buildings. An r factor is determined as the ratio of the theoretical vulnerability of the retrofitted subset to the average vulnerability of all buildings. The ratio is multiplied by the empirical vulnerability to reduce the effect of systematic theoretical error.

2.5 Other Relevant Literature

2.5.1 Moment Matching

One way of treating uncertainty in a probabilistic analysis is to use a process called moment matching. Moment matching is used in the methodology as a less time-consuming method for propagating uncertainty than Monte Carlo Simulation. Porter (In Progress) provides a discussion of the moment matching technique suggested for use in earthquake engineering by Ching et al. (2003), who adopt it from Julier and Uhlman's (2002) work in general mathematics. Moment matching is a generalization of quadrature, in which a function of a continuous random variable (a Gaussian variable) is approximated by replacing the Gaussian variable with a discrete set of sample values, each value associated with a probability-like weight, and applying the function to the weighted samples. Julier and Uhlman (2002) generalize from Gaussian random variables to others, and Ching et al. (2003) apply it to uncertainty propagation in PBEE-2 analysis of an individual building. Porter applies it to propagating uncertainties of portfolio seismic risk, in particular to uncertainties and correlation in ground motion, building value, and building vulnerability in suites of buildings (portfolios) subjected to a large number of possible future earthquakes. In comparing the method to the Monte Carlo simulation with a few simple functions, moment matching appears to achieve comparable accuracy in the mean and variance of a function with a few samples as Monte Carlo simulation does with 100 to 1000 samples. The first several moments of the function are evaluated using the samples and their weights, which are then used to generate a parametric distribution of the function—in this case loss—using the method of moments. Moment matching will be relevant here for propagating uncertainties in important characteristics of a building category.

2.5.2 Building Models

Part of the methodology calls for design of index buildings, and in the illustration, 14 buildings need to be designed. This is not within the scope of this research, so instead the results of analyses of two building models already created by Muto and Krishnan (2003) are used to represent the combined fragilities of the index buildings. Muto and Krishnan (2003) developed a plausible realization of the effects of the ShakeOut scenario earthquake on tall steel moment-frame buildings. They developed three welded steel-moment-frame building models: Building 1 is 18 stories and designed to the 1982 UBC, Building 2 is similar to Building 1, but redesigned to the 1997 UBC, and Building 3 is 19 stories and L-shaped, designed to the 1997 UBC. They used physics-based simulated ground motions developed for the ShakeOut by Graves et al. (2011) to simulate structural responses at 784 gridpoints spaced at about 4 km around Los Angeles. They determine the percentage of buildings in the 10-30-story range that fall into different damage states: 5% collapse, 10% red-tagged, 15% serious life-threatening damage, and 20% damage requiring building closure.

2.5.3 Risk Reduction

Kennedy and Short (1994) discuss a method to minimize the sensitivity of a particular seismic risk parameter to the value of logarithmic standard deviation (β) used in a lognormally distributed component fragility function. They suggest that if the value of the seismic excitation (the x-value in a fragility function) associated with 10% failure probability (x_{10}) is well established, the long-term failure probability is not particularly sensitive to the value of β. This point is particularly relevant when a fragility function appears to have too-low a value of β, for example when the test specimens that are used to establish the fragility function are not sufficiently representative of the diversity in the population they are supposed to represent. Their research is used to justify an increase in the logarithmic standard deviation in the illustration section for purposes of better representing the uncertainty found in a suite of buildings.

3. Methodology

3.1 Brief Overview

The proposed methodology is an analytical procedure to estimate the effect of a readily observable building feature on the seismic vulnerability or fragility of a class of buildings. Using second-generation performance based earthquake engineering (PBEE-2) procedures, this methodology estimates the probabilistic performance and vulnerability of a subset of the building class with the feature and then again of a subset without the feature. That is to say, a PBEE-2 analysis is performed on each building in 2 subsets of sample buildings, each of which belongs to the set or general class of interest. The sample buildings in one subset have the feature of interest, the others do not. The PBEE-2 analysis of each building is performed at a number of levels of excitation, and an estimate of some performance measure of interest (e.g., damage state, collapse probability, etc.) is calculated for each index building and each level of excitation. The performance of each subset is calculated as the weighted average of the performance measure of the sample buildings in the subset, the average calculated at each of several levels of excitation.

Note that the different buildings in each subset are selected or designed so that by their diversity they capture and propagate the most important uncertainties in the gross features of a building that belongs to the set or subset—number of stories, redundancy, and so on. In the context of PBEE-2, this means that important uncertainties in the asset definition (conditioned on membership in the subset and class) are captured and propagated. The PBEE-2 analysis of each building captures and propagates the most important uncertainties in ground motion, structural response, physical damage, and loss, conditioned on the asset definition. Thus, the methodology captures the most important uncertainties in the performance measure for each subset.

The ratio of performance measure for the subset with the feature to the subset without the feature will be referred to here as the seismic performance modification factor (SPM). This factor can be used in rapid visual screening processes, especially in FEMA-154 (ATC 2002a). The SPM has as a parameter the level of excitation at which the ratio is evaluated. For instance, the SPM might be defined in terms of collapse probability and evaluated at 2/3 x S_S x F_a, where S_S and F_a are the mapped MCE_R spectral response acceleration parameter at short periods and the corresponding site coefficient as defined in ASCE 7-10 (ASCE 2010). In FEMA 154 (2002a) the basic score represents the –log10(P[collapse|2/3 MCE, and Sa(1.0s, 5%)])

Porter (2006) proposes a similar methodology for determining the effect of a building feature for a related, but not the same problem. One difference is that this earlier work aims to produce the ratio of the average vulnerability of a subset of the building class with the feature to the average vulnerability of the entire building class (a mix of buildings both with and without the feature). He then relates that ratio to the empirical vulnerability, for use as an insurance rating factor. Another difference is that he does not use moment matching to do the uncertainty propagation.

3.2 Choose building class and performance metric

The procedure begins by choosing a building class of interest and a performance metric. Building class is often (though not universally) defined by material (e.g., steel, concrete, wood), lateral-force-resisting system (e.g. moment resisting frame), and height category (low-rise, mid-rise, or high-rise). An example of a building class would be mid-rise concrete moment-resisting-frame buildings. The building class must be consistent with the feature whose effect will be quantified (e.g., era of construction does not have an effect on a class of buildings for which the design codes have not changed over the years, as in the case of adobe buildings).

The performance metric could be quantified in terms of economic loss, human safety, or repair time ("dollars, death, and downtime" [Porter 2003]). Quantifying the effect of a feature requires a performance metric in the desired form, which could be any measure of damage (e.g., collapse or red tagging) or loss (e.g., ratio of repair cost to replacement cost), or other metric.

3.3 Identify S and P features

The observable feature whose effect will be quantified will be denoted as the primary feature, P. A readily observable feature is one that can be observed from the street, an aircraft, space, or a brief review of structural documents (e.g., building code in force, vertical and plan irregularities, number of stories, and mechanical characteristics of the major structural materials such as the grade of structural steel). A building class commonly has several relevant features that are all believed to be important to the seismic performance of that class. One can identify a limited number, *k*, of observable important features in addition to P. Let us denote them as "secondary" features, S, not to imply that they are less important than P but merely that they are other parameters that are not the primary one of interest. These S features can vary between individual buildings within a category. This thesis does not offer a general methodology for selecting S and P features, beyond simply reviewing relevant literature to identify features whose effects have not been quantified analytically, and which are believed to have a strong influence on the seismic performance of the class of buildings. FEMA 154 (ATC 2002a) and FEMA 310 (1998) contain lists of features the authors consider important for each building class. These lists along with opinions from other authors are discussed in Chapter 2.

3.4 Select Sample and Quantify Feature Probability Distributions

Select a building sample, that is, a number of specimens that are members of the class of interest. This thesis does not offer a general methodology for selecting the building sample, beyond the guidance that it should appear to be unbiased with respect to S and P features. The sample is intended to be representative of the entire class, so the analyst should have no reason to believe that the distribution of S and P features in the sample should differ substantially from that of the population. The sample should be large enough to produce probabilistic results—perhaps 20 or more buildings depending on population size. Each feature may be represented to a different extent in each building within the sample. For each building in the sample, tabulate the building's location or other identifier along with the value of each P and S feature. Quantify the probability distribution of each S and P feature within the sample. This thesis does not treat the potential for the values of features to be correlated, though that is a possibility. The observations needed to quantify the distributions can be obtained by means of remote sensing or in-person field obeservation.

Using the data collected from the building sample, one can assume and test the validity of a parametric probability distribution function that best represents the set of data for each feature. Procedures for doing so are described in common statistical texts and will not be detailed here. For present purposes, it is assumed that each probability distribution is determined by the central value (e.g., the mean or expected value) and a measure of dispersion (e.g., variance or standard deviation).

Once the parametric distributions have been fit, index buildings are designed to represent the important feature distributions of the building class. Some definitions are needed to begin.

Let j denote an index to a secondary feature S. For example, j = 1 might denote the index to the ratio of 1st story height to the typical upper-story height. Let S_j denote the uncertain value of the secondary feature j, and let s_j denote a particular value of S_j. Let $F_{Sj}(s_j)$ denote the cumulative distribution function of S_j, evaluated at s_j, and let n_p denote a particular value of the nonexceedance probability. Finally, let $F_{Sj}^{-1}(n_p)$ denote the value of S_j associated with probability n_p of not being exceeded. Thus, with $F_{Sj}(s_j)$ for each important feature, and a value of n_p for each feature, one defines all the readily observable important features of a single sample building from the population of interest, which is referred to here as an *index building*. These data do not define *all* the characteristics of the index building, just the ones deemed most important. Henceforth, let us assume that a building can be designed for purposes of PBEE-2 analysis with all the desired values of S_j such that $F_{Sj}(s_j) = n_p$ for each j, and for any set of n_p. This sample represents a sample for purposes of moment matching.

3.5 Quantify Probabilistic Performance of Index Buildings

Index buildings are designed to represent the important characteristics of the building class of interest. Within each subset, index buildings are designed to span the S space, using 3-point symmetric moment matching, as shown in Table 2. By "3-point symmetric moment matching" is meant that the moment-matching sample points and weights are additionally constrained such that for each feature j, there are 3 samples that span the distribution of S_j, with one at the median value, one at nonexceedance probability n_p in ($0.0 < g < 0.5$), one at nonexceedance probability $1–n_p$, and the weights of the latter two are equal. Also, it means here that n_p is constant regardless of the number of dimensions of the S space.

In Table 2, i is an index to the index buildings $i \in \{0, 1, \ldots 2k\}$, there are k secondary features, j is an index to them $j \in \{1, 2, \ldots k\}$, n_p is a particular nonexceedance probability of S_j, and w_i is the weight assigned to index building i. The value of n_p is constant. Porter (In Progress) suggests n_p = 0.03 and w_1 = 1/8 seems to work well for lognormally distributed random variables S_j that have logarithmic standard deviations between 0.2 and 0.6, and various simple forms of the function of the vector of S. The weight assigned to the building designed to represent the median value of all features is calculated by $1\text{-}2{*}k{*}w_{i\neq 0}$.

Table 2. Values of index-building design features

Index building *i*	$F_{S1}(s_1)$	$F_{S2}(s_2)$	…	$F_{Sk}(s_k)$	w_i
0	0.5	0.5		0.5	$1\text{-}2kw_{i\neq 0}$
1	n_p	0.5		0.5	$w_{i\neq 0}$
2	1- n_p	0.5		0.5	$w_{i\neq 0}$
3	0.5	g		0.5	$w_{i\neq 0}$
4	0.5	1- n_p		0.5	$w_{i\neq 0}$
…					
$2k$-1	0.5	0.5		n_p	$w_{i\neq 0}$
$2k$	0.5	0.5		1- n_p	$w_{i\neq 0}$

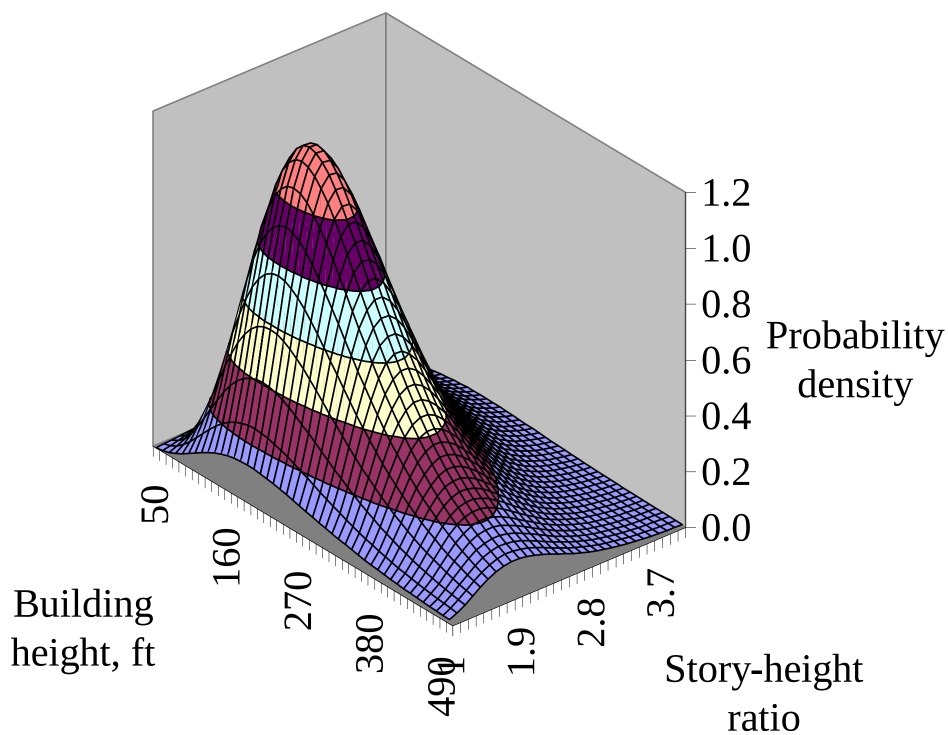
Figure 1 illustrates the method for designing the index buildings. The axis of each S feature is placed perpendicular to the other S features at the median of all the features. Samples are marked on the axes. Recall, there are k secondary features. Let q denote an index to a value of S_j, with 3 possible values: q=0 denotes the median value of S_j, q=1 denotes a low value, and q=2 denotes a high value of S_j. We select values of $S_{j,q}$ as follows:

$$\boldsymbol{S_{j,q} = F_{Sj}^{-1}(0.50), q = 0} \tag{1}$$

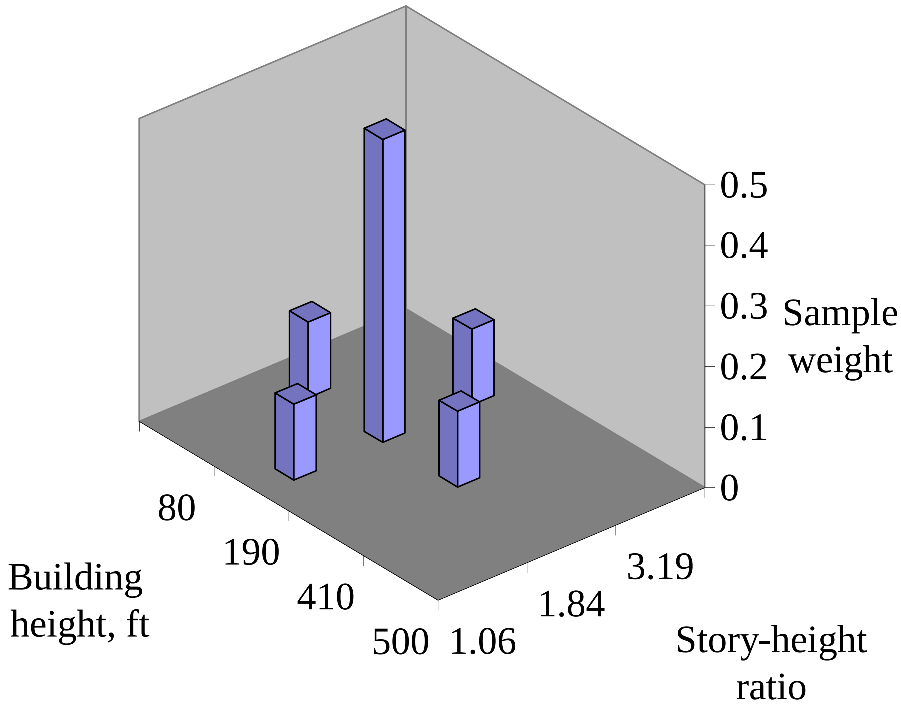
$$\boldsymbol{S_{j,q} = F_{Sj}^{-1}(g), q = 1} \tag{2}$$

$$\boldsymbol{S_{j,q} = F_{Sj}^{-1}(1-g), q = 2} \tag{3}$$

An index building is designed to represent each point on the axes, so as indicated in Table 2 there are 2k+1 buildings representing the building class. Index buildings are designed for several cases: one index building with all values of secondary features set at their median value, one with all parameters at the median value except that one feature has the lower-bound value, and one with all parameters at the median but with the same feature at the upper-bound value. Figure 1 (a) shows a joint probability density surface for an illustration of moment matching with just two features: building height and story-height ratio. Figure 1 (b) shows how the index buildings are weighted and designed to represent the probability distribution.



(a)



(b)

Figure 1- Moment Matching Illustration with Two Features

One set of index buildings is designed with all S specimens having the primary feature of interest, P. Let us denote this subset as i=0,1,2…$2k$. It is repeated with all specimens not having P. Let us denote this subset as $i = 2k + 1, 2k + 2 \ldots 4k + 1$. Once defined, each index building is analyzed using the now-becoming-standard PBEE-2 procedures of ATC-58 (ATC 2011) to quantify its probabilistic performance. Except for brief discussion of the means for propagating uncertainty, it is not within the scope of this research to report in detail on how PBEE-2 works. For detailed discussion, see Porter (2001) or Krawinkler (2005).

The main sources of uncertainty in a PEER and ATC-58 (ATC 2011) PBEE-2 analysis are listed in Chapter 6 of Krawinkler et al. (2005), along with descriptions of several methods used to propagate the uncertainties. The apparently standard approach to propagating uncertainty within the PBEE-2 analysis of a single index building is Monte Carlo Simulation.

The reader may wonder, why not use moment matching within the PBEE-2 analysis of an index building, or Monte Carlo simulation in the design of the index buildings? There are several reasons: First, within the structural-analysis stage of PBEE-2, one selects a number of ground-motion time histories consistent with each of several particular values h of the uncertain intensity measure level H. Conditioned on $H = h$, there is currently no concept of a particular ground motion having nonexceedance probability n_p. So at least for the structural-analysis stage, the only practical option at present seems to be Monte Carlo simulation. The second reason is practical. ATC-58 (2011) has designed an algorithm and software to carry out the damage and loss analyses of PBEE-2 using Monte Carlo Simulation. Third, it would probably be difficult and computationally expensive to design index buildings that are both code-complying and realistic in an automated fashion by Monte Carlo Simulation.

Returning to the analysis, the next step is to decide on the hazard measure, hazard level, and location of interest. Some reasonable options for the hazard measure include damped elastic spectral acceleration response at a given period and damping ratio; macroseismic intensity from a scale such as Modified Mercalli Intensity; or peak ground motion measure such as peak ground velocity. Spectral acceleration is commonly used because it strongly relates to structural response for a building with an approximately known elastic period. Macroseismic intensity is often used when specific ground motions are unavailable, and is easier to communicate to the public. Peak ground velocity is another measure sometimes used because it appears to be commonly understood by earth scientists to strongly relate to building damage, probably better understood than spectral acceleration. For this illustration I will use peak ground velocity (PGV) because it relates to the work that Muto and Krishnan (2003) have done with their building models.

The second decision is to select the location of the analysis. This might be a specific building site with a specific value of hazard or an entire region. This methodology will use the 784 sites developed by Muto and Krishnan (2003).

Finally, one decides on a hazard level. In the recent past (prior to ASCE 7-10), structural engineers have most commonly used the design hazard level from ASCE 7, i.e., 2/3 of the shaking in a maximum considered earthquake (MCE), defined as the earthquake shaking with 2% probability of exceedance in 50 years. In more recent years there has been a shift towards using uniform seismic risk rather than uniform seismic hazard (ASCE 7-10). This means that instead of using a probability of exceedance for a ground motion, uniform risk would be a probability of a specified level of risk or damage. If the location of the analysis is a region, one must use a general hazard value for the entire region, such as an average or weighted average. The design hazard level from ASCE 7 will be used here.

Let $y_i(x)$ denote the value of the performance metric for sample i subjected to excitation x. Estimate the average analytical performance of the population with the feature of interest, $\bar{y}(x)$, as in Equation $(\mathbf{4})$ and the average vulnerability of the subset without the feature of interest $\hat{y}(x)$, as in Equation (5).

$$\bar{\boldsymbol{y}}(\boldsymbol{x}) = \frac{\sum_{\boldsymbol{i=0}}^{\boldsymbol{2k}} \boldsymbol{w_i y_i(x)}}{\sum_{\boldsymbol{i=0}}^{\boldsymbol{2k}} \boldsymbol{w_i}} \quad (\mathbf{4})$$

$$\hat{y}(\boldsymbol{x}) = \frac{\sum_{\boldsymbol{i=(2k+1)}}^{\boldsymbol{(4k+1)}} \boldsymbol{w_i y_i(x)}}{\sum_{\boldsymbol{i=(2k+1)}}^{\boldsymbol{(4k+1)}} \boldsymbol{w_i}} \quad (\mathbf{5})$$

$$\boldsymbol{f}(\boldsymbol{x}) = \frac{\bar{y}(\boldsymbol{x})}{\hat{y}(\boldsymbol{x})} \quad (6)$$

In Equation (4) the subscripts include all of the specimens with P, i=0,1,2….2k. Equation (5) only involves the subset i = 2k+1, 2k+2...4k+1 without the feature P. Equation (6) estimates the average effect of the building feature as the ratio of the average vulnerability of the subset with the feature to the average of the subset without the feature. Let us denote this ratio as the seismic performance modification factor (SPM), or $f(x)$, as in Equation (6).

4. Illustration of Methodology

The methodology is here illustrated with one primary feature, three secondary features, and one building class. I apply the method using remote-sensing data (Google Earth) to identify the values of all the secondary features. At this point a shortcut is required. With 3-point symmetric moment matching, the methodology would require the design of 7 index buildings with the primary feature and 7 without. Because the design of 14 index buildings is beyond the resources and time available for this thesis, I will use previously developed fragility functions for just two building models rather than a suite of models that represent all the feature distributions. One of the models has the primary feature, the other does not. It will be imagined that the fragility function for each model represents the weighted average of 7. An ad-hoc adjustment is made to reflect the greater uncertainty in the fragility function for 7 buildings than would be observed for one building. The shortcut seems reasonable since this illustration is meant to aid in understanding of the methodology, not to provide results that could be used in practice for this building type and feature. Chapter 5 will include a description of what would need to be done to fully implement this method.

The building class chosen for the illustration is tall steel moment-frame (SMF) buildings in California built after 1997. Herein, 8 stories (above ground) or taller is considered tall. This category is chosen for ease of data-collection and because of its high-seismicity location. After the 1994 Northridge earthquake, a certain class of welded steel-moment-frame buildings was investigated and many were found to have unexpectedly brittle beam-column moment connections (FEMA 350, 351, 352 [2000]). As a result, building codes have been updated to reduce damage to connections in the future. The subset chosen includes all tall steel buildings for which construction began after 1997 or was completed after 1998 to ensure that the buildings were built to generally the same design standards.

The intention of this research is to provide a methodology to quantify the effects of an observable building feature on a class of buildings. A desired goal would be to relate the effect of one feature to the effects of other features in a meaningful way. Since the authors of FEMA 154 (2002a) have already analytically derived a modification factor for some features, it is concievable that this research could be used by FEMA to analytically derive the effects of the remaining features.

FEMA 154 (ATC 1988, 2002a) contains a commonly used building survey that evaluates the probability of collapse based on observable building features. The methodology produces a scaler score, S, which is an estimate of the negative base-10 log probability of collapse conditioned in 2/3 Maximum Considered Earthquake (MCE) shaking. So a score of 2 indicates a 0.01 collapse probability in design-level shaking.

As discussed in Chapter 2, plan irregularity is considered a readily observable feature with a significant impact on the seismic performance of a steel-moment-frame (SMF) building. I could find no English-language literature suggesting that anyone has analytically derived the effects on collapse probability from plan irregularities in SMF buildings. For this illustration, plan irregularity is the primary feature, P. In the survey plan irregularity will be observed for the long direction, and then again for the shorter direction. Plan irregularity is defined here as 1 minus D_2/D_1 (Equation (7)).

$$P(plan\ irregularity) = 1 - \frac{D2}{D1}, \quad D1 > D2 \tag{7}$$

where D_1 is the longest plan length in a direction and D_2 is the length of any shorter plan element in that same direction. The equation denotes setback as a fraction of width on that side of the building. For some examples of how to determine D_1 and D_2 see Figure 2 (a)-(c) and Figure 3. According to ASCE 7-10, Table 12.3-1, a building is considered to have a plan irregularity if $P_L > 0.15$ or $P_S > 0.15$ (where L and S denote the long and short plan direction respectively), and not to have one otherwise. (ASCE 7-10 also contains guidance for cutoff values of other irregularities). The value of Equation (7) in both long and short directions should be recorded in the survey of the building sample.

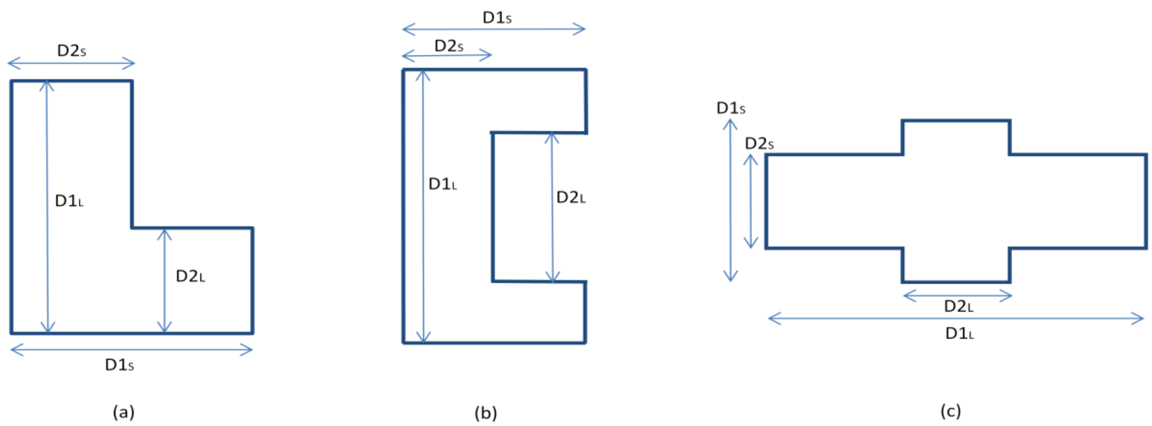


Figure 2-Plan Irregularity Definitions (D_1 &D_2; short and long side)

Figure 3 - Plan Irregularity Data for Building 3648

It should be noted that this research does not include guidance on how to decide a cutoff value for continuous random variables other than through reviewing relevant literature. It should also be noted that this method assumes each feature to be independent of any other feature. In addition, it assumes that plan irregularity in the long direction is independent of plan irregularity the short direction. Other features that are believed to affect the seismic performance of SMF buildings are number of stories (above ground), soft story conditions (sometimes general vertical irregularity), redundancy, connection type, and material strength (see Table 1). These will be denoted as the secondary features.

A conversation with the American Institute of Steel Construction Help Desk (July 22, 2011) determined that material strength among steel buildings built on the U.S. West Coast after 1997 is almost uniformly 50 ksi. Since it is not particularly variable, it will not be a secondary feature in the creation of the index buildings. Connection types also are uniform. After 1997, most steel moment frame building connections include a dog-bone beam reduction.

The Emporis database (www.emporis.com) was used to determine which buildings in California fall into the category of tall steel moment-frame buildings. Emporis gathers information from "data researchers, building-related companies, the public audience, vendors, and the editorial community" (www.emporis.com). Many recently built tall buildings have been constructed with a dual system consisting of a concrete core with a steel gravity- or moment-frame. Assuming that dual-system buildings are correctly listed in the Emporis database as "steel and concrete," it follows that buildings listed as "steel" are in fact steel moment frames. This search found 24 structures in the category of interest (Table 3). All 24 were examined using Google Earth.

Table 3 - Building Information for Survey

Emporis Building Number	Building Name	Exact Address	Postcode	Latitude	Longitude	Constr -uction Start	Constr -uction End
123288	Cal/EPA Building	1001 I Street	95812	38.58199	-121.492079	1998	2000
100482	JP MorganChase Building	560-584 Mission Street	94105	37.78894	-122.399427	2000	2002
100964	555 City Center	555 12th Street	94607	37.80371	-122.275327	2000	2002
206803	Dr. Martin Luther King Jr. Library	150 East San Fernando Street	95112	37.33547	-121.885076	2000	2003
100400	Constellation Place	10250 Constellation Boulevard	90067	34.05737	-118.417259	2001	2003
175535	Tower at Convention Center Court	855 M Street	93721	36.73457	-119.78592	2002	2003
153717	Caltrans District 7 Headquarters Building	100 South Main Street	90012	34.0512	-118.243382	2002	2004
102170	San Jose City Hall	200 East Santa Clara St.	95113	37.33802	-121.885344	2002	2005
132385	Advanced Equities Plaza	601-655 West Broadway	92101	32.71536	-117.168669	2003	2005
209871	Qualcomm Building WT	5751 Pacific Center Boulevard	92121	32.90374	-117.196355	2004	2006
205885	2000 Avenue of the Stars	2000 Avenue of the Stars	90067	34.05817	-118.414689	2004	2007
126142	EM Harris State Office Building	1515 Clay Street	94612	37.80647	-122.273514		1998
118708	101 Second Street	101-131 Second Street	94104	37.78815	-122.399041		1999
123321	U.S. Courthouse & Federal Building	501 I Street	95814	38.58358	-121.49864		1999
126736	Glendale Plaza	655 North Central Ave.	91203	34.15571	-118.258579		1999
137787	611 Gateway Boulevard		94080	37.65872	-122.400194		2000
134104	6080 Center Drive	6080 Center Drive	90045				2001
102178	Gap Building	2-98 Folsom Street	94104	37.79072	-122.391177		2001
100483	55 Second Street	39-67 Second Street	94105	37.7889	-122.400342		2002
100456	Manchester Grand Hyatt Seaport	1 Market Place	92101	32.71065	-117.168803		2003
138037	Sobrato Office Tower	488 South Almaden Boulevard	95113	37.32719	-121.88903		2003
188608	Adobe Systems Almaden Tower	151 South Almaden Boulevard	95110	37.33139	-121.893665		2003
149653	Fresno United States Courthouse	1014 O Street	93721	36.73733	-119.783539		2005
239313	M.H. de Young Memorial Museum	50 Hagiwara Tea Garden Drive	94122	37.77155	-122.468693		2005

Bonowitz and Maison (2003), along with the authors of FEMA 350 (SAC 2000a) and FEMA 310 (FEMA 1998), consider redundancy a feature that greatly influences seismic performance of a SMF building. In structural engineering, redundancy indicates multiple load paths. In the event that one load path is compromised, the loads could use a different path, keeping the building from potential collapse. One direction (front or side) of a building is typically longer than the other. Under seismic loading, and especially in the presence of torsion, the shorter side would be expected to experience more damage because it is farther away from the center of mass. Since it is shorter, it also likely has fewer bays, resulting in less redundancy and more potential for damage. Herein, redundancy is quantified as the number of bays along the shorter side of a building.

Another feature with significant influence on the seismic performance of steel buildings is the presence of soft story conditions (FEMA 154 [ATC 2000a], FEMA 350/352 [SAC 2000a/2000c]). FEMA 154 (ATC 2000a) defines a soft story as a story with a dramatic reduction in stiffness as compared to other stories. As defined in ASCE/SEI 31-03 (ASCE 2003), a soft story is one in which the lateral stiffness is less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above. It is difficult to observe an actual difference in stiffness, but the authors suggest that to be conservative, if the first story is much taller and has a lot of windows it can be considered to have a soft story. For this survey, rather than observing a yes or no value for soft story, the ratio between first story height and average upper story height is observed. The first story height is estimated using Google Earth street view (www.maps.google.com), along with the average upper story height (see Figure 4). This decision was made partly for the design process. The ratio of first story to average upper story height provides a continuous distribution and can be used in the design of the index buildings.

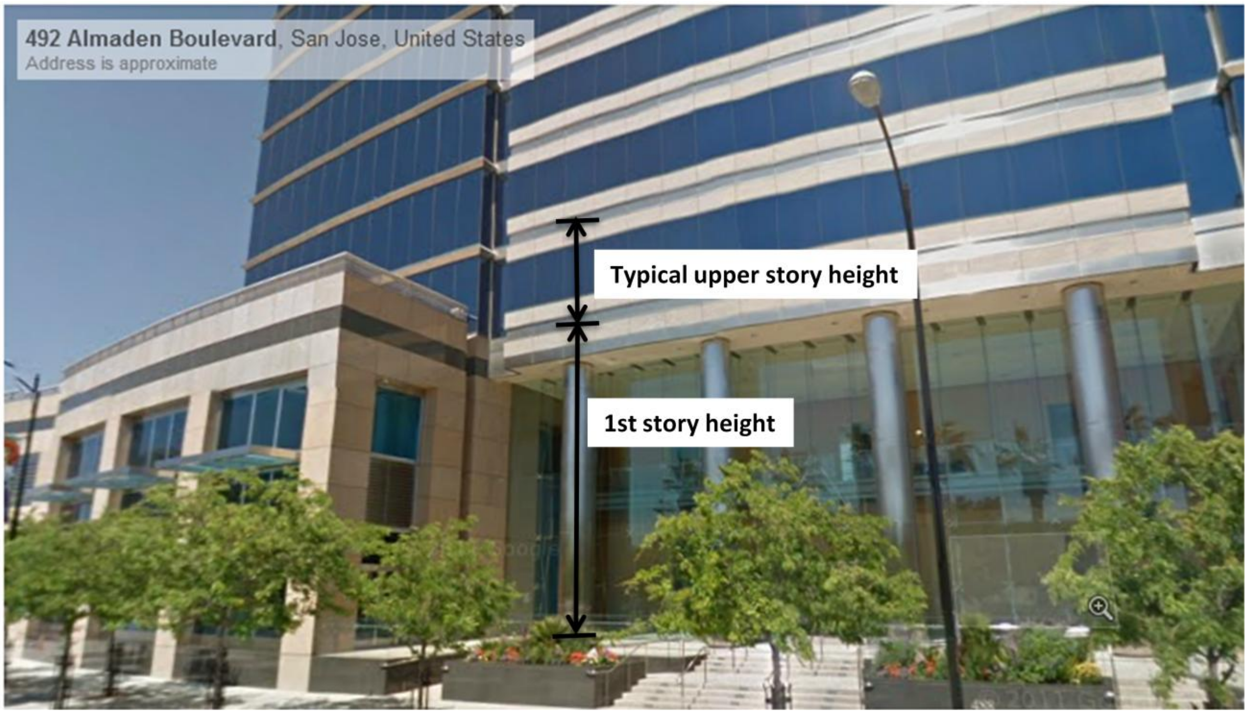


Figure 4 - Story Height Ratio Illustration

Thus, the secondary features employed here are:

1. Number of stories above ground
2. Soft-story conditions, parameterized here using the ratio of 1st-story height to 2nd-story height
3. Redundancy, parameterized here using the number of bays in the short plan direction

For the building survey, the important features were observed and quantified for each building using Google Earth (www.maps.google.com), with the exception of number of stories, which was taken from the Emporis database. Table 4 contains the survey results.

Table 4 - Survey Results

Emporis Building Number	Floors Overground	# Bays-Short Side	Story Ratio	Plan Irregularity-Long Side	Plan Irregularity-Short Side
123288	25	5	5.00	0.556	0.371
100482	31	8	3.88	0.000	0.000
100964	20	8	2.50	0.000	0.000
206803	8	9	0.89	0.318	0.545
100400	35	4	8.75	0.125	0.178
175535	11	7	1.57	0.000	0.000
153717	13	5	2.60	0.000	0.000
102170	18	7	2.57	0.182	0.250
132385	23	8	2.88	0.250	0.500
209871	12	9	1.33	0.167	0.400
205885	13	4	3.25	0.000	0.000
126142	22	6	3.67	0.261	0.360
118708	26	7	3.71	0.182	0.556
123321	18	5	3.60	0.158	0.381
126736	25	6	4.17	0.000	0.000
137787	12	3	4.00	0.223	0.403
134104	12	4	3.00	0.074	0.149
102178	15	8	1.88	0.286	0.571
100483	25	6	4.17	0.000	0.000
100456	34	5	6.80	0.000	0.290
138037	19	11	1.73	0.143	0.190
188608	18	10	1.80	0.381	0.300
149653	9	8	1.13	0.563	0.643
239313	9	5	1.80	0.500	0.455

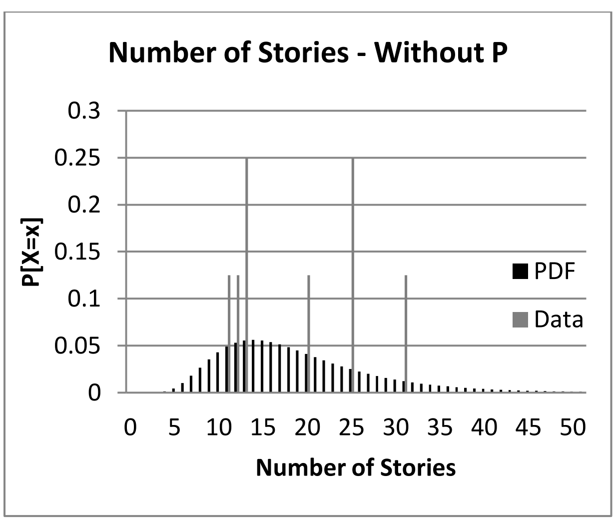
After the survey is complete, the data are sorted into two bins. One bin contains all data for buildings with plan irregularity (recall: plan irregularity is present if P(either side) > 0.15), and the other contains all the data for the buildings without plan irregularity. Parametric probability distributions are fit to the data for each feature in each bin. The distributions of all features are assumed to be lognormal. (A goodness of fit test, such as Kolmogorov-Smirnov, could be used to check whether lognormal sufficiently matches the data; this was not done.) Logarithmic normal probability density functions can be written as:

$$f_S(s) = \frac{1}{\beta s\sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{ln\, s - \lambda}{\beta}\right)\right] \tag{8}$$

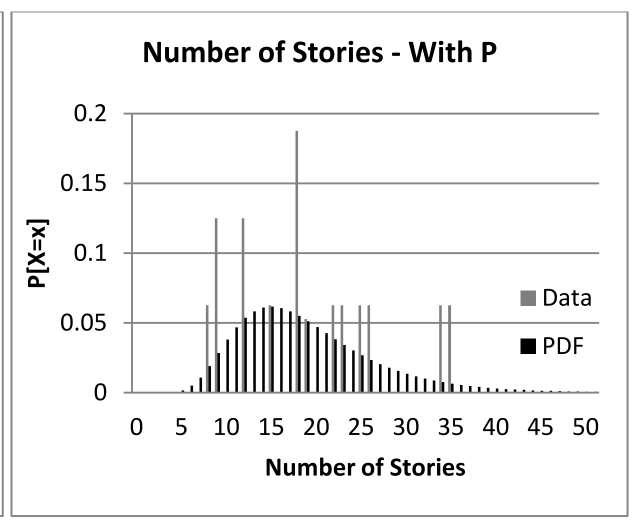
$$where: \beta = std.dev(\ln(s))\ and\ \lambda = mean(\ln(s))$$

In these distributions, s is a value of the feature (e.g. for number of stories: s = 18 means 18 stories). The graphical results for each feature are presented in Figure 5 through Figure 8.

To illustrate the difference between the data and the assumption that of a lognormal distribution, Figures 5 through 8 show the probability distribution function (PDF) or cumulative distribution function (CDF) for each feature in each bin with the data normalized and plotted along with the related parametric distribution. The data on a PDF plot shows the number of buildings observed at that value divided by the total number of buildings, while the data on the CDF plot tells the number of buildings with that particular value or less. Some of these features appear to be a better fit than others (e.g., the number of stories data does not show a very strong correlation to the PDF, while story height ratio seems to correlate fairly well), so using a lognormal distribution is not perfect, but a reasonable assumption.

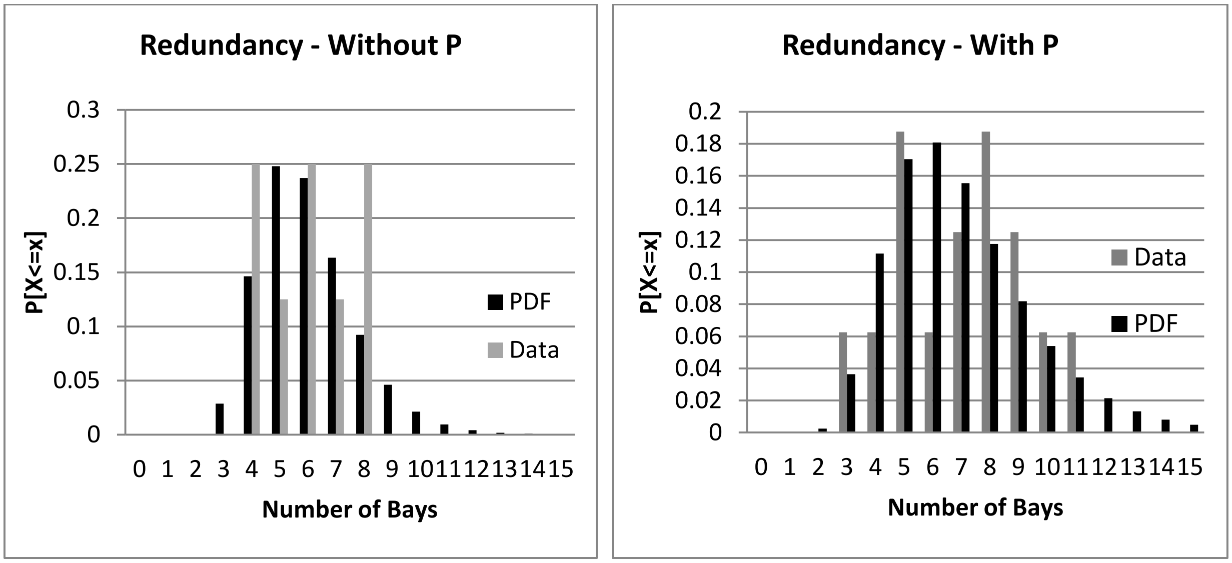


(a)



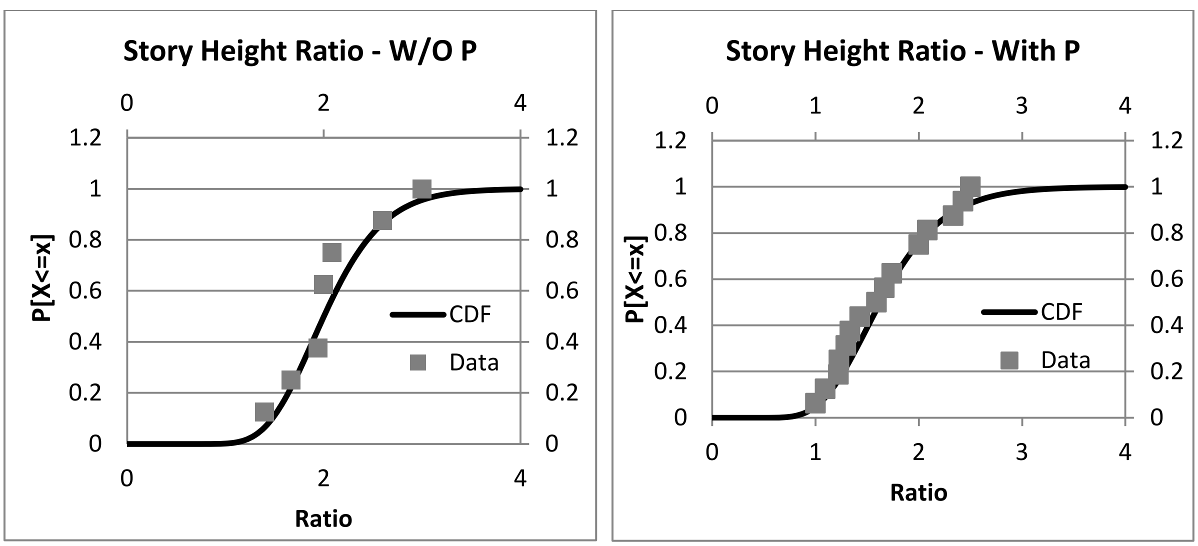
(b)

Figure 5 – Distributions of Number of Stories Without (a) and With (b) P



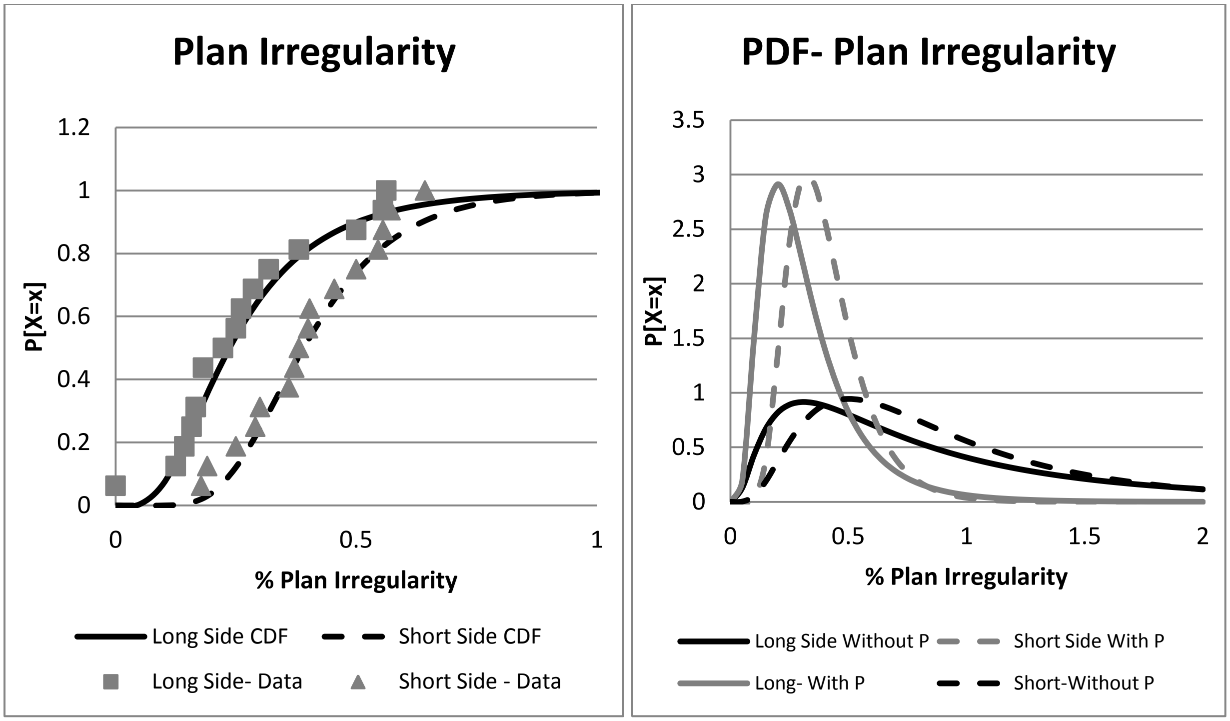
(a) (b)

Figure 6 – Distributions of Story Height Ratio Without (a) and With (b) P



(a) (b)

Figure 7 - Probability Density of Redundancy Without (a) and With (b) P



(a) (b)

Figure 8 - Distributions of Plan Irregularity With P (a) and PDF for All Cases (b)

For each feature, the cumulative distribution function of the parametric probability distribution that is fit to the data is used to estimate the nonexceedance probability of a feature. Next, it is necessary to find values of the features at a low probability of nonexceedance, the median, and a high probability of nonexceedance. As suggested by Porter (In Progress), q=1 denotes an index to a specimen with nonexceedance probability of some feature at 3% (g=0.03). The index q=0 refers to the building with the median value of the feature (g=0.5), and at q=2 the probability of nonexceedance is 97% (1-g=0.97). Per Equation (9), the value of each feature with assumed lognormal distribution is:

$$
\begin{aligned}
S_{j,q} &= \theta \cdot \exp(\Phi^{-1}(0.50) \cdot \beta) \qquad q = 0 \\
&= \theta \cdot \exp(\Phi^{-1}(0.03) \cdot \beta) \qquad q = 1 \\
&= \theta \cdot \exp(\Phi^{-1}(0.97) \cdot \beta) \qquad q = 2
\end{aligned} \tag{9}
$$

where θ is the median and β is the logarithmic standard deviation. The values are listed in Table 5.

Table 5-Index Values of Features at Low, Median, and High Probabilities of Nonexceedance

Index Values of Set without P	**$F_X^{-1}(0.03)$**	**$F_X^{-1}(0.5)$**	**$F_X^{-1}(0.97)$**
Number of Stories	8	17	36
Number of bays-short direction	4	6	11
Story Height Ratio	1.28	2.00	3.12
Plan Irregularity-Long direction	0	0	0
Plan Irregularity-Short direction	0	0	0
Index Values of Set with P	**$F_X^{-1}(0.03)$**	**$F_X^{-1}(0.5)$**	**$F_X^{-1}(0.97)$**
Number of Stories	8	18	43
Number of bays-short direction	4	7	14
Story Height Ratio	0.938	1.63	2.83
Plan Irregularity-Long direction	0.0785	0.390	0.000
Plan Irregularity-Short direction	0.190	0.390	0.801

**values of the features that have probability, p, of not being exceeded at low index (p=0.03), median (p=0.5), and high index (p=0.97).*

Each of these indices is then used in the moment matching process. The median on the distribution axis of each feature crosses the medians of all feature distributions (see Figure 1). Starting with the subset that represents buildings with plan irregularity, an index building is designed to represent the median value of every feature. Then, an index building is designed at each high and low secondary feature index, while keeping all other features at their respective medians. The methodology calls for fragility functions of two subsets, one with plan irregularity, and one without. Table 5 gives index values for both sets. In the subset (or bin) with plan irregularity present, all buildings are designed with the plan irregularity median index value for long and short plan dimensions of the building. It is important to design the building with plan irregularity in both directions because there are very few instances where plan irregularity exists in only one direction, though plan irregularity in the long short directions are considered independent of each other here. Using the median of the distributions seems reasonable and will save time. It is conceivable to design more sets of index buildings with plan irregularity at the high and low indices, but that is not done here. The uncertainty incurred by plan irregularity in different directions to different degrees is already considered by separating the data into subsets with and without the feature.

For the subset without plan irregularity, the index building design process is the same, but is completed using the index values from the data for the buildings without plan irregularity (Table 5). Each set contains 7 index buildings, resulting in 14 total index buildings that need to be designed for this illustration. A weight $w_{i\neq0}$ of 1/8 is assigned to each building designed at a high or low index, and w_0 of 1/4 is assigned to buildings designed at the all-median point. Table 6 indicates the values of the building features that each index building is designed for.

Table 6 - Index Buildings

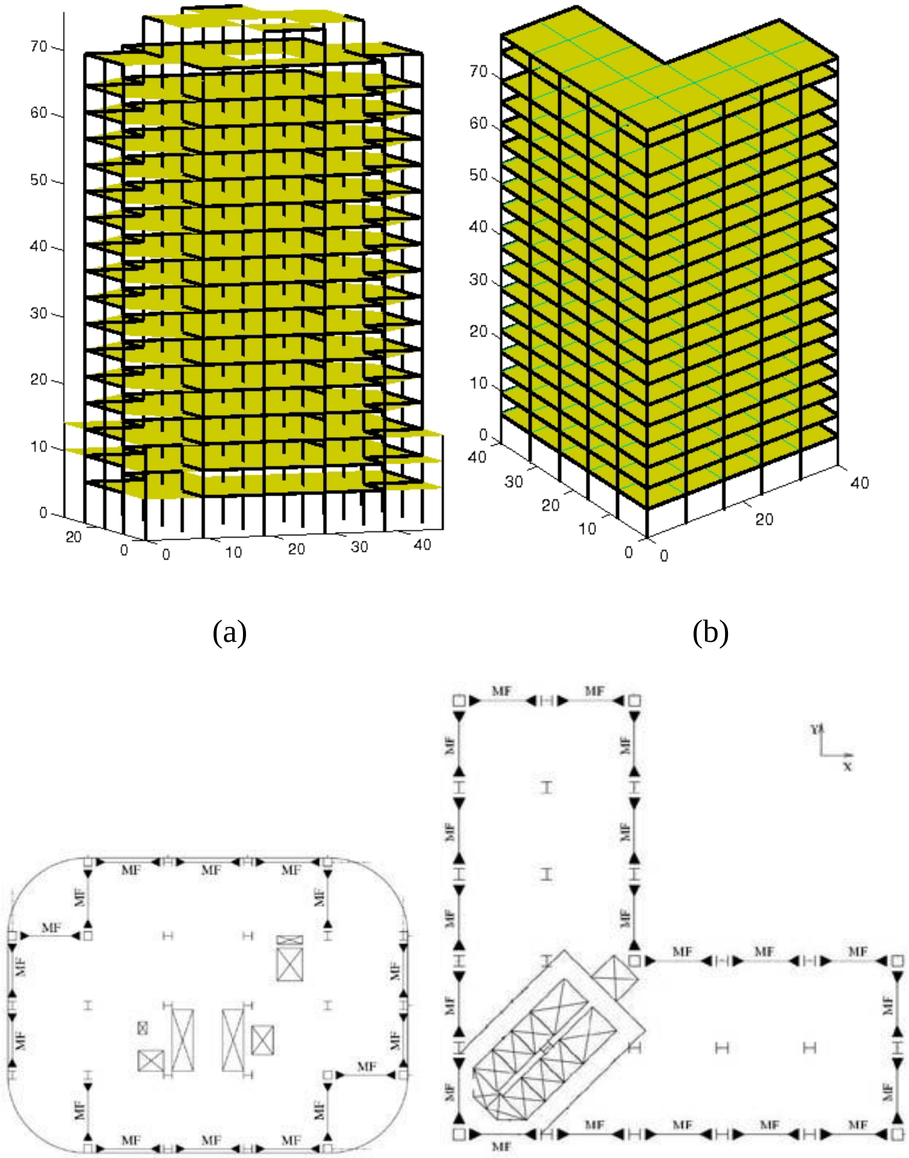
Index Building	# Stories	# Bays	Story Height Ratio	Plan Irregularity-Long Dir.	Plan Irregularity-Short Dir.	w_i
1	8	7	1.63	0.237	0.390	0.125
2	43	7	1.63	0.237	0.390	0.125
3	18	4	1.63	0.237	0.390	0.125
4	18	14	1.63	0.237	0.390	0.125
5	18	7	0.938	0.237	0.390	0.125
6	18	7	2.83	0.237	0.390	0.125
7	18	7	1.63	0.237	0.390	0.25
8	8	6	2.00	0.00	0.00	0.125
9	36	6	2.00	0.00	0.00	0.125
10	17	4	2.00	0.00	0.00	0.125
11	17	11	2.00	0.00	0.00	0.125
12	17	6	1.28	0.00	0.00	0.125
13	17	6	3.12	0.00	0.00	0.125
14	17	6	2.00	0.00	0.00	0.25

Per the shortcut mentioned earlier, let us imagine that each of these index buildings is designed and analyzed using PBEE-2 procedures for its collapse probability. Under that methodology, each index building model is subjected to a number of nonlinear dynamic structural analyses at each of a range of intensity levels. One then determines the fraction of buildings in each bin (with or without P) that collapse at each level of seismic excitation, and fit a collapse fragility function to each bin.

For the purposes of this study, fragility functions developed by Muto and Krishnan (2011) are borrowed. Those authors applied ground motion time histories calculated by physics-based modeling of the shaking resulting from a Mw 7.8 rupture of the southern San Andreas fault, calculated at 784 gridpoints spaced at about 4m throughout the San Fernando Valley, San Gabriel Valley, and the Los Angeles Basin. The analyses produce a maximum peak transient interstory drift ratio (IDR) for each building at each gridpoint. Models with peak IDRs of 0.075-0.10 are considered either partially or fully collapsed (CO). Models with peak IDRs of 0.05-0.075 are considered to be red-tagged, but not collapsed (RT).

For each building, Muto and Krishnan (2011) calculated the probability of collapse or red tagging, y_i, as a function of peak ground velocity (PGV). To these data they fit a fragility curve. Using Equations (**4**) and (5), the fragilities for all the buildings are combined into a cumulative fragility that represents the entire class of buildings with and without the feature.

Let us further imagine that the resulting collapse fragilities are those produced by Muto and Krishnan (2011) for their compliant-connection models with regular and irregular plan (their buildings 2 and 3). (Building 1 is designed to a previous building code and is omitted here). Building 2 is 18 stories tall and is a regular shaped building, meaning there is no plan irregularity present. Building 3 is a symmetric L-shaped building with P ≈ 0.605 (60.5%). See Figure 9 for building schematics and floorplans.

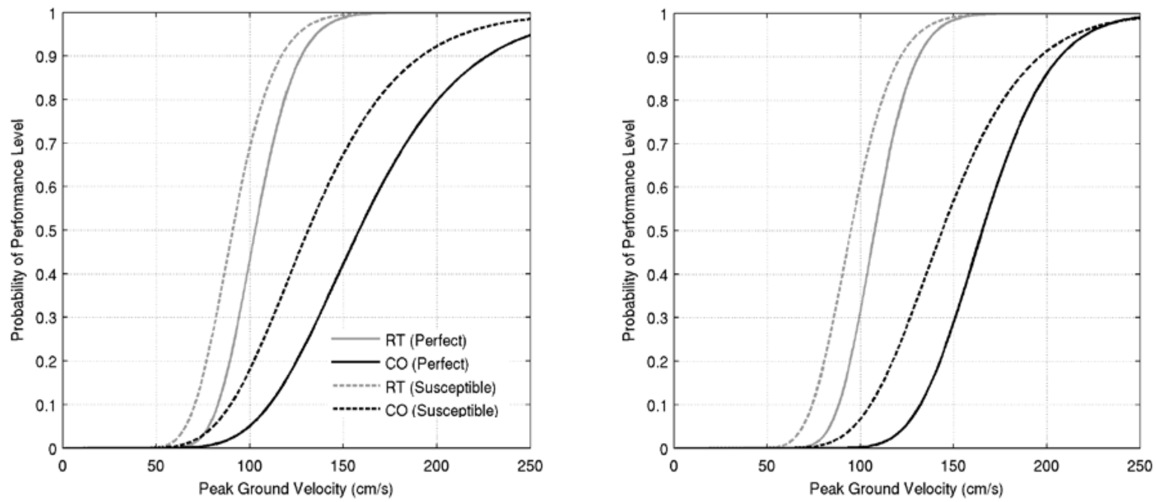


(a) (b)

(c) (d)

Figure 9 - Building 2 (Schematic (a) and Floorplan (c)), and Building 3 (Schematic (b) and Floorplan (d)) [Muto and Krishnan (2011)]

The fragility curve produced by Muto and Krishnan (2011) for the rectangular building will represent the fragility for the entire set of buildings without plan irregularity, $\bar{y}(x)$. The fragility of the L-shaped building will represent the curve for the set of buildings that do have plan irregularity, $\hat{y}(x)$. These cumulative fragility curves are shown in Figure 10 (a) and (b).



(a) (b)

Figure 10- Fragility Functions [Muto and Krishnan (2011)] for RT and CO Damage States for Buildings 2 and 3 for Perfect and Fracture-Susceptible Connections

The median, θ, and logarithmic standard deviation, β, for the fragilities of buildings 2 and 3 with perfect and fracture-susceptible connections for the red-tagged (RT) and collapse (CO) damage states were calculated by Muto and Krishnan (2011) and are shown in Table 7.

Table 7-Mean and Logarithmic Standard Deviation of the Fragility Functions for the RT (Red-Tagged) and CO (Collapse) Damage States for the Building Models with Perfect and Susceptible Connections

	Perfect Connections				Susceptible Connections			
	Red-Tagging		Collapse		Red-Tagging		Collapse	
BLDG.	θ	β	θ	β	θ	β	θ	β
2	102.91	0.1677	158.42	0.2807	90.84	0.1964	131.22	0.2965
3	107.68	0.1537	165.37	0.1752	94.9	0.1925	143.85	0.2425

Muto and Krishnan (2011) have provided values of θ and β for the fragility functions of two single buildings. Composing a single fragility from a suite of building fragilities would incur a greater scatter than the fragilities for individual buildings. One way to account for this greater uncertainty, without biasing the risk, is to increase the value of β while keeping the 10^{th} percentile fixed. This will increase the median. Let us denote the increased value of β as β' and new value of θ as θ'. The authors of HAZUS-MH (NIBS and FEMA, 2003) recommend β'=0.7 for the complete damage state of similar building classes, and that will be used here. Kennedy and Short (1994) claim that when the 10th percentile is fixed, the specific value of β will not have a significant influence on the analysis at higher levels of excitation. Equations (10) – (15) outline the process of establishing the 10th percentile and computing the new median.

$$\Phi^{-1}(0.1|\theta,\beta) = \Phi^{-1}(0.1\big|\theta',\beta') \tag{10}$$

$$\Phi\left(\frac{\ln\left(\frac{x_{10}}{\theta}\right)}{\beta}\right) = \Phi\left(\frac{\ln\left(\frac{x_{10}}{\theta'}\right)}{\beta'}\right) = 0.1 \tag{11}$$

$$\beta'\left(\frac{\ln\left(\frac{x_{10}}{\theta}\right)}{\beta}\right) = \beta'\left(\frac{\ln\left(\frac{x_{10}}{\theta'}\right)}{\beta'}\right) \tag{12}$$

$$\exp\left(\frac{\beta'}{\beta}\ln\left(\frac{x_{10}}{\theta}\right)\right) = \frac{x_{10}}{\theta'} \tag{13}$$

$$\theta' = \frac{x_{10}}{\exp\left(\frac{\beta'}{\beta}\ln\left(\frac{x_{10}}{\theta}\right)\right)} \tag{14}$$

$$x_{10} = \theta * \exp(-1.28\beta) \tag{15}$$

The process begins by setting the probability of collapse to 0.1 (10th percentile) given the original parameters. Through manipulation of the equations the median related to the increased logarithmic standard deviation is calculated using Equation (14). This process is denoted as rotation of the fragility because if plotted, the probability of collapse is increased after the 10th percentile and reduced before it (i.e., it appears to be rotates about a fixed point). Using β'=0.7, the rotation of the Muto and Krishnan's (2011) fragilities was performed, resulting in a new θ', as is shown in Table 8.

Table 8 - Fragility Rotation Calculation Results

	Perfect Connections				Susceptible Connections			
	x(10%)		θ'		x(10%)		θ'	
BLDG.	RT	CO	RT	CO	RT	CO	RT	CO
2	83.03	110.60	203.4	271.0	70.65	89.78	173.1	219.9
3	88.45	132.1	216.7	323.7	74.17	105.5	181.7	258.4

With the simplifications made for this research, this new fragility curve is the best estimate of the fragility for an entire class of buildings representing an increase in uncertainty from the individual building fragilities to the full suite of buildings. The median of each rotated fragility is higher than its respective median for the single building, resulting in the probabilities of collapse after the 10th percentile occuring at a higher levels of excitation. Fixing the 10th percentile preserves the long-term failure probability better than if the median was held because low levels of excitation are more likely to occur. See Figure 11 and Figure 12 for the rotated fragility functions of Building 2 (B2) and Building 3 (B3).

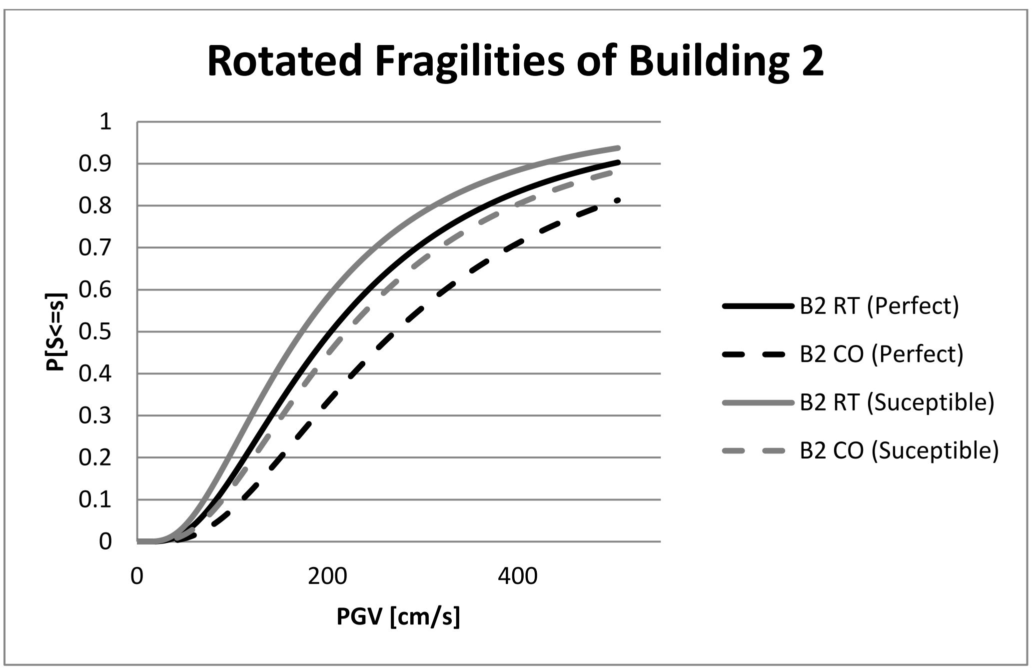


Figure 11 - Rotated Fragility Function of Building 2 (B2)

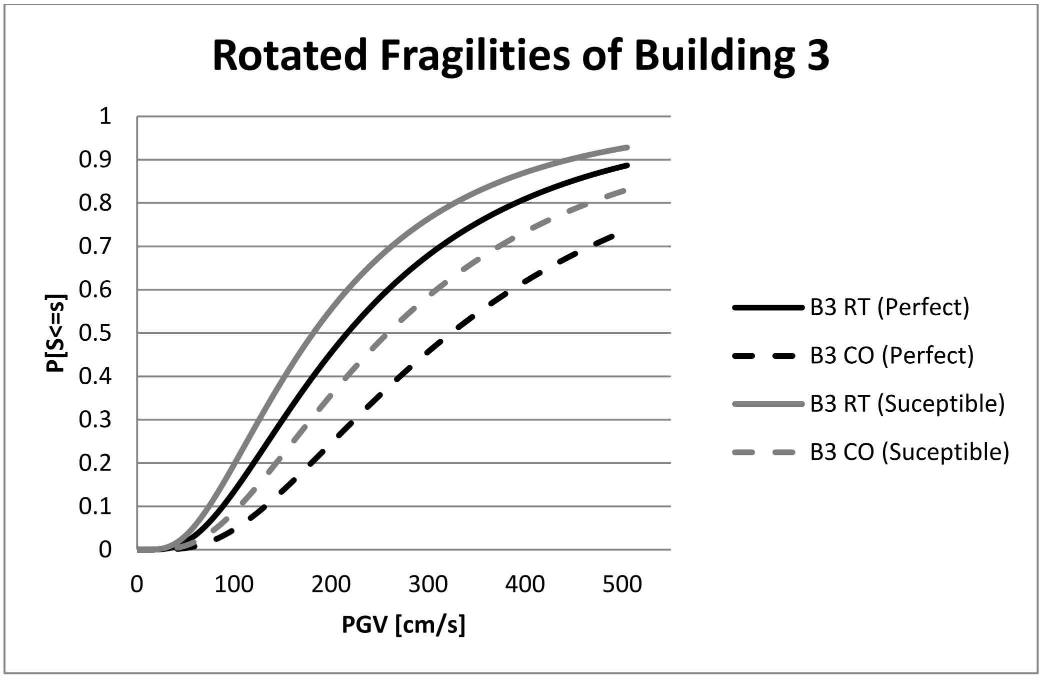


Figure 12 - Rotated Fragility Function of Building 3 (B3)

The rotated fragilities for Building 3 represent the combined fragility for the subset of buildings that were designed with plan irregularity (at its median). The rotated fragilities of Building 2 represent the combined fragility for the subset of buildings that do not have plan irregularity (at the median of the subset without plan irregularity). Since the effect of the building feature of interest has been defined as the ratio of the performance of the subset with the feature to the performance of the subset without the feature, the seismic performance modification factor is a function of the level of excitation (i.e. PGV), see Equation (16) and Figure 13.

$$SPM(x) = f(x) = \frac{Probability\ of\ Performance\ Level_{B3}(x)}{Probability\ of\ Performance\ Level_{B2}(x)} \qquad (16)$$

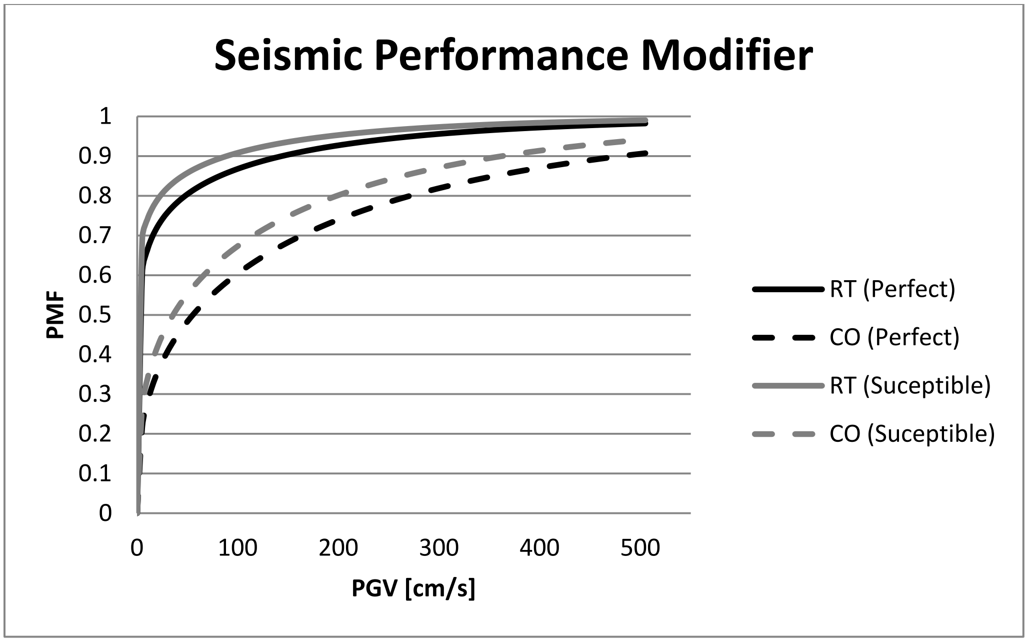


Figure 13 – Seismic Performance Modification Factor

Some applications may require a value of SPM at a specified level of excitation. The authors of FEMA 154 (ATC 2002a) chose the excitation level of 2% probability of exceedance in 50 years. On the U.S. Geological Survey Hazard Maps (USGS 2011), the entire region of interest is considered high seismicity. I chose the long period spectral acceleration (Sa) at 1.0 second because tall buildings (the chosen class is tall) have a longer period. According to the hazard map (Figure 14), the spectral acceleration in California ranges from 20 (%g) to 80 (%g).

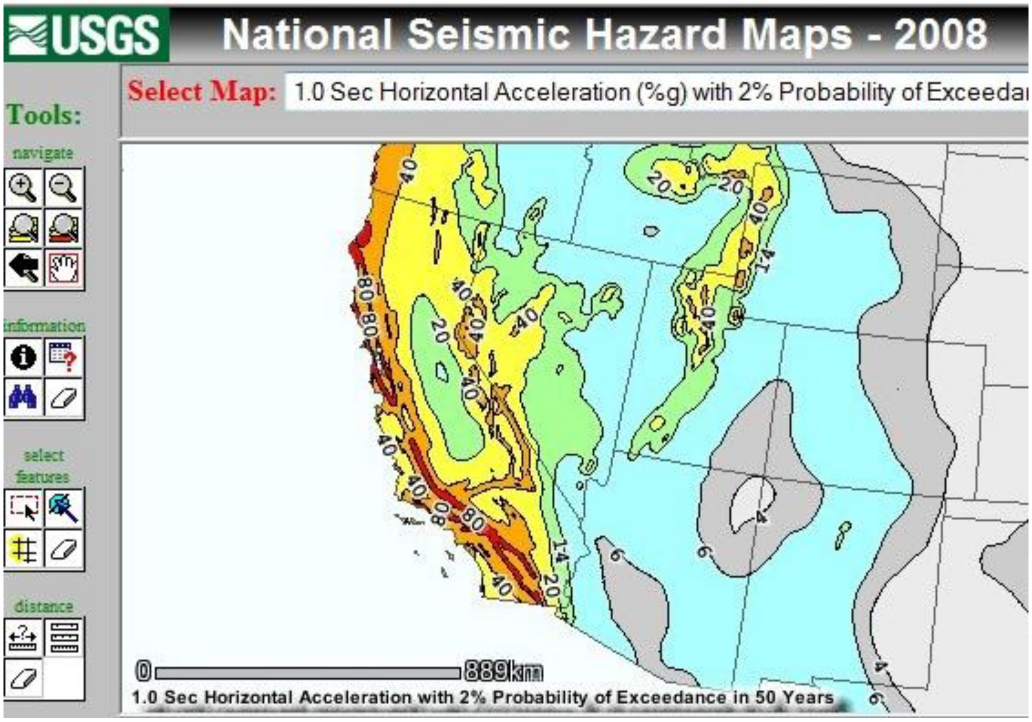


Figure 14 - USGS Seismic Hazard Map Showing Spectral Acceleration Values

To be conservative I will use Sa(1.0s, 5%) = 0.80g. In FEMA 154 (ATC 2002a) the values are then multiplied by 2/3, giving 0.533g. To find the seismic modification factor, the authors of FEMA 154 (ATC 2002a) consider the -$\log_{10}$(Collapse Probability|2/3 MCE,Sa(1.0s,5%)). The negative is incorporated because plan irregularity has a damaging effect on a building and a negative impact on the basic structural score. The modification factors in the rapid visual screening checksheet are positive if the feature will likely prevent earthquake damage and negative if it creates a potential for increased damage. The value of the plan irregularity performance modification factor used in the FEMA 154 Rapid Visual Screening (RVS) Data Collection Form is -0.5 (ATC 2002a).

Since the data and fragility relationships in this illustration so far are in terms of peak ground velocity, I need some way of relating them to FEMA 154 (ATC 2002a). I will use a relationship developed by Norman Abrahamson and cited in Anderson et al. (2008). Equation

(17) relates spectral acceleration, S_1 (at 1.0 second and 5% elastic damping) and earthquake magnitude, M, to peak ground velocity.

$$\ln(PGV) = 3.97 + 0.94\ln(S_1) + 0.013(\ln(S_1) + 2.93)^2 + 0.063M \qquad (17)$$

The PGV at S_1=0.533g and M=7.8, is 51.4 cm/s. At this value of PGV, Figure 13 shows 4 different values of SPM, this is because Muto and Krishnan (2011) designed two sets of 3 buildings (only two building data are used). One set consisted of the three buildings described with perfect connections, while the second set consisted of the same buildings but with fracture-suceptible connections. If this was another secondary feature it would have been part of the formation of the cumulative fragility, however this is not the case. The authors from FEMA did not distinguish between connection types and have one value of SPM for plan irregularity. Probability of red-tagging is not considered by the FEMA 154 (ATC 2002a) authors, and by specifying a set of buildings built after 1997, the fracture-susceptible connection data are not needed. At 51.4 cm/s, the SPM is -0.488.

This is not yet in the same domain as the modifiers in FEMA 154 (ATC 2002a). The authors create a basic score based that is S = –log10(y) (where y is collapse probability). To produce a modifier for features S1 and S0 will represent S evaluated for the set with the feature and without the feature respectively. DS will denote the modifier, which equals S1-S0. To evaluate the SPM from this research in the same domain, DSa will estimate DS based on y0 (collapse probability without the feature) and y1 (collapse probability with the feature). Estimating DSs is outlined in the following equations.

$$S1 = -log_{10}(y1) \quad (17)$$

$$S0 = -log_{10}(y0) \quad (18)$$

$$DSa = S1 - S0 = log_{10}(y0) - log_{10}(y1) \quad (19)$$

$$DSa = log_{10}\left(\frac{y0}{y1}\right) \quad (20)$$

Evaluated again at 51.4 cm/s, DSa = 0.31. This is slightly different than the modification factor of -0.5 that FEMA produced using expert opinion, but in the same range. Complete implementation of the methodology could result in a higher or lower SPM (or DSa) for this level of excitation. Therefore, the exact value in this illustration should not be used in further research or in a rapid visual screening process. What is important to note, however, is that the value is close to the value the expert panel produced to represent the modification factor in FEMA 154 (ATC 2002a). This shows that the methodology can result in a good estimate of the effect of a building feature on a building class. The value produced by expert opinion has not been validated either, but since the value produced through analytical methods is similar, it is likely they are both in the correct range. If the methodology were to be fully implemented, the SPM could be used.

5. Conclusions and Suggested Future Research

5.1 Conclusions

This work has presented and partly illustrated a new analytical methodology based on 2nd-generation performance-based earthquake engineering to characterize the effect of a readily observable building feature on the seismic performance of a class of buildings. Such a method does not seem to have been developed before. The alternatives are to use expert opinion and to evaluate the effect of the building feature using empirical evidence from post-earthquake observations. Both alternatives have their advantages and disadvantages. Expert opinion is relatively easy but fairly unscientific. An empirical approach requires substantial data that generally do not seem to exist in the public domain.

The method here can be used to develop a seismic performance modification factor (SPM) that relates the effect of a subset of the building category with the feature of interest to that of a subset without the feature of interest. A SPM could be developed for any or all observable features believed to have a significant influence on the seismic performance of a building class and repeated for any and all building classes. It could be used to check whether a feature of interest has a strong effect or not. It could be applied to a rapid visual screening process such as FEMA 154 (ATC 2002a), to public risk models such as HAZUS-MH (NIBS and FEMA 2003), or to commercial catastrophe risk models such as ST-RISK or the portfolio risk models of RMS, AIR, or EQECAT. The methodology requires creating a number of PBEE-2 building models with and without the feature of interest, where the models span a few of the features that are deemed to be most important to the seismic performance of the building class. By doing so, the methodology estimates the variability of performance within the building class, and within each subset of the class with and without the feature.

In the illustration of the effect of plan irregularity, the SPM is a function of excitation. To relate it to the modification factors in the data collection form of FEMA 154 (ATC 2002a), I chose the value of the SPM at the same hazard level as used in FEMA 154 (ATC 2002a). It was impractical to fully illustrate the methodology, since doing so requires PBEE-2 models of several buildings, so this study used Muto and Krishnan's (2011) models of post-Northridge welded-steel-moment-frame highrise buildings as proxies, and reevaluated their PGV-based fragility function in terms of Sa(1.0sec,5%), using a conversion by Abrahamson (Anderson 2008). This study finds a SPM = -0.488 and a DSa = 0.312, whereas the FEMA 154 (ATC 2002a) authors judged the same DS = -0.50. Considering the simplifications made, these two values are somewhat close, but perhaps a coincidence. However, the result produced by the illustration looks reasonable compared to what the FEMA 154 (2002a) authors produced by expert opinion. The value produced here may differ from the value produced through a thorough implementation of the methodology and should not be used as a factor in practice. But it shows that the methodology can produce a SPM that can be used to make decisions regarding the seismic performance of buildings.

Uncertainty in the methodology is treated two ways: uncertainty in the asset definition (i.e., designing the index buildings so that they approximate the variability of the asset class in question) is treated using moment matching. Moment matching is a generalization of Gaussian quadrature involving the selection of a few carefully chosen points on a probability distribution and assigning weights to them so that their weighted moments (mean, variance, etc.) match the first few moments of the continuous distribution in question. Uncertainty in the seismic hazard, structural response, damage, and loss are treated with standard PBEE-2 methods, which as of this writing is generally Monte Carlo simulation.

Moment matching may be less familiar to the reader than is Monte Carlo simulation. The reason for using it is that, while moment matching does require significant time and effort, it is probably more efficient than the Monte Carlo method. By using moment matching, only 14 index buildings need to be designed to span 3 independent variables with and without the primary feature of interest, rather than the many buildings (probably several dozen or more) that would be needed to represent the building class using the Monte Carlo method. Design and analysis of 14 index buildings is a time consuming and costly process, but not completely impractical. Muto and Krishnan (2011) designed and analyzed 6 buildings: one set of three designs with perfect connections, and the same set with fracture-susceptible connections. Deriving the SPM for a particular feature, P, is not something that would be done on a regular basis. Once the values had been found, they can be recorded and distributed for design, rapid visual screening, or research use.

Using as proxies the fragility functions derived from only two index buildings by Muto and Krishnan (2011) probably underestimates the uncertainty of collapse capacity for an entire building class. The fragility functions were therefore rotated, i.e., their uncertainty parameter increased to a reasonable value for an asset class, and their median value adjusted so that one point on the fragility function remains the same as before. Kennedy and Short (1994) suggest that if the x-value in a lognormal fragility function associated with 10% failure probability (x_{10}) is well established, the long-term risk is not particularly sensitive to the value of the logarithmic standard deviation (β). Therefore, I chose the $\beta = 0.7$ based on HAZUS-MH (NIBS and FEMA

2003), and rotated the Muto and Krishnan (2011) fragility functions about x_{10}, while having the higher β. The rotation appears to be a reasonable estimate of the fragility for a suite of buildings based on the results of the available models. If the methodology were properly carried out, i.e., with 7 index buildings per subset, there would be no need to perform this rotation, since their weighted average would have a more reasonable value of uncertainty.

5.3 Suggestions for Future Research

Any further research should begin by fully carrying out the complete methodology presented in this report, including the analysis for an entire suite of buildings with the primary and secondary features. Each index building should be designed and subjected to earthquake ground motion time histories and determine the response of the structure. It is not necessary to use interstory drifts to predict collapse probability and probability of red-tagging the same way that Muto and Krishnan did - nor is it necessary to consider perfect/fracture-susceptible connections the same way. The probability of collapse becomes the value of y_i for each building. Cumulative fragilities for the suites of buildings with plan irregularity and without plan irregularity are created using Equations (4) and (5). As in the illustration, the SPM (Equation(6)) is a function of excitation, but specific values can be singled out at specified levels of excitation. If the methodology is carried out to be related to FEMA 154 (ATC 2002a), spectral acceleration (rather than PGV) would be a more proper intensity measure.

The methodology laid out here could be carried out to determine the effect of each of the secondary features on the seismic performance of this particular class of buildings. To do this, simply switch the feature that will become the new primary feature with the old primary feature. Now the old primary feature becomes a secondary feature. With the feature distributions already known, the analysis can begin at the stage of designing index buildings. Index buildings designed for the median value of the new primary feature will still represent the correct distributions, however, the building designed at the high and low index for the new primary feature are no longer needed. It will be necessary to design the entire set of index buildings without the feature of interest, and the set without the previous primary feature is no longer needed. With fairly trivial modifications, the methodology could also be used to create analytical seismic vulnerability functions for other classes of buildings. It will always suffer a challenge of validation until sufficient empirical evidence can be collected to compare with the analytical approach, so it would be valuable to gather the necessary empirical data from a large earthquake, such as the Christchurch, New Zealand event of 2011.

This research is suited for use in rapid visual screening processes such as in FEMA 154 (ATC 2002a) and ATC-50 (ATC 2002). As the method results in a SPM similar to the modification factor found in the FEMA 154 (ATC 2002a) Rapid Visual Screening Process, one application of this method would be to apply it to the features in FEMA 154 (ATC 2002a) that have not been analytically derived, i.e., the plan and vertical irregularity modifiers

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