RAPID SPATIAL DISTRIBUTION SEISMIC LOSS ANALYSIS FOR MULTISTORY BUILDINGS

A Thesis

by

PANKAJ BHAGVATRAO DESHMUKH

Submitted to the Office of Graduate Studies of
Texas A&M University
in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

May 2011

Major Subject: Civil Engineering

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Approved by:

Co-Chairs of Committee,	John Mander
	Monique Head
Committee Member,	Dara Childs
Head of Department,	John Niedzwecki

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ABSTRACT

Rapid Spatial Distribution Seismic Loss Analysis for Multistory Buildings.

(May 2011)

Pankaj Bhagvatrao Deshmukh, B.Tech., Mumbai University, Mumbai

Co-Chairs of Advisory Committee: Dr. John B. Mander
Dr. Monique Head

Tall building frames that respond to large seismic ground motions tend to have significant spatial variability of damage over their height, often with a concentration of that damage in the lower stories. In spite of this spatial variability of damage, existing damage and loss models tend to focus on taking the maximum story drift and then assuming the same drift applies over the entire height, damage is then calculated for the building—clearly a conservative approach. A new loss analysis approach is thus recommended that incorporates the effects of spatial distribution of earthquake induced damage to frame buildings. Moreover, the approach aims to discriminate between required repair and replacement damages. Suites of earthquakes and incremental dynamic analysis along with the commercial software SAP2000 are used to establish demands from which story damage and financial losses are computed directly and aggregated for the entire structure. Rigorous and simplified methods are developed that account for spatial distribution of different damage levels arising from individual story drifts.

DEDICATION

Dedicated to my family

ACKNOWLEDGEMENTS

I would like to use this opportunity to thank my advisor, Dr. Mander, for his guidance and continuous support throughout the duration of my graduate studies. This research would not have been possible without his vision, inspiration and direction.

I would also like to thank my committee members, Dr. Head and Dr. Childs, for their support and feedback. I would like to thank the faculty and staff at Texas A&M for their assistance in administrative matters. I would also like to thank Christopher Urmson for his help in generating the finite element model.

I would like to thank my parents, my brother and other family members for their support and encouragement. This journey at Texas A&M would not have been possible without their support, blessings and love.

NOMENCLATURE

θ	Story drift
θ_c	Critical drift
θ_{on}	Story drift at onset of damage
θ_{DBE}	Story drift for design basis earthquake
β	Dispersion
β_U	Uncertainty in modeling
β_{RD}	Randomness in demand
β_{RC}	Randomness in capacity
$\beta_{f\|L}$	Dispersion in annual frequency, given loss ratio
$\beta_{L\|f}$	Dispersion in loss ratio given annual frequency
β_{UL}	Uncertainty in Loss estimation
CCANZ	Cement and Concrete Association of New Zealand
DAD	Damage Avoidance Design
DBE	Design Basis Earthquake
DS	Damage State
EAL	Expected Annual Loss
EDP	Engineering Demand Parameter
f	Annual frequency of earthquake
f_{DBE}	Annual frequency for design basis earthquake

f_{on}	Annual frequency of earthquake for onset of damage
f_{rr}	Annual frequency of earthquake when $L_{\max} = 1$
f_u	Frequency of earthquake when loss ratio is L_u
FEMA	Federal Emergency Management Agency
GIS	Geographic Information System
HAZUS	Hazards United States
HAZUS-MH	Hazards United States – Multi Hazard
HF2V	High Force to Volume
IDA	Incremental Dynamic Analysis
L_{on}	Loss Ratio at onset of damage
L_{DBE}	Loss Ratio for design basis earthquake
L_u	Loss ratio at collapse
MCE	Maximum Considered Earthquake
NIBS	National Institute for building sciences
NOAA	National Oceanic and Atmospheric Agency
NRC	Nuclear Regulatory Commission
PGA	Peak Ground Acceleration
PGD	Peak Ground Displacement
PGV	Peak Ground Velocity
SRA	Seismic Risk Assessment
WSMF	Welded Steel Moment Frame

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1. INTRODUCTION

1.1 Background

Construction of buildings, bridges and other civil structures require significant investments and natural calamities put these investments at risk. Natural hazards like earthquakes lead to damaged structures, which in turn may lead to loss of life and facility downtime. As a result a consistent effort is necessary to limit the financial loss, ensure life safety and reduce economic downtime arising due to loss of amenities. Rehabilitation work can start only after an assessment of damage and restoration cost is completed and financing is provided. In order to reduce economic downtime there is a need for a rapid analysis method to determine the extent of structural damage, and help decide an appropriate choice between repairing and rebuilding the structure.

Loss ratio which is the ratio of repair costs to reconstruction cost can be used to quantify structural damage and to help guide the rehabilitation strategy. There is a need to provide a loss estimation framework which can be used to estimate the financial losses after assessing the spatial distribution of damage across the structure and hence help in deciding the most suitable approach to restore the structure. This research focuses on the first of these, structural damage, and specifically, to discriminate between damage that requires repair versus damage that necessitates rebuilding. The proposed model helps in more accurately assessing Expected Annual Loss (EAL) for frame structures.

This thesis follows the style of *Journal of Structural Engineering.*

Mander and Sircar (2009) have proposed a simple, direct loss estimation methodology for seismically damaged structures by developing a loss modeling framework without the use of fragility curves. However in their previous studies, they assumed damage to be uniformly distributed over the height of the structure and governed by the maximum story drift alone. This approach does not account for spatial distribution of damage in a framed structure; clearly this is conservative. Moreover in their approach, Mander and Sircar (2009) did not specifically discriminate amongst the various types of damage, more specifically between damage that requires repairs versus the damage that requires complete reconstruction and replacement. Therefore, there is a need to assess total losses more accurately while accounting for story damage over the height of the structure.

1.2 Previous Work and the Evolution of Loss Estimation Methodologies

This section traces, in rough chronological terms, the evolution of seismic risk and loss analysis for structures. It first commences with the early work of Cornell (1968) and goes on to describe work done within and for the nuclear industry in the 1970's and 80's. In the 1990's a broader community interest regarding seismic risk arose and the Federal government through FEMA led an initiative that developed the HAZUS software platform. This work enabled the evolution of the seismic risk to complete communities. Following this effort, interest turned to using the NIBS / HAZUS techniques toward specific structures to enable complete loss analysis. This work was mostly conducted by various researchers within a broad range of universities.

1.2.1 Seismic Hazard Analysis

An engineer responsible for design of a project has to predict possible natural hazards and modify the design to mitigate the possible damage due to these hazards. Often in doing so a tradeoff has to be made between costly damage resistant structures and higher financial or economic losses. Due the uncertainty of the magnitude, location and occurrence of natural hazard, historically risk is expressed in terms of return period of the hazard. (Blume 1965, Blume et al. 1961, Housner 1952, Muto et al. 1963, Gzovsky 1962). Return period as a concept is open to abuse in its correct interpretation, therefore more recently the annual frequency or rate of an event of certain magnitude or intensity is considered more meaningful expression than the return period concept.

Cornell (1968) developed a method for evaluation of uniform seismic risk at the site of an engineering project and presented the results in terms of ground motion versus average return period. His pioneering study accounted for influence of all potential earthquakes and activity rates assigned to them and helped to make engineering analyses consistent with the seismic hazard information available. It helped in the determination of the rate of decay in risk with an increase in the resistance of a system's design. Such quantitative relationships facilitated in establishing reasonable tradeoffs with respect to operating regulations, below standard performance or system malfunction.

1.2.2 NRC and Seismic Risk Analysis

Kennedy et al. (1980) described the Seismic Risk Assessment (SRA) approach developed for use in the nuclear industry and the Nuclear Regulatory Commission (NRC). The probability of failure or damage to a nuclear power plant due to a seismic event was calculated and then compared with the probability of failure due to other events. They developed fragility curves for structures and equipment as a function of peak ground acceleration and used it to estimate the probability of nuclear core melt due to ground motions. This was done in three steps: (a) estimating ground motion in terms of peak ground acceleration (PGA) and its uncertainty as a function of annual probability of occurrence; (b) estimating conditional probability of failure and uncertainty of structural performance, equipment etc as a function of ground acceleration; and (c) combining the above two estimates to evaluate the probability of earthquake induced failure and additionally assess the uncertainty in this estimate.

Kennedy and Ravindra (1984) discuss key contributing factors to seismic risk, the significance of possible correlation between component failures and potential design and construction errors. They developed seismic fragilities of critical structures and equipment as families of conditional failure frequency curves plotted against peak ground acceleration. Based on the fragility evaluation of about a dozen nuclear plants they concluded that if energy absorption capabilities of a structure are properly accounted for, unnecessary conservatism in the seismic design could be done away with.

Kennedy (1999) developed a framework for a design criteria aimed at any desired seismic risk goal defined in terms of annual probability of seismically induced failure. A key feature of this framework was an establishment of acceptable seismic margin above the design safe shutdown earthquake. He observed that specific goals must be established for both seismic demand analysis and seismic capacity evaluation in order to approximately achieve the established target seismic margin.

1.2.3 FEMA and the Development of HAZUS

Seismic loss estimation methodology for civil structures has evolved over the period of four decades with initial surveys to estimate and predict seismic losses and led to development by the Federal Emergency Management Agency (FEMA) of software like HAZUS99 (NIBS 1999). Later this was expanded to incorporate multi-hazards in HAZUS-MH (NIBS 2003). The latter incorporates the losses due to seismic activity, hurricanes and flooding and is capable of estimating general structural damage from losses due to building damage and breakdown of public infrastructure using information within a GIS mapping framework.

Algermissen et al. (1972) studied seismic loss estimation by conducting a survey titled 'Earthquake losses for San Francisco Bay area.' The study was sponsored by National Oceanic and Atmospheric Agency (NOAA) and was one of the first in seismic loss estimation.

In NRC (1989), FEMA published the National Academy of Science report to estimate losses from future hypothetical earthquakes. This report focused on loss estimates intended for local and state governments to use in disaster response planning or help the formation of strategies to reduce hazard due to earthquakes.

FEMA (1992) entered into an agreement with the National Institute of Building Science (NIBS) to develop a nationally applicable standardized methodology for estimating potential earthquake losses on a regional basis. In 1993, the Project Work Group and Project Oversight Committee (both organized by NIBS) overlaid methodologies for seismic loss estimation. The groups were involved in preparing an extensive set of objectives for developing the methodology, and generating a standardized list of methodology outputs for earthquake related damage.

NIBS (1994) conducted a comprehensive survey of over thirty major regional earthquake loss studies to identify methodologies relevant to loss estimation. The methodologies were further evaluated to determine their potential as components of a standardized methodology. Deficiencies were identified in the methodology and studies were conducted to amend them. Beginning in 1994, the consulting firm RMS and a consortium of some thirty earthquake experts, developed the earthquake loss estimation methodology under contract to NIBS. This eventually led to the development of the HAZUS software.

HAZUS (NIBS 1999) is a powerful risk assessment methodology for analyzing the potential losses due to earthquakes. It provides an extensive coverage of seismic vulnerability information for diverse structure types for construction common in United States. In HAZUS, hazard related damage is estimated before or after earthquakes by coupling engineering knowledge with the latest GIS technology. The general procedure involves the classification of a damaged structure into one of five different damage states and generation of fragility curves for each damage state. Fragility curves are defined as cumulative lognormal distributions plotted against an intensity measure such as peak ground acceleration (PGA), peak ground velocity (PGV), peak ground displacement (PGD) or an appropriate spectral acceleration (S_a). In certain cases, loss ratios are assigned to specific damage states.

Whitman et al. (1997) summarized the development of applying GIS based analysis for regional loss estimation methodology in United States as funded by NIBS and FEMA. The methodology represents several important new advances in loss estimation technology. The paper provides an overview of the methodology, implementing software, and discusses its potential uses and applications. The methodology was implemented into the software package HAZUS that operates through a GIS application 'MapInfo.' A standardized methodology was laid out for estimating potential earthquake losses on a regional basis. A wide range of losses resulting from scenario earthquakes were evaluated to provide a basis for decisions concerning preparedness and planning of disaster response and to simulate and assist in planning to reduce potential future losses. The methodology relied on census tract information to aggregate the general building stock but was site specific regarding essential facilities and components of lifelines. A secondary purpose of loss estimates was to provide a basis for allocation of national resources and emergency funds for future seismic disasters.

Kircher et al. (1997) described methods for estimating the probability of discrete states of structural and nonstructural damage to buildings that were developed for the FEMA/NIBS earthquake loss estimation methodology. Probabilities of building damage states, which are based on quantitative measures of ground shaking were estimated and used by the FEMA/NIBS methodology as inputs to estimate of building losses, including economic loss and casualties. These functions represent a significant step forward in the prediction of earthquake impact, and consist of two basic components; (i) capacity curves which were based on engineering parameters like yield, ultimate strength and ultimate displacements; (ii) fragility curves which used the structural and nonstructural components to describe the damage to buildings. For a given level of building response, fragility curves distribute the damage between five physical states: None, Slight, Moderate, Extensive and Complete. As opposed to the earlier practice that used an approach based on Modified Mercalli Intensity, Kircher et al. (1997) used quantitative intensity measures (IM) of ground shaking and analyzed families of buildings similar to engineering analysis of a single structure. For validation, the economic losses predicted by these methods were compared with observed losses in Los Angeles County residences damaged by the 1994 Northridge Earthquake. The FEMA/NIBS methodology enabled quantitative evaluation of building losses and mitigation of alternatives that previously could only be judged in a qualitative manner. These tools enabled engineers and planners to develop strategies for earthquake hazard mitigation which could combine both elements of pre-event action and post-event response and recovery in a more rational manner.

Mander and Bazos (1999) proposed fragility curves for bridges to be used in seismic vulnerability assessment of a highway network as employed by the HAZUS software. These fragility curves were also used to estimate direct economic losses due to damage to a highway bridge. The authors quantified direct economic losses due to damage to highway bridges and estimated loss ratios for different damage states. Loss ratios defined as a ratio of repair cost to cost of reconstruction of a damaged structure, were proposed for different damage states. A new approach was recommended to develop fragility curves as opposed to earlier work which was based on empirical observations alone. Combining the fragility curves with repair cost ratios, estimates for the total loss were obtained for either scenario earthquakes or for discounted loss over life cycle of bridge. One of the applications of the result was to mitigate damage by retrofitting the structure.

HAZUS-MH (NIBS 2003) estimates financial losses to local community due to natural hazards like earthquakes, hurricanes and flooding. Losses are estimated at three levels of accuracy. A rough estimate of losses is calculated in a first level analysis and it is based only on available national databases and the HAZUS-MH software. Analysis at second level uses professional judgment, detailed information about demographic data, building and other information at local level to arrive at a more accurate estimate. Most accurate estimates are obtained from analyses at the third level and are based on engineering inputs and a customized methodology specific to that community. Losses analyzed by HAZUS-MH can be quantified into following categories: (1) physical damage such as damage to residential and commercial buildings and other infrastructure; (2) economic losses such as loss of jobs, interruptions in business and rehabilitation costs; and (3) social impacts such as shelter requirements, displaced households and population exposed to scenario floods, earthquakes and hurricanes.

Kircher (2003) described a loss estimation procedure for WSMF buildings. A static pushover analysis method was used to establish structural capacity whereas structural demand was established by defining a 5% damped response spectrum from a scenario earthquake. Loss functions were developed using building data, and probability of damage was calculated using fragility curves developed from pushover analysis results. Economic and functional losses were estimated by combining probability and damage loss functions. His paper provides structural engineers with tools to estimate structural damage and estimate financial losses based on quantitative description of ground motions and specific building types.

1.2.4 Nonlinear Analysis of Structures

Various software DRAIN 2D (Kanaan and Powell 1973), DRAIN 2DX (Prakash et al.1992), RAUMOKO (Carr 1998) and Opensees (Mazzoni et al. 2006) have been developed for advanced transient seismic structural analysis particularly to analyze nonlinear behavior of structures. However, all the above are university led research software tools and therefore do not necessarily have the official standing or credibility expected for industry-wide acceptance that fully commercial programs like SAP2000 (Computers and Structures 2009) have.

Hysteretic models like the well-known bilinear modal, the Clough degrading stiffness model (Clough and Johnston 1966), the Ramberg-Osgood model (Ramberg and Osgood 1943) plus numerous other variants have been proposed to describe the load-deformation characteristics of structures, members or hinges under reverse cyclic loading. The Clough degrading stiffness model was one of the first to include the effects of stiffness degradation. However, perhaps the most enduring model that is relevant to real structural systems is the Takeda model (Takeda et al. 1970). This is one of the main nonlinear models in the SAP2000 software.

Takeda et al. (1970) proposed a hysteresis model for predicting the nonlinear behavior of a reinforced concrete system. The response reflected the changes in stiffness of the structure during loading and unloading as a function of previous loading history and was based on a static force-displacement relationship. The proposed hysteretic model can successfully capture the continuously varying stiffness and energy absorption characteristics of a structure subjected to strong earthquake motions. The dynamic response was satisfactorily predicted by hysteresis loops defined by the proposed force displacement relationship and no other additional sources of energy absorption were required to predict the response.

Vamvatsikos and Cornell (2002) presented a detailed application of incremental dynamic analysis (IDA) and interpretation of the results to performance based earthquake engineering. IDA offers a thorough seismic demand versus capacity analysis through a series of nonlinear dynamic analyses using a multiple scaled suite of ground motions. A step-by-step approach was detailed to demonstrate practical application of IDA on a nine story steel moment resisting frame. Their paper also discusses various choices available to the user at each stage of the IDA and its implications on final result.

Dhakal et al. (2006) established a procedure to select a set of critical ground motions which can be used for physical testing or computational analysis. Incremental dynamic analysis (IDA) was performed using a suite of earthquakes on a finite element model of the structure. The results of IDA were analyzed and grouped into 50^{th} and 90^{th} percentile bands and critical ground motions that are close to these defining probabilistic curves at ground motion intensities corresponding to DBE and MCE are identified. These ground motions were identified as DBE (The Design Basis Earthquake having 10% probability in 50 years) with 90% confidence of non- exceedance and MCE (Maximum

Considered Earthquake having 2% probability in 50 years) representing the median response and MCE representing 90% confidence of non- exceedance.

1.2.5 The Stanford School of Thought

Shome et al. (1998) investigated the effects of scaling ground motions on the performance of multi-degree-of-freedom system's non-linear structural response for an earthquake. Using a model with five-degree-of-freedom, the authors demonstrated an appropriately chosen scaling factor reduced the number of nonlinear analyses by a factor of four, and proper scaling of ground motions did not introduce any bias. Also, when ground motion records were normalized or scaled to median spectral acceleration at fundamental frequency of the structure, as compared to unscaled sets, the median variables had reduced variability. The authors also observed that scaling the ground motion records to 5% damped spectral acceleration at fundamental frequency of the structure gave best results. The results from analysis were used in estimating annual probability of exceeding a specified inter-story drift or damage measure.

Shome and Cornell (1999) established efficient procedures for evaluation of nonlinear seismic behavior of multi-degree-of-freedom structures including the probabilistic analysis of this behavior. Procedures were developed for probabilistic seismic demand analysis, i.e., to estimate the annual probability of exceedance of seismic demand at a particular site due to further ground motions at that site. The demand hazard procedures were illustrated through 5 story and 20 story special moment-resisting frame buildings.

Porter et al. (2001) quantified structural losses due to seismic events by using an assembly based vulnerability of buildings to evaluate the performance of the structure. Simulation based approach was used to determine the structural response of various building component groups known as assemblies. An assembly is a group of any structural or nonstructural components like pipe fixtures, ceiling, beams, columns etc. This response was then applied to its fragility functions to simulate damage to each assembly in the structure. The total loss was computed by aggregating the losses due to damage in each assembly.

Goulet et al. (2007) analyzed a four story reinforced concrete building and estimated financial losses from structural damage. The analysis relates seismic hazard to collapse safety and economic losses. The authors quantified performance in the following categories: structural damage, non-structural damage, repair costs, collapse statistics and losses due to fatalities. The analysis accounted for the uncertainties arising due to record to record variation and structural modeling. Losses were estimated after evaluating extent of damage in each of the individual structural and nonstructural components in the building.

1.2.6 Loss Analysis Studies: Developments by Mander and His Co-workers

Robertson (2005), under the supervision of Mander at the University of Canterbury estimated the losses due to seismic damage to a ten story reinforced concrete building. She used the incremental dynamic analysis (IDA) to obtain structural response and categorized the results into the five HAZUS damage states based on the extent of the interstory drifts. Resilience curves, fragility curves and damage states were combined to estimate the expected annual loss (EAL). Robertson significantly simplified the analysis and loss modeling as damages were not computed separately for each individual component but a variable 'loss ratio' was used to describe structural and non-structural damage in the entire structure. Loss ratio is the ratio of repair costs to reconstruction cost of the damaged structure. She estimated expected annual loss (EAL) from financial losses in each damage state. The EAL of the structure can be used to compare different structures and find the one with superior performance.

Dhakal and Mander (2006) developed a financial risk assessment methodology for natural hazards to relate system capacity, demand and financial risk. A theoretical model was developed to estimate the losses to constructed facilities in terms of financial risk. Incremental dynamic analysis was used to assess the seismic response of the structure. Response was measured with engineering demand parameter (column drift) whereas ground motion was expressed in terms of intensity measure (Peak ground acceleration, PGA). Results from the analysis were used to measure record to record randomness in response of the structure. Since stakeholders find it easy to comprehend the extent of loss when expressed in terms of financial risk rather than in terms of structural damage, a methodology was established to estimate the overall risk to a facility when exposed to a natural hazard. The authors developed a financial risk assessment methodology to relate structural damage and financial loss.

Mander et al. (2007) investigated the structural response, damage analysis and financial losses in highway bridges using IDA in a performance based earthquake engineering context. The quantitative risk analysis procedure involved performing an

IDA on a finite element model of a structure after adopting suitable suite of earthquakes, parameterization of the IDA results into various percentile bands and integrating the results with respect to hazard intensity recurrence relations with probabilistic risk. The damage to the structure was quantified using the five predefined (HAZUS) damage states. The uncertainty in estimation of randomness in structural capacity and seismic demand was incorporated in the model along with epistemic uncertainty. The analysis used a quadruple integral total probability equation to estimate expected annual loss.

Solberg et al. (2008) established a rapid IDA-EAL method to assess seismic financial risk. The authors presented a simplified method to estimate EAL without conducting time-consuming nonlinear dynamic analyses or IDA. A probabilistic demand model is generated after accounting for epistemic and aleatory uncertainties in relationships between intensity measure and engineering demand parameters resulting from pushover analysis using implicit capacity spectrum formulation. Damage measures were established so that damage states can be defined and loss ratios assigned. Financial implications due to seismic damage were quantified by calculating EAL after integrating total losses over all likely earthquake scenarios. The methodology was verified by performing incremental dynamic analysis and processing the results using a novel distribution-free methodology. The applications of the proposed method were illustrated by comparing the seismic vulnerability of two highway bridge piers.

Mander and Sircar (2009) simplified the analysis by developing a loss estimation framework that bypasses the need of complex fragility curves. To achieve this, they developed an empirically calibrated loss model in the form of a power curve with upper and lower cutoffs. Unlike earlier practice, loss was not estimated based on damage states but rather was assumed to be a continuous function of a demand parameter such as drift. A four-step methodology was laid out to relate the hazard analysis to structural demand and loss ratio. The four steps progress from (a) hazard analysis; (b) structural analysis; (c) damage and hence repair cost analysis; and (d) loss estimation. These steps when plotted in log-log space lead to four inter-related diagrams or graphs: a→ b → c → d → a as follows: (a) the hazard analysis involves evaluating the seismic hazard at constructed facility site and generating intensity measures representing local hazard levels; this hazard analysis was based on a demand model developed by Cornell et al. (2002) relating intensity measure to engineering demand parameter (EDP); (b) the structural analysis used story drifts as the EDP to evaluate the structural damage; (c) the damage analysis consists of estimating damage or repair costs in terms of loss ratio; and (d) the loss analysis involves estimating structural and nonstructural damage.

1.2.7 Recent Work at University of Canterbury

Bradley et al. (2009) presented a seismic loss estimation methodology and applied it to a ten story reinforced concrete moment frame structure. This methodology quantifies the seismic risk of engineered structures and thus enabled a consistent communication and rational decision making for acceptance and mitigation of seismic risk. The authors illustrated the use of seismic loss estimation methodology to interpret seismic performance in terms of seismic demand and associated economic loss as a function of ground motion intensity. It was shown that economic losses due to nonstructural components and contents are significant over a large range of ground motion shaking intensities. They illustrated the use of expected annual loss within the decision making framework to make rational loss based decisions via a retrofit example.

Bradley et al. (2010a) analyzed the effects of different intensity measures on the seismic performance of a ten story reinforced concrete building. The authors used the following intensity measures to record performance of the structure: Peak ground acceleration, peak ground velocity, elastic and inelastic spectral displacement and spectrum intensity. Response of the structure was measured as peak story drift and maximum floor acceleration. All the intensity measures investigated were found to be insufficient in predicting the response in at least one of the following: magnitude of ground motion, source to site distance or ε. Therefore the authors suggest careful selection of suite of earthquakes to predict spatially distributed demands without significant bias. Losses were computed separately for each individual structural and nonstructural component for damage. Total loss in the structure is given by summation of losses in all the components in the structure.

From the relevant insights from previous work, it was observed that the assembly based vulnerability approach quantified the structural damage by computing losses separately for individual components of the structure leading to complex set of analysis (Porter et al. 2001; Goulet et al. 2007). Further computing losses for individual structural component for different intensity measures (Bradley et al. 2010b) make the results difficult to comprehend, and hence it is contended a simpler loss estimation methodology is needed.

Moreover, damage is assumed to be uniformly distributed across the height of the structure and it is related to maximum story drift in a structure (Robertson 2005; Mander and Sircar 2009). However inter-story drifts are not uniform but their magnitude decreases along the height of the structure (Bradley et al. 2010b). There is a need to provide a loss estimation framework which analyzes the structural damage and estimates losses after accounting for spatial distribution of damage along the height of the structure. Currently there is no mechanism which helps to discriminate damage that requires repairs versus damage that necessitates rebuilding. Such a framework should also account for the modeling uncertainties and help estimate the rehabilitation costs.

1.3 Research Objectives

a) To develop a loss estimation framework linking hazard analysis to floor by floor structural response so damage at each story in a frame structure is accounted for while estimating the overall loss.

b) To develop an algorithm to aggregate the various story losses and generate a loss model. To refine this loss model so that it helps in deciding an economical choice between repairing and rebuilding the damaged structure.

c) The above algorithm uses results from one scenario earthquake event that will have a certain annual probability of occurrence. The final objective is to then integrate losses from all possible scenario events (regardless of their probability of occurrence) to obtain an expression to estimate expected annual loss (EAL) for the loss model.

d) To validate the above loss modeling approach with a case study of a ten story reinforced concrete building using IDA and a three story welded steel moment frame (WSMF) steel structure in conjunction with the commercial finite element software (SAP2000).

1.4 Outline of Thesis

This thesis is divided into five sections. Following this introductory section, Section 2 discusses about the existing loss modeling techniques and goes on to develop a new loss modeling approach which accounts for spatial distribution of damage over the height of the structure. It is contended that the proposed model estimates the losses more accurately. The proposed model is validated for a 10 story reinforced concrete structure known as 'Redbook Building.' Further simplified loss estimation methods are proposed.

Section 3 of the thesis validates the proposed model for low rise, welded steel moment frame (WSMF) steel structures with three different types of beam-to-column connections.

Section 4 closes the thesis with summary, conclusions and recommendations for future research.

1.5 What Then Is Particularly New in This Thesis?

1. The proposed loss model provides a loss analysis framework that can be used for quick analysis of seismically damaged structures and assists in discriminating between structures that require repairs and the ones that necessitate rebuilding.
2. The proposed model accounts for spatial distribution of loss over the height of the structure while estimating the composite loss ratio of the structure.

3. The proposed model is validated for a ten story reinforced concrete structure using a finite element model generated in the well-known widely used commercial software, SAP2000.
4. The proposed model is also validated for a three story WSMF structure and the effects of spatial distribution of damage over the height of the structure while estimating the overall loss ratio of tall and short structures are evaluated.

2. LOSS MODELING AFTER ACCOUNTING FOR SPATIAL DISTRIBUTION OF DAMAGE

2.1 Introduction

Natural calamities like earthquakes, hurricanes etc, have the capacity to significantly paralyze economic activity in a region and cause loss of life and limb. In such cases it is important that the extent of damage be rapidly quantified and that finance is made available for rehabilitation work as quickly as practicable. Risk mitigation consists of predicting the catastrophic events and development of financial instruments which help limit financial loss (Mander and Sircar 2009). Over the years efforts have been made to predict the damage to a structure due to seismic events and estimate the losses. Several methods have been developed over the past two decades to estimate losses due to seismic damage to a structure. One common method is to employ the HAZUS (NIBS 1999) approach which classifies the damage severity into five different damage states and expresses this probabilistically in the form of fragility curves for each damage state. The total loss is obtained by aggregating the losses for each damage state for a given intensity measure. (Whitman et al. 1997; Kircher et al. 1997; Mander and Basoz 1999).

Another method called 'assembly based vulnerability' estimates loss ratio after detailed analysis of various assemblies of structural and nonstructural components in the structure (Porter et al. 2001). An assembly is a group of any structural or nonstructural components like pipe fixtures, ceilings, beams, columns etc. Fragility curves are developed for each assembly in the structure based on its damage state and total loss is obtained by summation of losses in each of the individual assemblies.

Dhakal and Mander (2006) developed a financial risk assessment methodology for natural hazards to relate system capacity, demand and financial risk. Losses to constructed facilities were estimated in terms of financial risk, by developing a theoretical financial risk assessment methodology.

Mander and Sircar (2009) developed a four step approach to estimate financial losses for seismically damaged structures. This method simplified the loss estimation procedure bypassing the need for developing fragility curves. The four steps can be summarized as: (a) hazard analysis (evaluating the seismic hazard at constructed facility site and generating intensity measures representing local hazard levels); (b) structural analysis (evaluating the structural damage model using engineering demand parameter (like story drifts); (c) damage and hence repair cost analysis; (estimating damage or repair costs in terms of loss ratio); and (d) loss estimation (estimating structural and nonstructural damage).

In previous loss analysis work on buildings Mander and Sircar (2009), Robertson (2005) have assumed damage to be uniform across the height of the frame structure. There is a need to estimate the loss after accounting for spatial distribution of damage across the height of the structure.

The objective of this section is to extend the four-step approach to estimate losses after considering the spatial distribution of damage across the height of the structure and develop a loss model which helps to discriminate the damage that requires repairs versus that damage that necessitates reconstruction of the structure. A loss estimation framework is presented where the story drifts, obtained from incremental dynamic analysis (IDA) of the structure are analyzed to determine the spatial distribution of losses and from these results total losses are assessed. The model also accounts for epistemic and aleatory uncertainties in the estimation of the composite building specific loss ratio. Loss ratio, defined as the repair or replacement cost with respect to the cost of renewal under steady state (non-disaster) conditions, is an effective tool to represent structural damage in terms of financial loss. Simplified and rigorous algorithms are proposed to estimate the effect of the spatial distribution of structural damage under a range of seismic conditions of increasing severity. The proposed approach is validated using a case study of the 'Redbook Building' (NZS1170 2002).

2.2 Loss Modeling Overview

The "Four Step" approach proposed by Mander and Sircar (2009) is used herein to estimate loss from structural damage. Their ideas were an expansion of the concepts derived from the relationships developed by Kennedy (1999) and Cornell et al. (2002). Kennedy (1999) presented seismic hazard recurrence relationship given by $f_o(IM) = k_o(IM)^{-k}$. This is a relationship between intensity measure (IM) and annual frequency (f_o) where, k and k_o are best fit empirical constants. Cornell et al. (2002) developed a relation between IM and EDP (drift) given by $D = aS_a{}^b$ where $D = \theta$ is drift and S_a is spectral acceleration; 'a' and 'b' are empirical constants.

Mander and Sircar (2009) developed their "Four Step" approach to estimate structural losses. Their loss estimation methodology is summarized here for the sake of completeness. The four steps can also be depicted on log-log graphs as shown in Figure 1, and summarized using following compound equation

$$\frac{L}{L_{DBE}} = \left|\frac{\theta}{\theta_{DBE}}\right|^{c} = \left|\frac{S_a}{S_{a\ DBE}}\right|^{bc} = \left|\frac{f}{f_{DBE}}\right|^{d} \tag{2.1}$$

in which, DBE = design basis earthquake; L_{DBE} = loss ratio for design basis earthquake, θ = engineering demand parameter (EDP) which in this case is the drift in a structure for the considered event; θ_{DBE} = story drift in a structure for the design basis event; $S_{a\ DBE}$ = spectral acceleration demand for design basis event; f_{DBE} = frequency of seismic event for design basis earthquake typically taken as 10 percent in 50 years (1/475); and k,b,c and d are exponents which are interrelated by the following

$$d = \frac{bc}{-k} \tag{2.2}$$

The above exponents are slopes of the four log-log plots, as shown in Figure 1. It should also be noted that the model in Eq (2.1) is represented by the median response and behavior curves. The intensity of damage, as defined by an EDP, is classified into the five damage states used in HAZUS (Kircher et al. 1997; Mander and Basoz 1999; Kircher 2003), that is: (1) none; (2) slight; (3) moderate; (4) extensive; and (5) complete. As shown in Figure 2, for an earthquake which generates a specified EDP, the total probable financial loss is sum of corresponding values for the damage states and is given by

$$L[EDP] = \sum_{i=2}^{5} P_i[EDP]L_i \tag{2.3}$$

in which $P_i[EDP]$ is probability and L_i is the loss ratio for the i^{th} damage state.

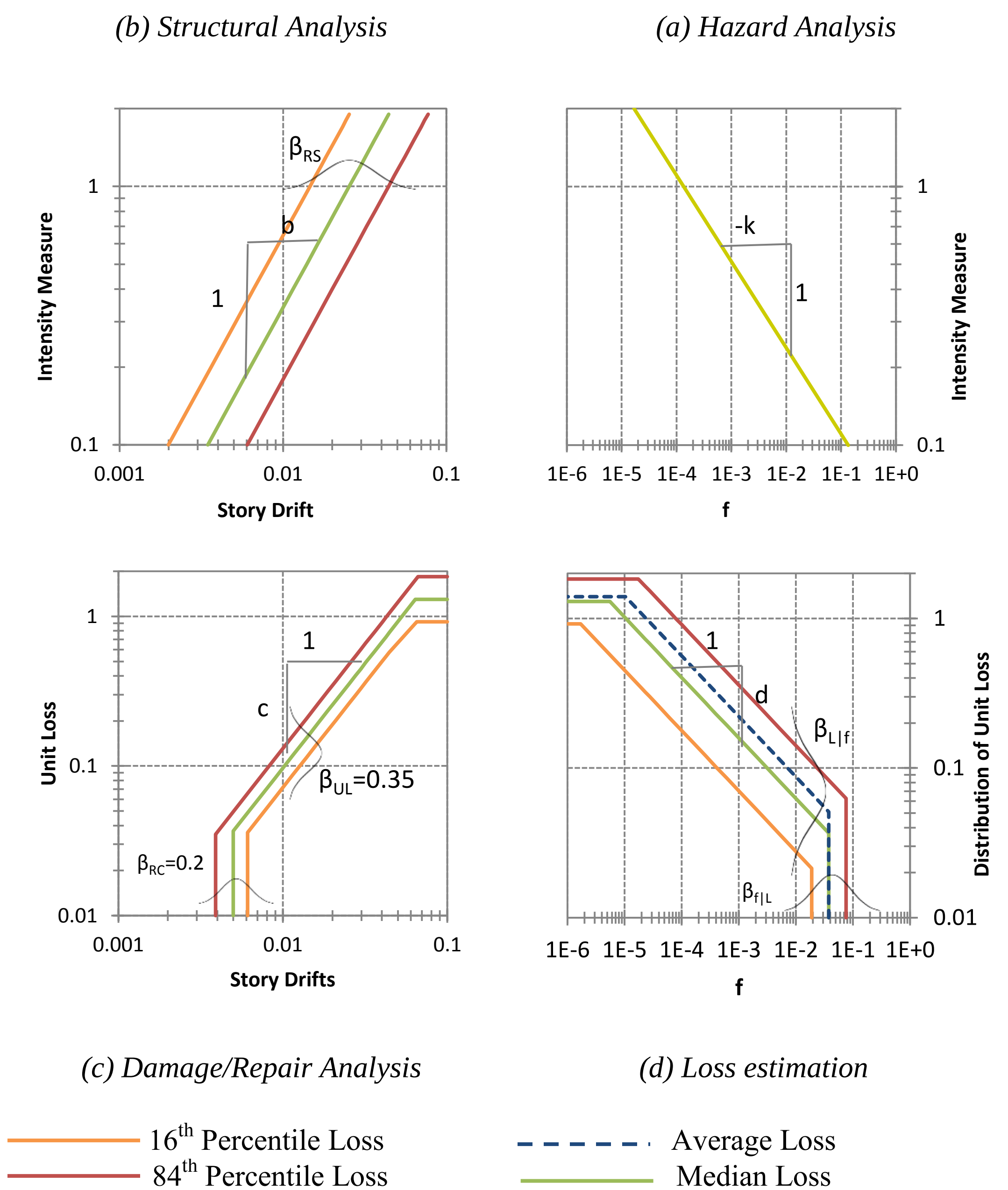


Figure 1: Loss estimation framework along with various dispersion factors used in the analysis.

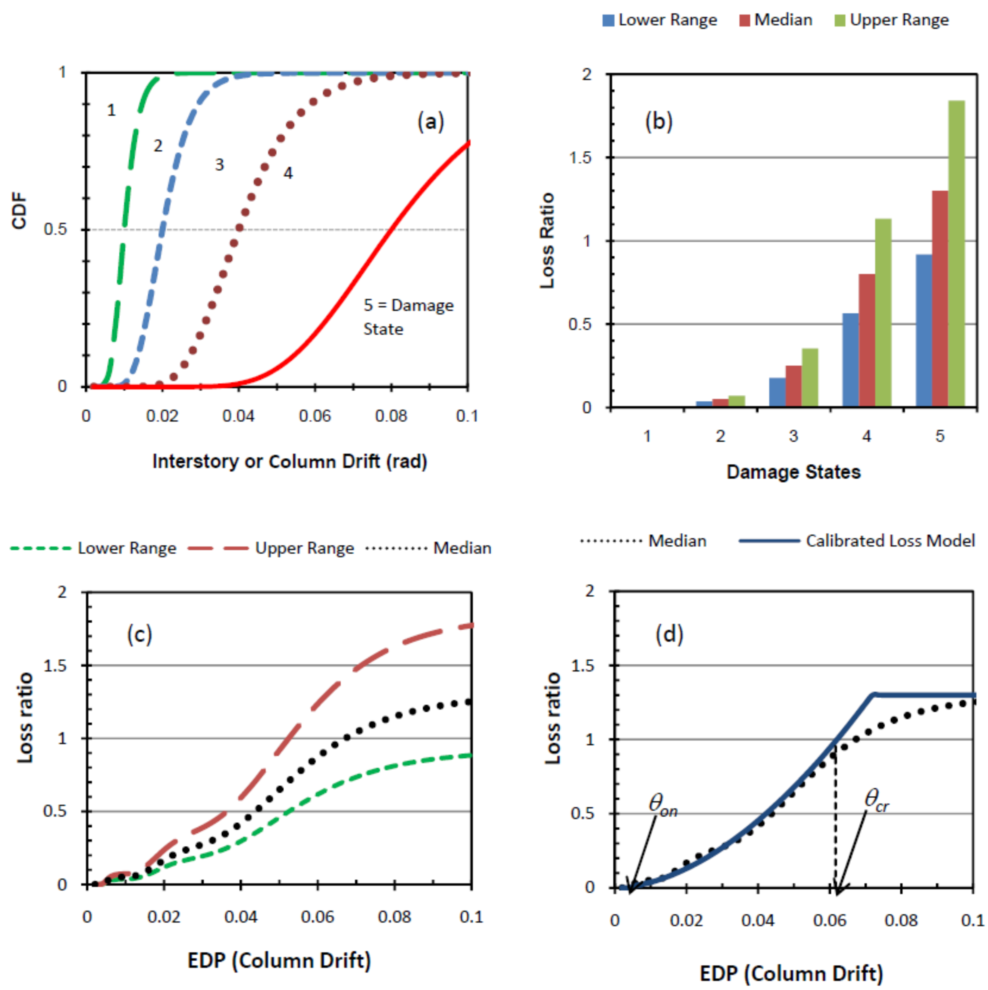


Figure 2: General procedure for estimating loss ratio used by Mander and Sircar (2009). (a) Estimate EDP-based vulnerability curves; (b) define damage states and corresponding loss ratios; (c) generate composite loss ratio by combining damage with losses for all damage states to give composite loss ratio; (d) parameterization of loss model.

The Loss model can be represented as

$$\frac{L}{L_c} = \left|\frac{\theta}{\theta_c}\right|^c \tag{2.4}$$

in which θ_c = critical drift $= f\theta_{DS5}$ where θ_{DS5} = drift at collapse; f = factor to adjust for low damage in structures; and L_c = unit loss. Eq (2.4) is also capped such that $L \le \tilde{L}_U$ Note that $L_u > 1$ to account for the expected post-disaster price surge of the repair and rebuilding process, where it is suggested a median value of L_u =1.3 be used. Also, when $\theta < \theta_{on}$, $L = 0$ where, θ_{on} = the onset of damage normally taken as “yield” of the structure.

Because the loss model developed above is not crisp, it incorporates epistemic and aleatory uncertainties in the loss estimation. The loss model conforms to a lognormal distribution and is described using median values and the lognormal standard deviation or dispersion associated with it. The dispersion of all combined uncertainty and randomness (β_{RS}) is given by root-sum-squares method. (Kennedy et al. 1980, Solberg et al. 2008)

$$\beta_{RS} = \sqrt{\beta_{RD}^2 + \beta_U^2 + \beta_{RC}^2} \tag{2.5}$$

where β_{RC} = randomness in capacity = 0.2; β_U = uncertainty in modeling = 0.25 (SAC 2000); and β_{RD} = randomness in demand. The dispersion in estimation of the annual frequency of event; $\beta_{f|L}$ for a given loss ratio is given by

$$\beta_{f|L} = \frac{k}{b}\beta_{RC} \tag{2.6}$$

The dispersion in loss estimation for a given annual frequency of event $\beta_{L|f}$ depends upon uncertainty in predicting capacity of the structure and on uncertainty in estimating losses for that capacity.

$$\beta_{L|f} = \sqrt{\beta_{UL}^2 + c^2 \cdot \beta_{RS}^2} \tag{2.7}$$

where β_{UL} = uncertainty in loss estimation = 0.35. (Mander and Sircar, 2009).

The expected annual loss (EAL) is given by the area under the average loss curve in Figure 1-d (Mander and Sircar 2009).

$$EAL = \frac{\bar{f}_{on}\bar{L}_{on} + d\bar{f}_u\bar{L}_u}{1+d} \tag{2.8}$$

where $(\bar{f}_{on}, \bar{L}_{on})$ and $(\bar{f}_u, \bar{L}_u)$ are the mean cut-off co-ordinates and are defined by

$$\bar{L}_{on} = \tilde{L}_{on}\exp(1/2\,\beta_{L|f}^2) \tag{2.9}$$

$$\bar{f}_{on} = \tilde{f}_{on} \tag{2.10}$$

$$\bar{L}_u = \tilde{L}_u \exp(1/2\,\beta_{UL}^2) \tag{2.11}$$

$$\bar{f}_u = f_{DBE} \cdot \left|\bar{L}_u / \bar{L}_{DBE}\right|^{1/d} \tag{2.12a}$$

$$\tilde{f}_u = f_{DBE} \cdot \left|\tilde{L}_u / \tilde{L}_{DBE}\right|^{1/d} \tag{2.12b}$$

in which, β_{UL} = 0.35 is uncertainty is loss estimation (Mander and Sircar 2009); β_{RC} = randomness in capacity of the structure = 0.2 (Solberg et al. 2008); $\bar{f}_{on}$ = mean frequency of earthquake at onset of damage and since a normal distribution in material yield point is assumed with a coefficient of variation of 20%, the normal standard deviation equivalent becomes β_{RC} = 0.2 and hence in Eq (2.10) $\bar{f}_{on} = \tilde{f}_{on}$ in which,

$$\tilde{f}_{on} = f_{DBE} \cdot \left| \tilde{L}_{on} / \tilde{L}_{DBE} \right|^{1/d} \tag{2.13}$$

$\bar{L}_{on}$ = mean loss ratio at the onset of damage and is given by

$$\bar{L}_{on} = \tilde{L}_{on} \exp(1/2\, \beta_{L|f}^{2}) \tag{2.14}$$

where $\tilde{L}_{on}$ is given by

$$\tilde{L}_{on} = \tilde{L}_{DBE} \left| \tilde{\theta}_{on} / \tilde{\theta}_{DBE} \right|^{1/d} \tag{2.15}$$

$\bar{L}_{DBE}$ is mean loss ratio for design basis earthquake given by

$$\bar{L}_{DBE} = \tilde{L}_{DBE} \exp(1/2\, \beta_{L|f}^{2}) \tag{2.16}$$

in which $\tilde{L}_{DBE}$ is median loss ratio for design basis earthquake and is given by

$$\tilde{L}_{DBE} = L_c \,|\, \tilde{\theta}_{DBE} / \theta_c \,|^{c} \tag{2.17}$$

where $\tilde{\theta}_{DBE}$ is obtained by regression of IDA results.

2.3 Development of Loss Model by Considering Spatial Distribution of Losses Over the Building Height

2.3.1 Maximum Loss Model

Although there is significant variability in the structural damage over the height of the structure, in the 'Maximum Loss Model' the damage in each story is assumed to be uniform and equal to loss obtained from maximum individual story drift in the structure. This approach is similar to the one followed by Mander and Sircar (2009) and Robertson (2005). One of the limitations of this model is that it provides quiet a conservative loss estimate. It ignores the spatial variability of damage in the structure while concentrating only on the story with maximum damage. The model can be expressed as

$$\frac{L_{\max}}{L_c} = \text{maximum}\left(\left|\frac{\theta_i}{\theta_c}\right|^c\right) \tag{2.18}$$

$$\left|\frac{\theta_{\max}}{\theta_c}\right| = \left|\frac{L_{\max}}{L_c}\right|^{1/c} \tag{2.19}$$

in which $\theta_{\max}$ = maximum story drift in the structure; θ_c = critical drift; and $L_{\max}$ = effective loss in the structure.

2.3.2 Average Loss Model

Spatial distribution of losses across the height of the structure can be analyzed by adapting the Mander and Sircar (2009) loss modeling framework presented above. The extension of their work is based on the simple idea of merely calculating losses for each story (not just the maximum as before), and then aggregating the individual story losses to develop the total building loss. Individual story losses are calculated from the individual story drifts (θ_i) obtained from IDA. Average Loss Model can be numerically expressed as:

$$\frac{L_{avg}}{L_c} = \frac{\sum_{i=1}^{n}\theta_i^c}{n\cdot\theta_c^c};\ L_{avg} \le 1 \tag{2.20}$$

$$\theta_{avg} = \left(\sum_{i=1}^{n}\frac{\theta_i^c}{n}\right)^{1/c};\ \theta_{\max} \le \theta_c \tag{2.21}$$

where n = total number of stories in the structure; θ_i = maximum drift in the i^{th} story; and θ_{avg} = the effective average damaging drift in the structure.

2.3.3 Proposed Model

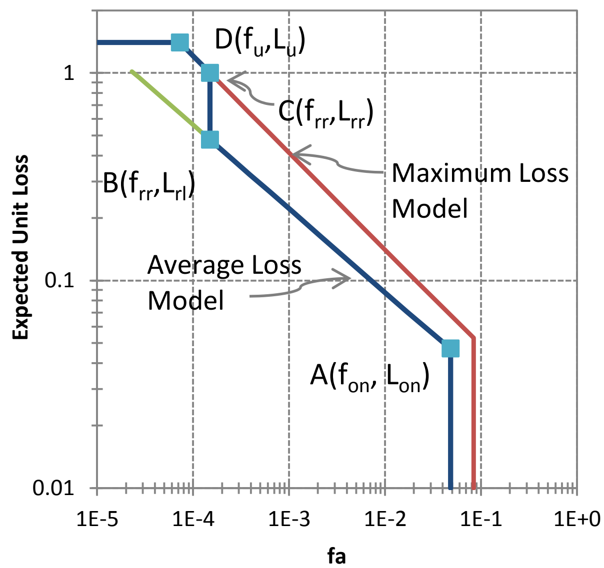
It should be noted that the total loss, when averaged amongst all floors, will inevitably be less than the maximum calculated story loss. This poses a problem when maximum loss in any of the individual stories, $L_{\max}$ tends to 1 ($L_{\max} \rightarrow 1$) as failure or rebuilding of that floor is necessary. In turn, this brings into question the viability of using the average loss alone. Therefore, it is suggested that a conditional loss model be adapted as follows.

$$\begin{aligned} L_{eff} &= L_{avg} \qquad (L_{on} \le L_{\max} < 1) \\ L_{eff} &= L_{\max} \qquad (1 \le L_{\max} \le L_u) \end{aligned} \tag{2.22}$$

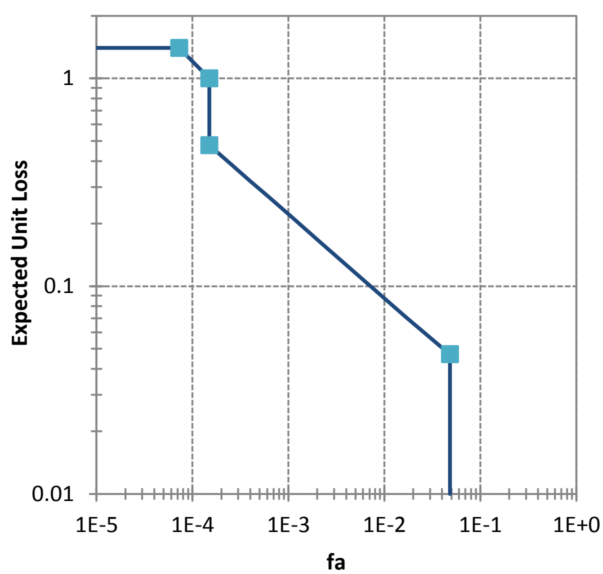
where $L_{\max}$ = the loss ratio obtained from maximum story drift in the structure, L_{eff} = the loss ratio for the proposed model. Conceptual construction showing the implication of the Eq (2.22) is shown in Figure 3a.

2.4 Simplified Analysis Methods

Loss analysis methods presented above involve rigorous structural analysis (IDA), quantification of the damage using engineering parameters and estimation of financial losses. These are very rigorous methods requiring skilled professionals and considerable computational time. In spite of this, the loss models are not without errors and the results may vary considerably. Herein simpler loss estimation methods are proposed which present a simple framework to estimate losses and do away with the need for rigorous structural analysis to quantify damage.



(a)



(b)

Figure 3: Construction of the proposed loss model from the Maximum and Average Loss Models. (a) Key coordinates necessary to develop the proposed loss model from the average loss curve of Maximum and Average Loss Models; (b) typical representation of the 'average loss curve' of the proposed loss model developed from the Average and Maximum Loss Model.

2.4.1 Root-Mean c Framework

The 'Root-Mean c' framework is a proposed empirical method to estimate average structural loss. If $\theta_{1,}\theta_{2\ldots}\theta_n$ are inter-story drifts, with $\theta_{\max}$ being maximum inter-story drift then the framework can be expressed as

$$\frac{L_{avg}}{L_{\max}} = \frac{\sum_{i=1}^{n}\left|\frac{\theta_i}{\theta_{\max}}\right|^c}{n} \tag{2.23}$$

This result is then used.

2.4.2 Modal Analysis

Mode shapes influence the deflected shape of the structure and hence it is contended that mode shapes can be used to estimate the losses in the structure. The mode shape for the first mode is normalized with maximum story displacement to be unity. Assuming these modal displacements to be story displacements, the average to maximum loss ratio can be calculated as per the 'Root Mean c' framework given by Eq (2.23).

2.4.3 Pushover Analysis

Pushover analysis is a very versatile tool to estimate structural capacity as well as to study post yield behavior of the structure. If story displacements are used from pushover analysis at $\theta = \theta_c$, the average to maximum loss ratio can be estimated using the 'Root-Mean c' framework given by Eq (2.23).

2.5 Computing Losses

Annual losses (AL) for the proposed model may be estimated by simply integrating the area beneath the average loss curve shown in Figure 3-b. The curve is plotted on log-log axes. The following integral may be used to estimate expected annual loss (EAL) for the proposed model

$$EAL = \int \bar{L}\, df \tag{2.24}$$

where $\bar{L}$ = average loss for a particular scenario event that has an annual frequency f .

The integral in Eq (2.24) has a solution given by following expression:

$$EAL = \frac{\bar{L}_u \bar{f}_u}{d_m + 1}\left[d_m + \left(\frac{f_{rr}}{\bar{f}_u} \right)^{d_m + 1} \right] + \frac{\bar{L}_{on} \bar{f}_{on}}{d_a + 1}\left[1 - \left(\frac{f_{rr}}{\bar{f}_{on}} \right)^{d_a + 1} \right] \tag{2.25}$$

in which d_m and d_a are given by

$$d_m = \frac{\ln\left(\bar{L}_u / L_{rr} \right)}{\ln\left(\bar{f}_u / f_{rr} \right)} \tag{2.26}$$

$$d_a = \frac{\ln\left(L_{rl} / \bar{L}_{on} \right)}{\ln\left(f_{rl} / \bar{f}_{on} \right)} \tag{2.27}$$

and $\bar{L}_{on}$ = Loss at onset of damage; $\bar{L}_u$ = Ultimate loss; $L_{rr} = 1$; $\bar{f}_{on}$ = annual frequency of earthquake at onset of ultimate loss; $\bar{f}_u$ = annual frequency of earthquake at ultimate loss; and f_{rr} = annual frequency of earthquake. These variables can be given by

$$f_{rr} = f_{DBE} \left| L_{rr} / \bar{L}_{DBE} \right|^{1/d} \tag{2.28}$$

L_{rl} is the loss ratio corresponding to annual frequency f_{rr} and can be expressed as

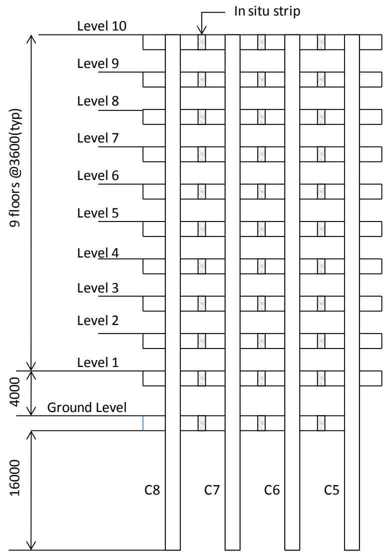
$$L_{rl} = \overline{L}_{DBE} \cdot \left| f_{rr} / f_{DBE} \right|^{d} \tag{2.29}$$

2.6 Case Study: The "Redbook Building"

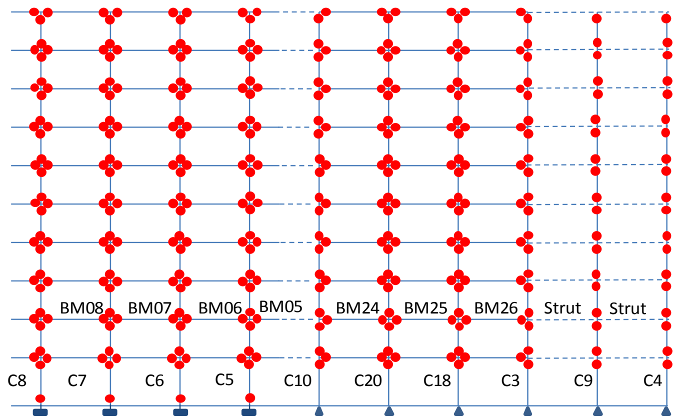
2.6.1 The "Redbook Building"

The proposed loss model was implemented on the "Redbook Building" (CCANZ 1998), a three bay, ten story reinforced concrete building designed to the New Zealand loadings standards, NZS3101 (1995) and concrete structures standards, NZS1170 (2002). In New Zealand this structure has been used as a basis of education of seismic design for university and engineering undergraduates and also practicing professionals for some two decades. It was selected as an example of current state of practice in New Zealand building design. The design philosophy used in these codes ensure the formation of a ductile structure strong-column/ weak-beam sidesway mechanism that is able to sustain large post-yield deformations to a target structure ductility of at least $\mu = 4$.

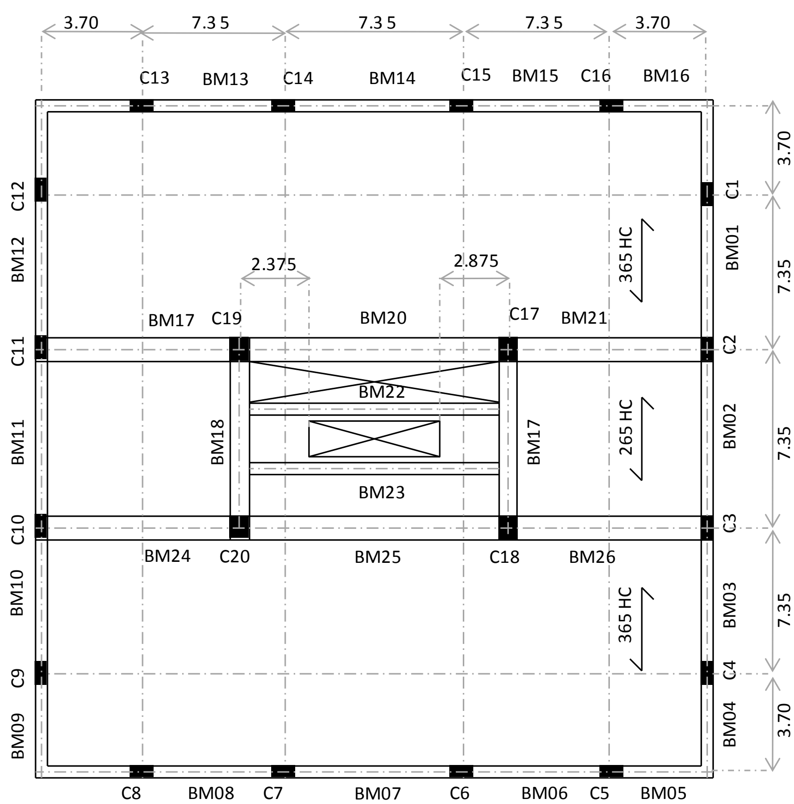
Figure 4 presents the Redbook Building which is a regular office building with floor area of about 900 m^2. The structure consists of four seismic perimeter frames designed to withstand lateral loads. The internal gravity columns are principally designed to bear the gravity load and are also detailed to undergo deformation imposed by lateral seismic frames. Beam and column reinforcement is considered to be uniform for all stories and were designed based on the design actions of the second story. Details of critical



(a) Elevation



(b) Extended 2D structural model developed in SAP2000



(c) Plan of Redbook Building

Figure 4: Prototype "Redbook building."

Table 1 : Redbook building- section details.

Element	Size	Longitudinal Reinforcement	Transverse Reinforcement
Perimeter Beams	900 x 400 mm	4-H24[1] Top 4-H24 Bottom	4 legs HR10[2]@140c/c
Cantilever Beams	900 x 400 mm	3-H24 Top 3-H24 Bottom	4 legs HR10@140c/c
Perimeter Columns at ground level	900 x 460 mm	12-H20	5 legs HR12 @ 90c/c 3 legs HR12 @ 90c/c
Perimeter Columns above ground level	900 x 460 mm	12-H20	5 legs HR12 @ 115 c/c 3 legs HR12 @ 115 c/c
Main interior beams	750 x 530 mm	Not Specified	Not Specified
Interior Columns	650 x 600 mm	Not Specified	Not Specified

[1] H24 is a reinforcement bar with 24mm diameter

[2] HR10 is steel stirrup with 10mm diameter

Table 2: Beam distributed gravity loads and cumulative tributary column axial loads for Redbook building under ultimate earthquake loads.

(a) Perimeter Frame

Floor Level	Beam UDL (kN/m)	Beam Point Loads (kN)	Cumulative Tributary Column Axial Loads (kN) Interior (A_{trib} =41.4 m^2)	Exterior (A_{trib} =40.2 m^2)
Roof	23.8	51.6	265	257
9	26.4	51.6	550	534
8	26.4	51.6	835	811
7	26.4	51.6	1120	1088
6	26.4	51.6	1406	1365
5	26.4	51.6	1691	1642
4	26.4	51.6	1976	1919
3	26.4	51.6	2262	2196
2	26.4	51.6	2547	2473
1	26.4	51.6	2832	2750

(b) Gravity frame

Floor Level	Beam UDL (kN/m)	Beam Point Loads (kN)	Cumulative Tributary Column Axial Loads (kN) Interior(A_{trib}=91.8m^2)	Exterior (A_{trib} =41.5m^2)
Roof	36.9	85.1	587	266
9	41.2	85.1	1219	551
8	41.2	85.1	1852	837
7	41.2	85.1	2485	1123
6	41.2	85.1	3117	1409
5	41.2	85.1	3750	1695
4	41.2	85.1	4382	1981
3	41.2	85.1	5015	2267
2	41.2	85.1	5647	2553
1	41.2	85.1	6280	2839

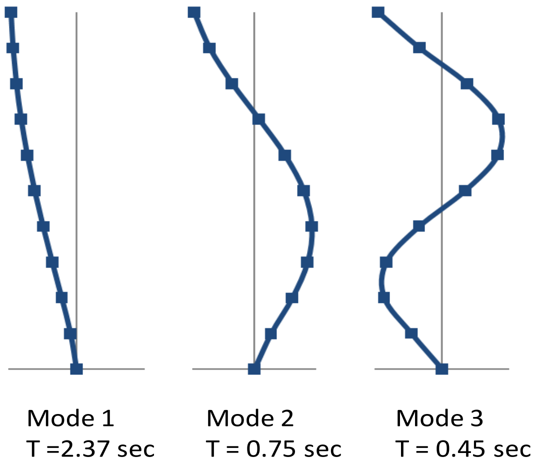
sections are tabulated in Table 1. The “Redbook” does not specify reinforcement details for gravity columns and their column capacities were assessed based on level of axial loads. The floor system consists of unidirectional precast hollow core concrete system with in-situ topping. Throughout the structure the strength of the concrete used was 30 MPa while that of steel was 430 MPa.

2.6.2 Modeling Details

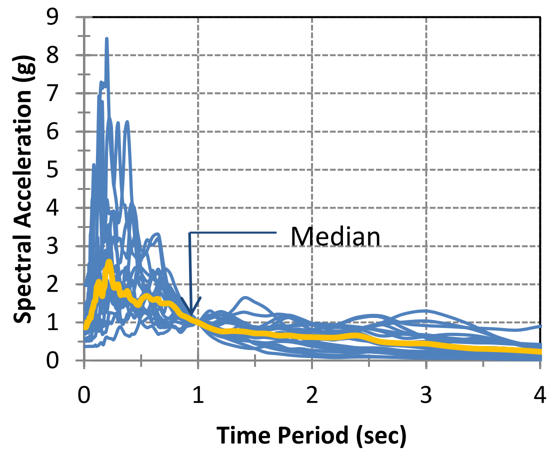
The commercially available widely used finite element software, SAP2000, was used in this study for modeling the structure. As depicted in Figure 4-b, one half of the Redbook frame was modeled consisting of a perimeter frame and a gravity frame connected by high stiffness pinned struts at every floor level. The floor diaphragms were assumed to be rigid.

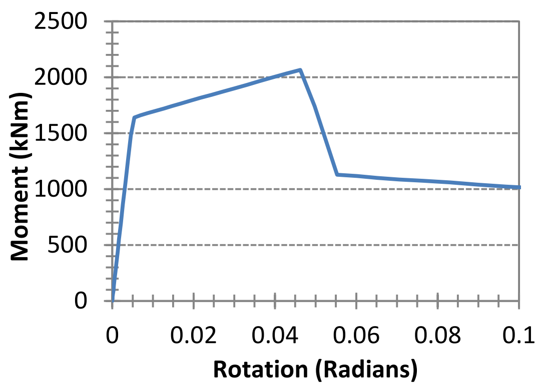
The structural loads were modeled using the self-weight option in SAP2000. The gravity load from walls on side frames are lumped on the end frame as point loads. A basic live load of 2.5 kPa is specified for Redbook Building in NZS 1170.5:2004 (SNZ 2004, SNZ 2006). A seismic floor load of 4.9 kPa was applied to beams at all levels below roof level. Different loads used in analysis are summarized in Table 2.

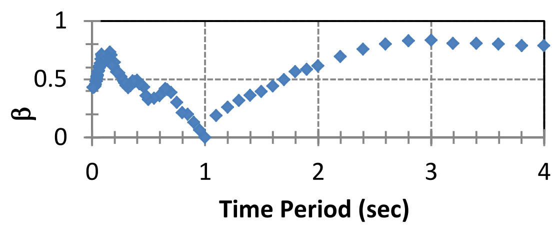
Modal analysis was carried using the ‘Eigen vector mode’ option in SAP2000. Figure 5-a shows the first three mode shapes of vibration for the Redbook Building. To account for cracking of the concrete, the beams were modeled with an effective stiffness of 0.2 EIg and columns are modeled with an effective stiffness of 0.5 EIg, where EIg = gross flexural rigidity. The structure has a fundamental period of 2.37 sec. Raleigh damping of 5% was specified for modes 1 and 9.



(a) Mode shapes and periods for modes 1-3

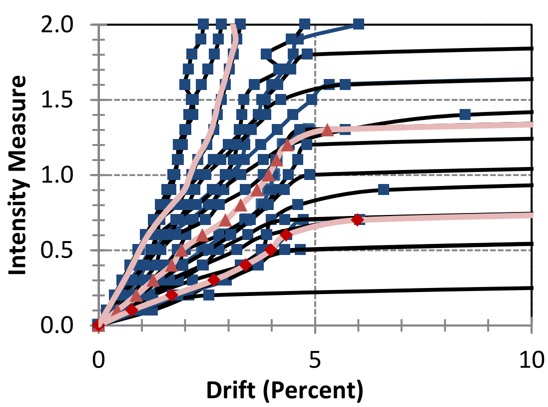


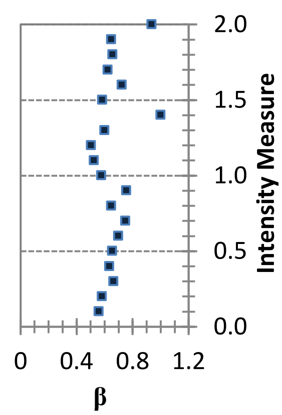




(b) Pushover analysis results for the hinge at the base story of column C5

(c) Acceleration spectra for the Vamvatsikos and Cornell (2002) suite of 20 selected earthquakes and dispersion.





(d) IDA curves obtained from time history analysis and dispersion, β

Figure 5: Results for analysis of “Redbook building.”

2.6.3 Nonlinear Modeling and IDA

Finite element software, SAP2000 was used for nonlinear analysis of the structure. Nonlinear behavior was induced with the application of plastic hinges in each beam at the face of every column, similarly for the column members. The length of the hinge was assumed to be one half the depth of the member. Links were modeled to follow the Takeda hysteresis rule in order to induce the nonlinear behavior of the structure (Takeda et al. 1970). The backbone curve used for defining links was obtained from moment - curvature analysis for each section. The algorithm used for this analysis considers the stress stress-strain relationships for confined and unconfined concrete, and steel to develop the moment-curvature relationship. The algorithm used the probable strength of concrete as 45MPa and steel strength as 450MPa for analysis. P-Δ effects were included in the analyses.

Pushover analysis was used to validate the structural model. Gravity loads were applied followed by lateral loads using the equivalent static method as described in NZS110.5:2004. Horizontal shear was calculated according to the New Zealand loadings standard (NZS1170 2002) for a ductile frame with μ=4 on intermediate soil with period of approximately 1.65. Table 3 gives the lateral load at individual floor level used in the analysis. The results for the pushover analysis at the base of column C5 are presented in Figure 5-b.

A suite of twenty ground motions used by Vamvatsikos and Cornell (2002) was used for the analysis. The selected set of ground motions belong to a class of magnitude of 6.5-6.9 events that occurred at moderate distances, all recorded on firm soil; these

Table 3: Vertical distribution of lateral forces.

Story	Weight (kN)	H_i (m)	W_iH_i (kNm)	F_i(kN)
Roof	6209	36.4	226008	460
9	6296	32.8	206509	283
8	6296	29.2	183843	252
7	6296	25.6	161178	221
6	6296	22	168512	190
5	6296	18.4	115846	159
4	6296	14.8	93181	128
3	6296	11.2	70515	97
2	6296	7.6	47850	65
1	6372	4.0	25488	35
	$\sum$ = 62949		**$\sum$ = 1268929**	**$\sum$ = 1888**

Table 4 : Details of 20 earthquake records used in IDA analysis.

No	Event		Station	ϕ° [1]	M[2]	R[3] (km)	PGA (g)
1	Loma Prieta	1989	Agnews State Hospital	90	6.9	28.2	0.159
2	Imperial Valley	1979	Plaster City	135	6.5	31.7	0.057
3	Loma Prieta	1989	Hollister Diff. array	255	6.9	25.8	0.279
4	Loma Prieta	1989	Anderson Dam	270	6.9	21.4	0.244
5	Loma Prieta	1989	Coyote Lake Dam	285	6.9	22.3	0.179
6	Imperial Valley	1979	Cucapah	85	6.5	23.6	0.309
7	Loma Prieta	1989	Sunnyvale Colton Ace.	270	6.9	28.8	0.207
8	Imperial Valley	1979	El Centro Array # 13	140	6.5	21.9	0.117
9	Imperial Valley	1979	Westmoreland Fire Station	90	6.5	15.1	0.074
10	Loma Prieta	1989	Hollister South and Pine	0	6.9	28.8	0.371
11	Loma Prieta	1989	Sunnyvale Colton Ace.	360	6.9	28.8	0.209
12	Superstition Hills	1987	Wildlife Liquefaction Array	90	6.7	24.4	0.181
13	Imperial Valley	1979	Chihuahua	282	6.5	28.7	0.254
14	Imperial Valley	1979	El Centro Array #13	230	6.5	21.9	0.139
15	Imperial Valley	1979	Westmoreland Fire Station	180	6.5	15.1	0.110
16	Loma Prieta	1989	WAHO	0	6.9	16.9	0.370
17	Superstition Hills	1987	Wildlife Liquefaction Array	360	6.7	24.4	0.207
18	Imperial Valley	1979	Plaster City	45	6.5	31.7	0.042
19	Loma Prieta	1989	Hollister Diff. Array	165	6.9	25.8	0.269
20	Loma Prieta	1989	WAHO	90	6.9	16.9	0.638

[1] Component. [2] Moment Magnitude [3] Closest Distance to Fault Rupture
Source: PEER Strong Motion Database, *http://peer.berkeley.edu/smcat/*

earthquakes are listed in Table 4. The selected ground motions were normalized to spectral acceleration of 1g at the natural period of 1 second for 5 percent damping, as shown in Figure 5-c.

Incremental dynamic analysis (IDA) was used to perform a series of time history analyses on the structure using the selected suite of earthquakes. The response of the structure was recorded in terms of story drifts. The intensity of the suite of earthquakes is scaled from 0.1g to 2g in increments of 0.1g. The results of the IDA analysis are summarized in Figure 6-c. Increase in magnitude of the ground motion may lead to numerical instability in the program implying structural collapse. However since the story drifts obtained from IDA for a particular IM are lognormally distributed, the missing drift values (due to numerical instability) were estimated by fitting lognormal cumulative distribution curve through available values using a least squares approach.

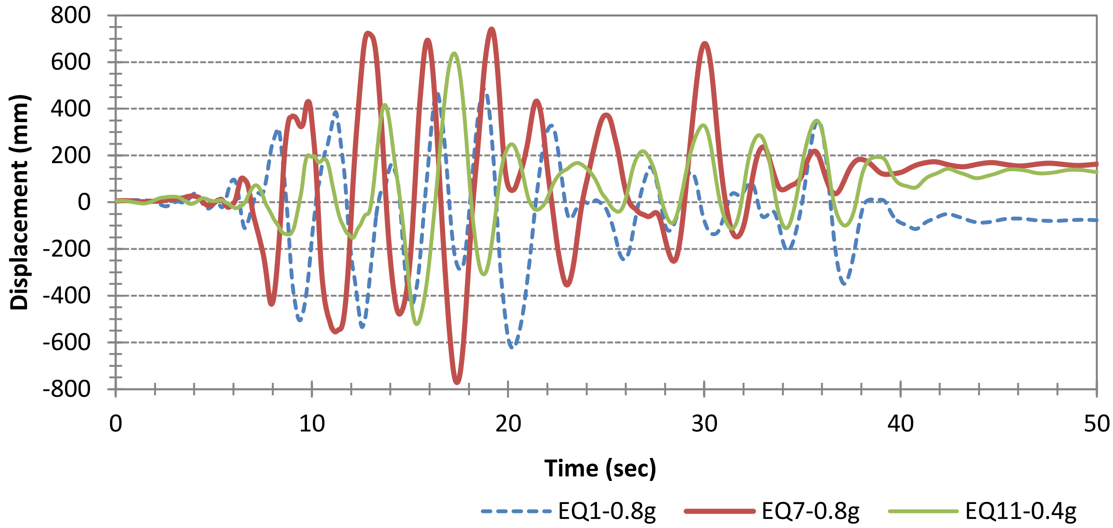
Time-history analysis results for some key critical earthquakes (Dhakal et al. 2006) are shown in Figure 6-a:

i. Earthquake 11 at 0.4g- Loma Prieta at Sunnyvale Colton Ace (N-S Component), 1989. (90^{th} percentile DBE)

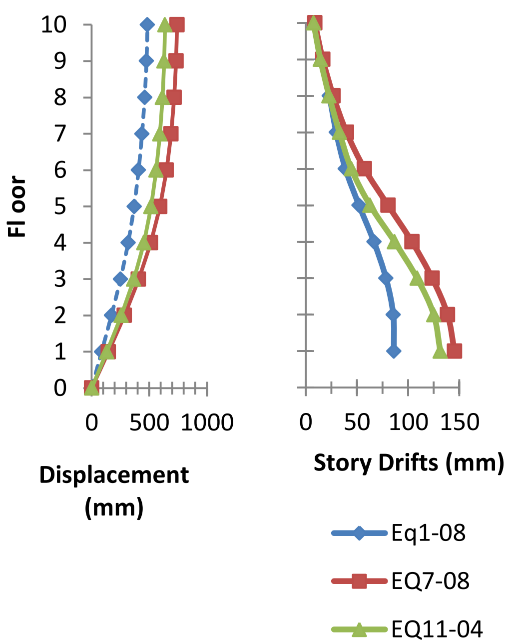
ii. Earthquake 1 at 0.8g –Loma Prieta at Agnews State Hospital (E-W Component) 1989. (50^{th} percentile MCE)

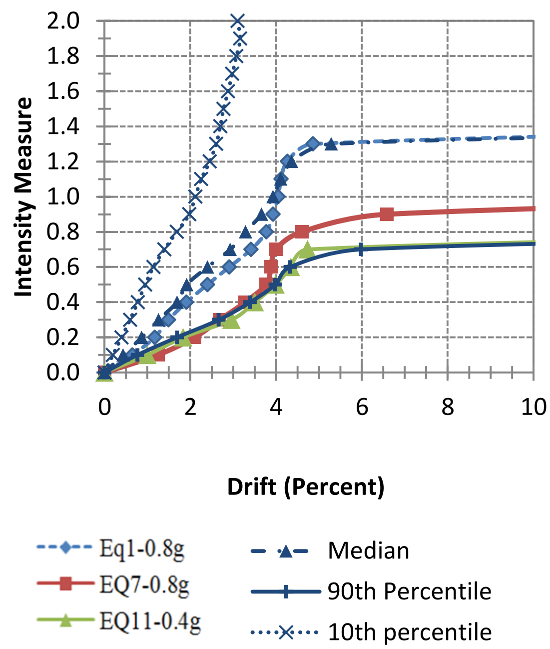
iii. Earthquake 7 at 0.8g – Loma Prieta at Sunnyvale Colton Ace (E-W Component), 1989. (68^{th} percentile MCE)

Selected results for these ground motions are shown in the Figures 6 and 7. Story drifts obtained from IDA analysis are used to validate the proposed loss model.



(a) Time history analysis for the selected ground motions.

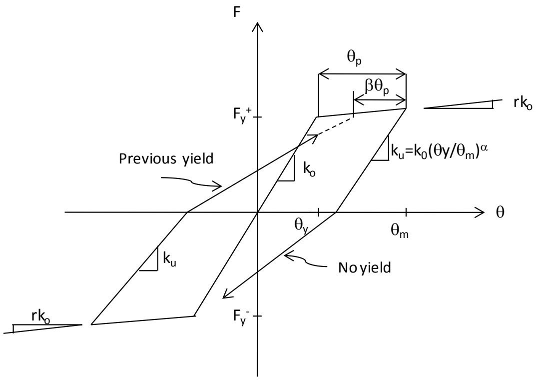




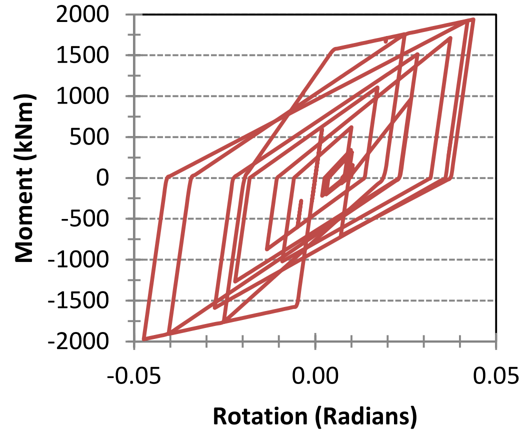
(b) Snapshot when displacement of roof is maximum.

(c) Selection of critical earthquakes using IDA curves

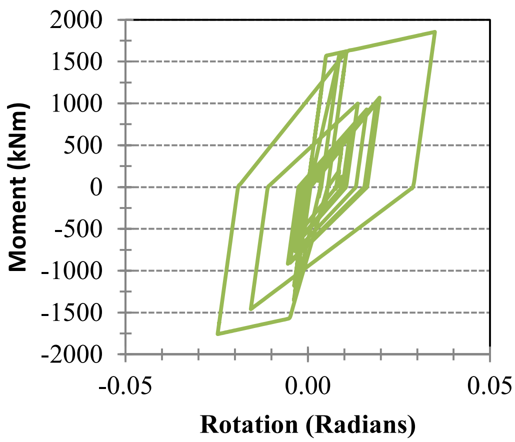
Figure 6: Results of selected nonlinear analyses. EQ 11 with IM =0.4g is representative of 90th percentile DBE. EQ 1 at IM=0.8g is representative of 50th percentile MCE (50%) and EQ 7 at IM=0.8g is representative of 68th percentile MCE. (IM = intensity measure).



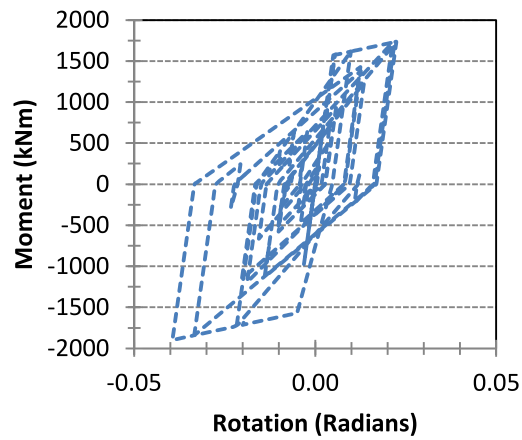
(a) Takeda hysteresis model



(b) Hysteresis loop due to earthquake 7 at IM= 0.8g



(c) Hysteresis loop due to earthquake 11 at IM= 0.4g



(d) Hysteresis loop due to earthquake 1 at IM= 0.8g

Figure 7: Hysteresis loops generated at the base of the column C5 from the selected critical earthquakes.

2.6.4 Results and Discussion

The loss model proposed above is applied to the Redbook Building. The results from incremental dynamic analysis (IDA) were analyzed separately to generate a 'Maximum Loss Model' and 'Average Loss Model' and finally a composite model was generated from the analysis. The results are summarized in Table 5. It tabulates median values necessary to describe the loss model along with the dispersion in the median values. The latter half of the table presents the mean values and expected annual loss (EAL) for three models.

The results are also presented graphically in Figures 8 and 9 for Maximum Loss Model and Average Loss Model respectively. The proposed model is presented in Figure 10. The drifts presented in Figure 8b-1 and 9b-1 are obtained from the IDA. Drifts in Figure 8b-2 and 9b-2 incorporate the randomness in demand and aleatoric and epistemic uncertainty (Solberg et al. 2008). The loss model in Figures 8-d and 9-d describe the loss ratio for ground motions with different intensity. It may be noted that because of dispersion induced due to randomness in the capacity of the structure, seismic demand and loss estimation there is a significant variability in the loss ratio for a given frequency of earthquake.

Comparing the loss ratios for the proposed loss model and 'Maximum Loss Model' for specific scenario earthquakes, it is observed that the loss ratio is considerably reduced in the proposed model as it considers spatial distribution of loss over the height of the structure. In case of design basis earthquake (DBE), loss ratio is reduced to 0.16 from 0.31. For MCE (maximum considered event having a probability of 2% in 50 years) the loss ratio for proposed model is 0.32, as compared to 0.68 for the Maximum Loss Model. Also, it can be seen from Figure 10 that an earthquake with probability of 0.75% in 50 years (return period ~ 6667 years), will lead to structural collapse.

The simplified analysis is used to evaluate the maximum to average loss ratio. Figure 11 presents the results for the simplified analysis. In case of the modal analysis loss ratio is equal to 0.7, whereas for the pushover analysis loss ratio is equal to 0.63. These values are adequately close to the loss ratio obtained from the Average Loss Model which is equal to 0.48.

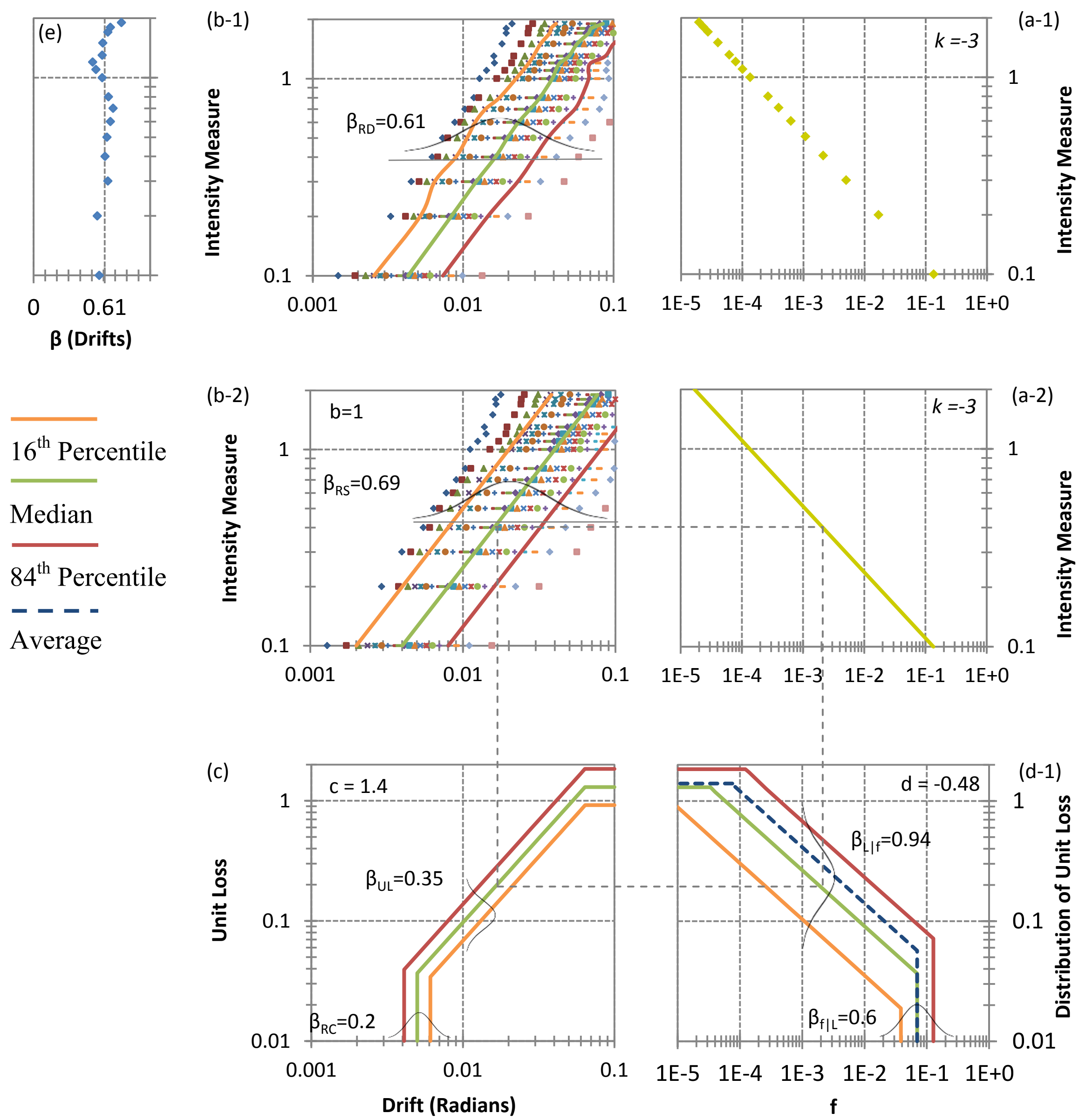


Figure 8: Step-by-step approach to calculate losses from story drifts using "Maximum Loss Model." (a-1) Hazard recurrence relation; (b-1) story drifts obtained from IDA; (b-2) story drifts along with uncertainty in modeling and randomness in capacity; (c) loss ratio obtained from story drifts; (d) estimated loss after incorporating the aleatory and epistemic uncertainty. EAL is given by the area under the "Average loss curve."

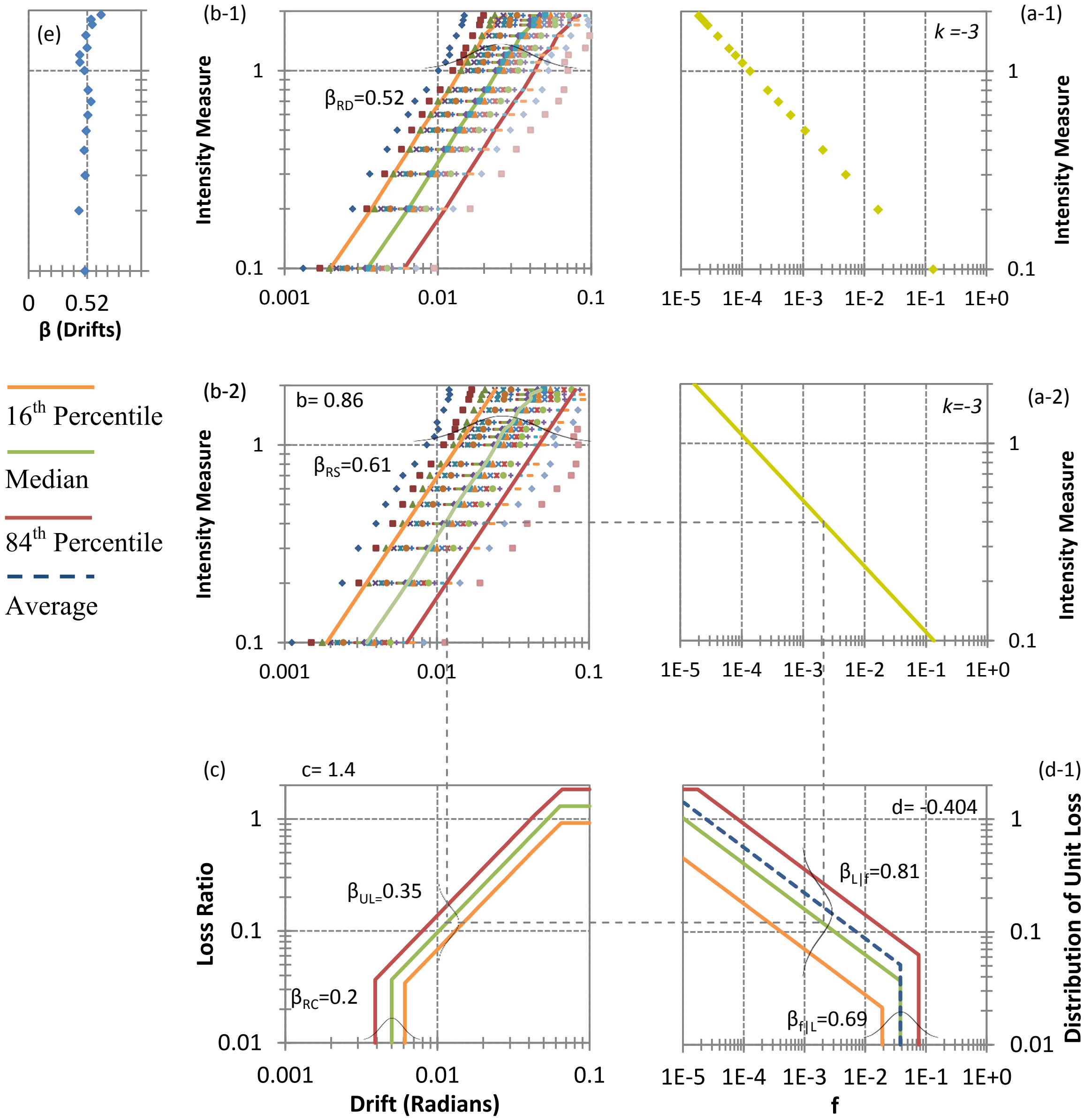


Figure 9: Step-by-step approach to calculate losses from story drifts using "Average Loss Model." (a-1) hazard recurrence relation; (b-1) story drifts obtained from IDA; (b-2) story drifts along with uncertainty in modeling and randomness in capacity; (c) loss ratio obtained from story drifts; (d) estimated loss after incorporating the aleatory and epistemic uncertainty. EAL is given by the area under the "Average loss curve."

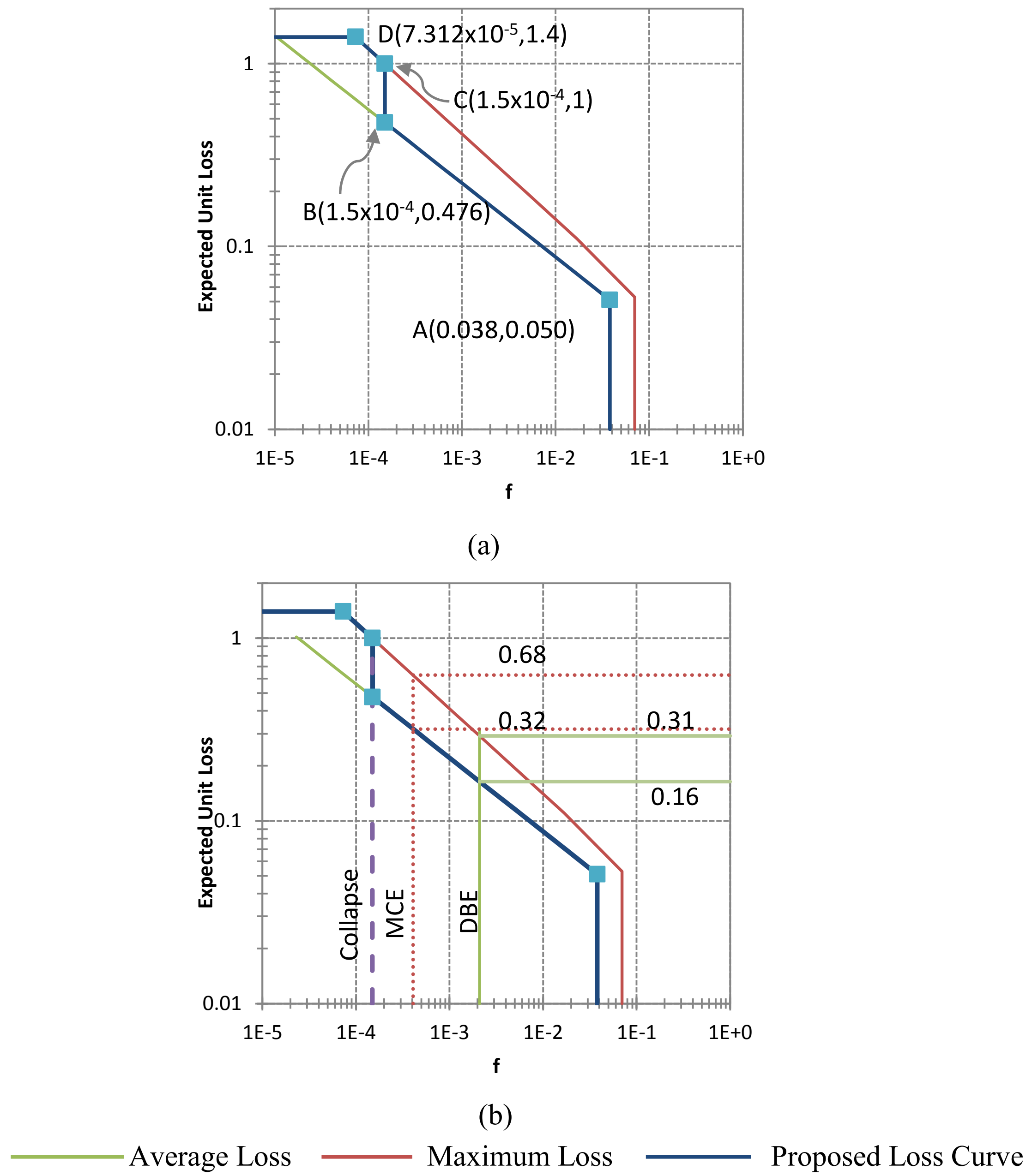
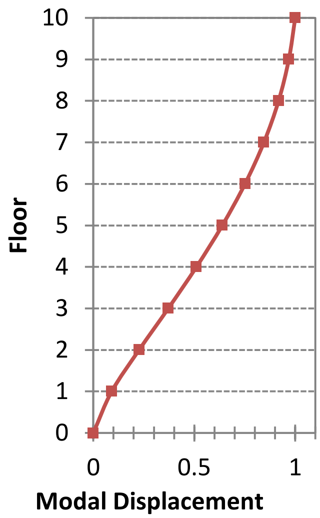
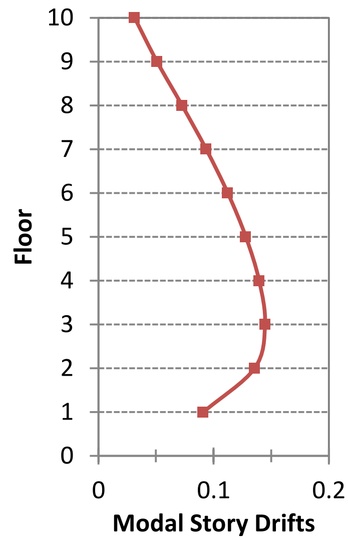


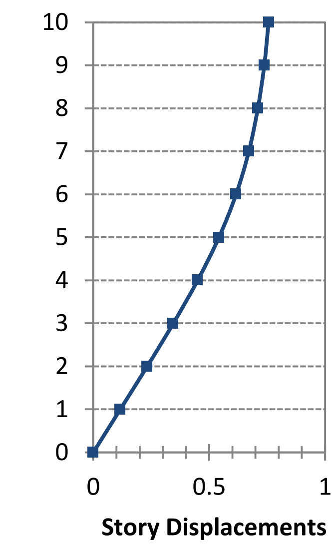
Figure 10: Proposed loss model. (a) Steps to generate proposed loss model. The three curves above represent the average loss curve for Maximum Loss Model, Average Loss Model and the proposed loss model. The area under the average loss curve gives the EAL for the structure. (b) Results for different scenario events used to compare the loss estimation results with and without spatial distribution of loss.



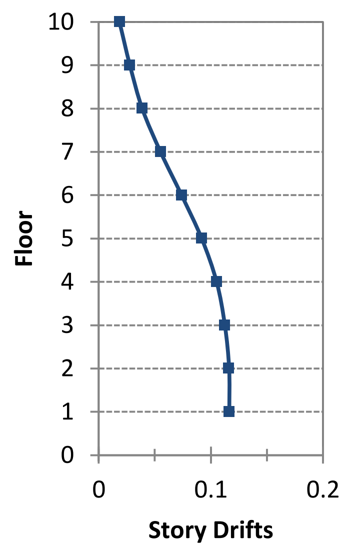
(a-1) Modal Story Displacements for first mode normalized to unity.



(a-2) Modal story drifts corresponding to normalized first mode



(b-1) Snapshot of story displacements for pushover analysis at $\theta = \theta_c$



(b-2) Story Drifts corresponding to story displacements at $\theta = \theta_c$

Figure 11: Simplified methods for estimating Maximum to Average loss ratio for tall structures. (a) Modal Analysis Method; (b) Pushover Analysis Method.

Table 5: Summary of parameters used in loss estimation of 'Redbook Building.'

Parameter	Max model	Average Model	Proposed Model	Remarks
θ_{on} (Rad)	0.005	0.005	0.005	Assigned
θ_c (Rad)	0.053	0.053	0.053	Calibrated
$\tilde{\theta}_{DBE}$ (Rad)	0.016	0.011	-	Regression analysis of IDA results
f_{DBE}	0.002105	0.002105	-	10% in 50 years
k	-3	-3	-3	(Solberg et al. 2008)
b	1	0.87	-	Observed (IDA)
c	1.4	1.4	-	Calibrated
d	-0.467	-0.404	-	Eq (2.2)
$\tilde{L}_{DBE}$	0.187	0.118	-	Eq (2.17)
$\tilde{L}_{on}$	0.0367	0.0367	-	Eq (2.15)
$\tilde{L}_u$	1.3	1.3	-	Assigned
$\tilde{f}_{on}$	0.0687	0.0377	-	Eq (2.13)
$\tilde{f}_u$	3.31×10^{-5}	5.52×10^{-6}	-	Eq (2.12b)
β_{RD}	0.61	0.52	-	From IDA
β_U	0.25	0.25	-	(Solberg et al. 2008)
β_{RC}	0.2	0.2	-	(Solberg et al. 2008)
β_{RS}	0.69	0.61	-	Eq (2.5)
$\beta_{f\|L}$	0.6	0.69	-	(Eq 2.6)
β_{UL}	0.35	0.35	-	(Mander and Sircar 2009)
$\beta_{L\|f}$	0.94	0.81	-	Eq (2.7)
$\bar{L}_{DBE}$	0.2908	0.163	-	Eq (2.16)
$\bar{f}_{on}$	0.0824	0.038	0.038	Eq. (2.10)
$\bar{L}_{on}$	0.052	0.050	0.050	Eq (2.9)
$\bar{L}_u$	1.4	1.4	1.4	Eq (2.11)
$\bar{f}_u$	7.28×10^{-5}	1.03×10^{-5}	7.28×10^{-5}	Eq (2.12a)
L_{rr}	-	-	1	Assigned
f_{rr}	-	-	1.5×10^{-4}	Eq (2.28)
L_{rl}	-	-	0.47	Eq (2.29)
d_m	-	-	-0.467	Eq (2.26)
d_a	-	-	-0.404	Eq (2.27)
EAL/($ million)	**$ 6849**	**$ 3240**	**$ 3342**	Eq(2.25)

2.7 Conclusions

Based on the work presented within this section the following conclusions are drawn:

1. Incremental dynamic analysis (IDA) of suites of earthquakes was used to establish demands for a tall reinforced concrete structure, from which story damage and financial losses were computed directly and then aggregated over the entire structure. SAP2000, a commercial software was used to perform IDA. However it is disappointingly slow with file size exceeding two gigabytes for each analysis.
2. It is observed that moment frame structures have damage concentrated in the lower stories.
3. It is observed that loss estimation after accounting for spatial distribution of damage over the entire structure leads to significant reduction in Expected Annual Loss (EAL)
4. The proposed model which was based on both maximum and average story losses, enables the engineer to discriminate between those cases when the structure has to be completely rebuilt versus those structures were only repairs to damaged components are needed.

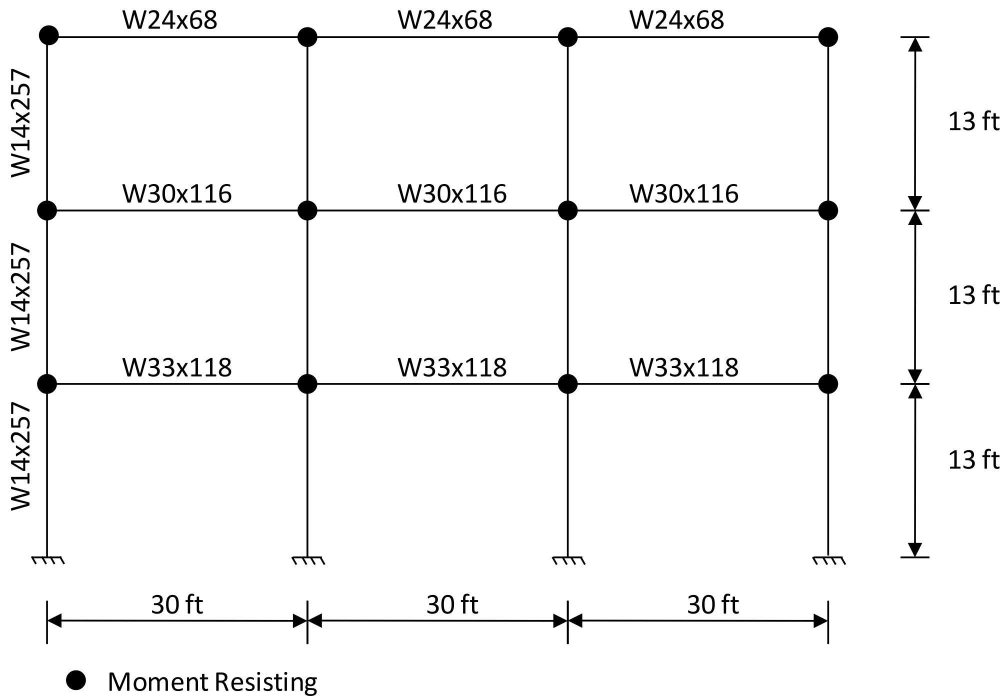
3. LOSS MODELING FOR WSMF STEEL STRUCTURES

3.1 Introduction

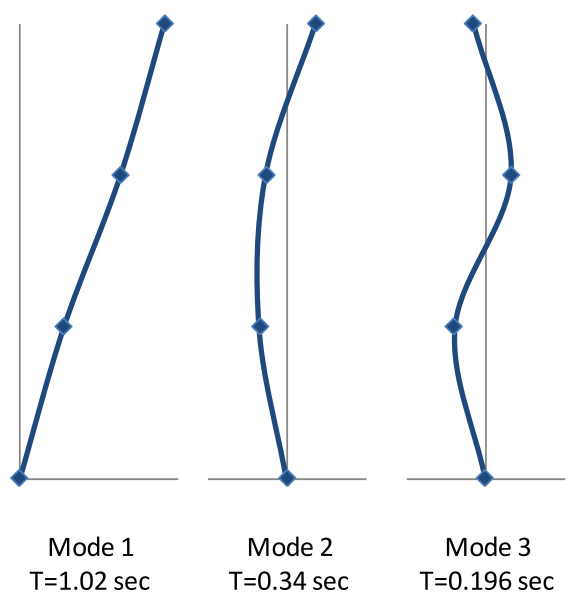
Frame structures tend to exhibit a significant variability in seismically induced damage over the height of the structure. In the previous section, a new loss modeling methodology was proposed which incorporated the effects of spatial distribution of loss over the height of the structure and validated for the Redbook Building (a tall structure). In this section the proposed model is applied and validated for a low-rise, three story WSMF steel structure. The spatial distribution of story losses are analyzed and studied in context with three different beam-to-column connections. For this study the SAC-LA3 structure from SAC Phase II Project was selected for analysis (Somerville et al. 1997). In contrast to the IDA approach used in the previous section, the three unscaled suites of earthquakes (used in the SAC-II project) representing different annual probabilities of occurrence were used to establish demands. Analysis results from a two dimensional model of the structure developed in SAP2000 for three different beam-to-column connections were used to generate a loss model using the proposed loss estimation methodology. Expected Annual Loss was calculated and compared for the different structures.

3.2 Building Design Details

SAC-LA3 structure from SAC Phase II Project was selected for analysis (Somerville et al. 1997). SAC LA3 is a symmetric three bay, three story WSMF steel structure. The design details for exterior moment frame (NS direction) are shown in the Figure 12. The columns of the structure are fixed at base and extend full height of the structure.



(a) Dimensions and member section details for SAC-LA3 structure (NS Elevation)



(b) First three mode shapes and periods for SAC3 structure

Figure 12: Details of SAC-LA3 structure.

Response of the structure was recorded in terms of story drifts and was investigated for three different beam to column connections; (a) the pre-northridge earthquake connections; (b) post-northridge earthquake connections; and (c) high force to volume (HF2V) damper connections.

3.3 Modeling Details

The structure was analyzed by Mander (2008) and the analysis details are presented here for the sake of completeness. SAC LA3 structure is symmetric and hence half of the seismic weight was assigned to the moment resisting frame considered for analysis. The loads were distributed evenly across each floor and lumped at nodes. The seismic weights used in the analysis were 4.7, 4.7 and 5 MN. The structure was modeled with 5% damping for WSMF structure and 8% damping for HF2V structure to account for viscous behavior of HF2V dampers. The fundamental period of vibration of the structure is 1.04 seconds. Mode shapes and periods for first three modes of vibration are mentioned in the Figure 12-b.

The commercially available software, SAP2000, was used to model the moment resisting frame in two dimensions to represent the SAC-LA3 structure. The floor was assumed to be rigid. The dampers were modeled as kinematic springs with a yield force proportional to plastic capacity of the beam. The nonlinear behavior of beams and columns was modeled by introducing a hinge at the face of each beam and column. In case of HF2V connections, the hinge properties were modeled separately for post and pre-northridge connections. To avoid undue damage to the connections, the rotational springs were designed to become active only after the beam reaches the capacity of

$0.8M_p$. Dampers had capacity proportional to story shear and were placed at all joints in each story. The dampers had a design force capacity such that connection has a yield capacity of about $0.8M_p$ of the corresponding beam.

3.4 Earthquakes

Three suites of earthquakes generated for SAC project (Sommerville et al. 1997), each with twenty ground motions have been used for the analysis. The suites represent a 50, 10 and 2 percent probability of occurrence in 50 years in Los Angeles region resulting in the return periods of 72, 475 and 2450 years respectively and are presented in the Tables 6 to 8. The suites of earthquakes used were unscaled and they should be used as suites and not individually or in subsets as representative of the probability levels specified.

3.5 Development of Loss Models

The structure was analyzed for three different types of beam-to-column connections. The frame with post-Northridge connection details is defined here as ductile structure, the frame with pre-Northridge connections is defined as brittle structure and the one with HF2V dampers is defined as HF2V structure.

The response of the structure is recorded in terms of story drifts. The story drifts are used with the proposed loss model presented in the previous section to develop a loss model for the SAC-LA3 structure. A graphical approach was used to define the variables, k, b and d. The dispersion observed in Figure 1-d includes the variability due to loss estimation and β_{RS} calculated using Kennedy's method (Kennedy et al. 1980). Table 10 lists the different parameters used in the analyses of three cases.

Table 6: Details of Los Angeles ground motions having a probability of exceedence of 50% in 50 years.

No	Record	Earthquake Magnitude	Distance (km)	Scale Factor	PGA (cm/sec^2)
1	Coyote Lake, 1979	5.7	8.8	2.28	578.34
2	Coyote Lake, 1979	5.7	8.8	2.28	326.81
3	Imperial Valley, 1979	6.5	1.2	0.4	140.67
4	Imperial Valley, 1979	6.5	1.2	0.4	109.45
5	Kern, 1952	7.7	107	2.92	141.49
6	Kern, 1952	7.7	107	2.92	156.02
7	Landers, 1992	7.3	64	2.63	331.22
8	Landers, 1992	7.3	64	2.63	301.74
9	Morgan Hill, 1984	6.2	15	2.35	312.41
10	Morgan Hill, 1984	6.2	15	2.35	535.88
11	Parkfield, 1966, Cholame 5W	6.1	3.7	1.81	765.65
12	Parkfield, 1966, Cholame 5W	6.1	3.7	1.81	619.36
13	Parkfield, 1966, Cholame 8W	6.1	8	2.92	680.01
14	Parkfield, 1966, Cholame 8W	6.1	8	2.92	775.05
15	North Palm Springs, 1986	6	9.6	2.75	507.58
16	North Palm Springs, 1986	6	9.6	2.75	371.66
17	San Fernando, 1971	6.5	1	1.3	248.14
18	San Fernando, 1971	6.5	1	1.3	226.54
19	Whittier, 1987	6	17	3.62	753.70
20	Whittier, 1987	6	17	3.62	469.07

Table 7: Details of Los Angeles ground motions having a probability of exceedence of 10% in 50 years.

No	Record	Earthquake Magnitude	Distance (km)	Scale Factor	PGA (cm/sec^2)
1	Imperial Valley, 1940, El Centro	6.9	10	2.01	452.03
2	Imperial Valley, 1940, El Centro	6.9	10	2.01	662.88
3	Imperial Valley, 1979, Array #05	6.5	4.1	1.01	386.04
4	Imperial Valley, 1979, Array #05	6.5	4.1	1.01	478.65
5	Imperial Valley, 1979, Array #06	6.5	1.2	0.84	295.69
6	Imperial Valley, 1979, Array #06	6.5	1.2	0.84	230.08
7	Landers, 1992, Barstow	7.3	36	3.2	412.98
8	Landers, 1992, Barstow	7.3	36	3.2	417.49
9	Landers, 1992, Yermo	7.3	25	2.17	509.70
10	Landers, 1992, Yermo	7.3	25	2.17	353.35
11	Loma Prieta, 1989, Gilroy	7	12	1.79	652.49
12	Loma Prieta, 1989, Gilroy	7	12	1.79	950.93
13	Northridge, 1994, Newhall	6.7	6.7	1.03	664.93
14	Northridge, 1994, Newhall	6.7	6.7	1.03	644.49
15	Northridge, 1994, Rinaldi RS	6.7	7.5	0.79	523.30
16	Northridge, 1994, Rinaldi RS	6.7	7.5	0.79	568.58
17	Northridge, 1994, Sylmar	6.7	6.4	0.99	558.43
18	Northridge, 1994, Sylmar	6.7	6.4	0.99	801.44
19	North Palm Springs, 1986	6	6.7	2.97	999.43
20	North Palm Springs, 1986	6	6.7	2.97	967.61

Table 8: Details of Los Angeles ground motions having a probability of exceedence of 2% in 50 years.

No	Record	Earthquake Magnitude	Distance (km)	Scale Factor	PGA (cm/sec^2)
1	1995 Kobe	6.9	3.4	1.15	1258.00
2	1995 Kobe	6.9	3.4	1.15	902.75
3	1989 Loma Prieta	7	3.5	0.82	409.95
4	1989 Loma Prieta	7	3.5	0.82	463.76
5	1994 Northridge	6.7	7.5	1.29	851.62
6	1994 Northridge	6.7	7.5	1.29	925.29
7	1994 Northridge	6.7	6.4	1.61	908.70
8	1994 Northridge	6.7	6.4	1.61	1304.10
9	1974 Tabas	7.4	1.2	1.08	793.45
10	1974 Tabas	7.4	1.2	1.08	972.58
11	Elysian Park (simulated)	7.1	17.5	1.43	1271.20
12	Elysian Park (simulated)	7.1	17.5	1.43	1163.50
13	Elysian Park (simulated)	7.1	10.7	0.97	767.26
14	Elysian Park (simulated)	7.1	10.7	0.97	667.59
15	Elysian Park (simulated)	7.1	11.2	1.1	973.16
16	Elysian Park (simulated)	7.1	11.2	1.1	1079.30
17	Palos Verdes (simulated)	7.1	1.5	0.9	697.84
18	Palos Verdes (simulated)	7.1	1.5	0.9	761.31
19	Palos Verdes (simulated)	7.1	1.5	0.88	490.58
20	Palos Verdes (simulated)	7.1	1.5	0.88	613.28

Variables like $\theta_{on,}$ θ_c and c were obtained from calibration of the loss model. The calibration results are listed in Table 9 and are shown in Figure 13. The loss model can be defined by following equation

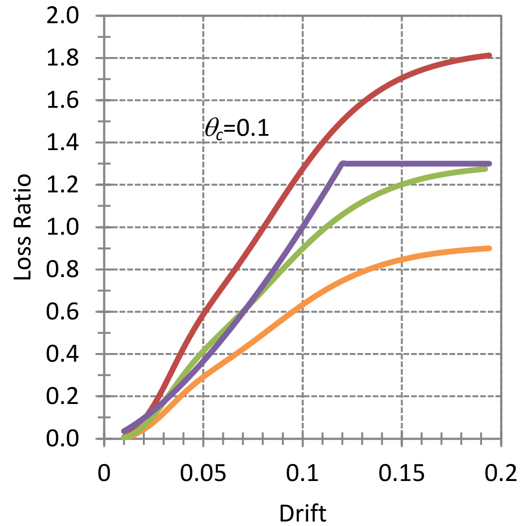
$$\frac{L}{L_c} = \left|\frac{\theta}{\theta_c}\right|^c \quad L_{on} \le L < L_u \tag{3.1}$$

in which, L_{on} is loss at onset of damage.

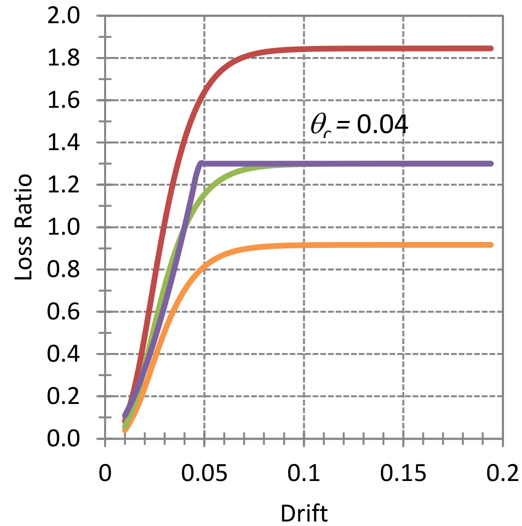
Table 9: Parameters used for calibration of loss model.

Parameter	Ductile Structure	Brittle Structure	HF2V Structure
θ (DS2)	0.005	0.005	0.03
θ (DS3)	0.02	0.02	0.04
θ (DS4)	0.03	0.03	0.06
θ (DS5)	0.08	0.04	0.08
L (DS2)	0.065	0.1	0.0001
L (DS3)	0.15	0.3	0.15
L (DS4)	0.65	0.7	0.65
L (DS5)	1.3	1.3	1.3

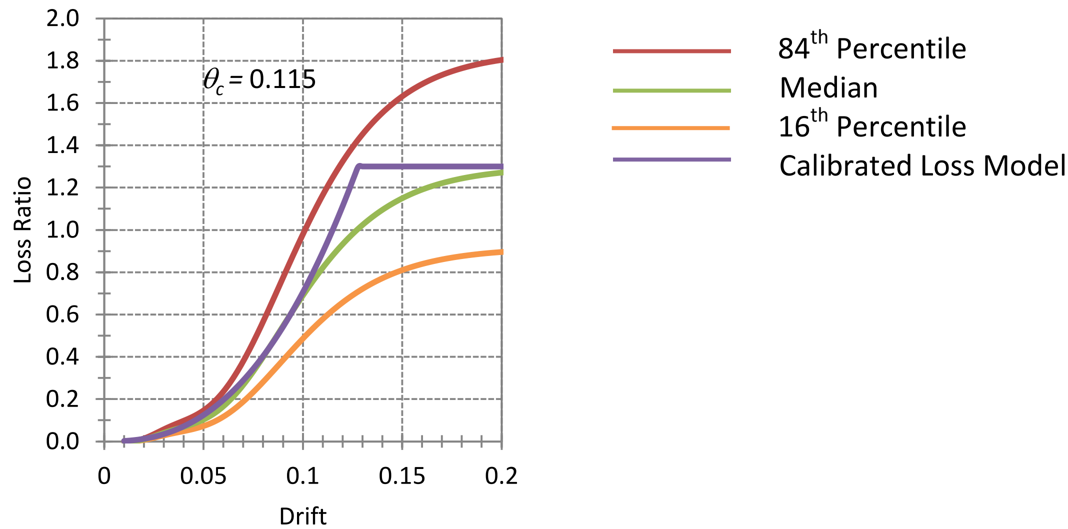
DS = Damage State



(a) Ductile Structure (c =1.45)



(b) Brittle Structure (c = 1.6)



(c) HF2V Damper structure (c =2.5)

Figure 13: Calibration results for the three loss models.

3.6 Results and Discussion

The proposed loss model was used to analyze the steel structure. Table 10 tabulates the analysis of the EAL for the maximum and average losses, and the composite results are given in the Table 11. Scenario results for DBE and MCE are tabulated in Table 12. Figures 14-19 present the results for the two loss models for three different structural connection types. A composite graph of results is presented in Figure 20.

As the SAC-LA3 building is a short, low-rise, relatively long-span structure it may be observed that there was not much variability in the damage over the height of the structure and hence, the two loss models Maximum and Average provide somewhat similar results. This is evident when we consider scenario losses. For the ductile structure the loss ratio for Maximum Loss Model at DBE is 0.27 whereas for the proposed model it reduces slightly to 0.21. In case of brittle structure the losses are higher with loss ratio for Maximum and proposed loss model equal to 0.31 and 0.25 at DBE respectively. The damped structure with HF2V dampers does not show any damage for DBE.

For ductile structure, the return period of the structure is some 19000 years (or the probability of collapse is 0.26% in 50 years) while for the same structure with brittle welded details the return period is about 2400 years (the probability of collapse being 2.1% in 50 years) The alternate damage avoidance design (DAD) construction with bolted simple connections plus HF2V dampers yield a return period of about 8600 years. This is less than the ductile structure which is understandable since the same structural elements are used, but the connection strength is only 80 percent of the full strength welded moment frame counterpart. Moreover, the response to the classic design earthquake levels of DBE and MCE is quite superior with less than one-half the amount of damage expected for MCE. However, in spite of similar results for loss ratios of the proposed and Maximum Loss Model, the proposed model has lower EAL. For a ductile structure EAL of the proposed model is $2579 significantly lower than the EAL for Maximum Loss Model which is $3595.

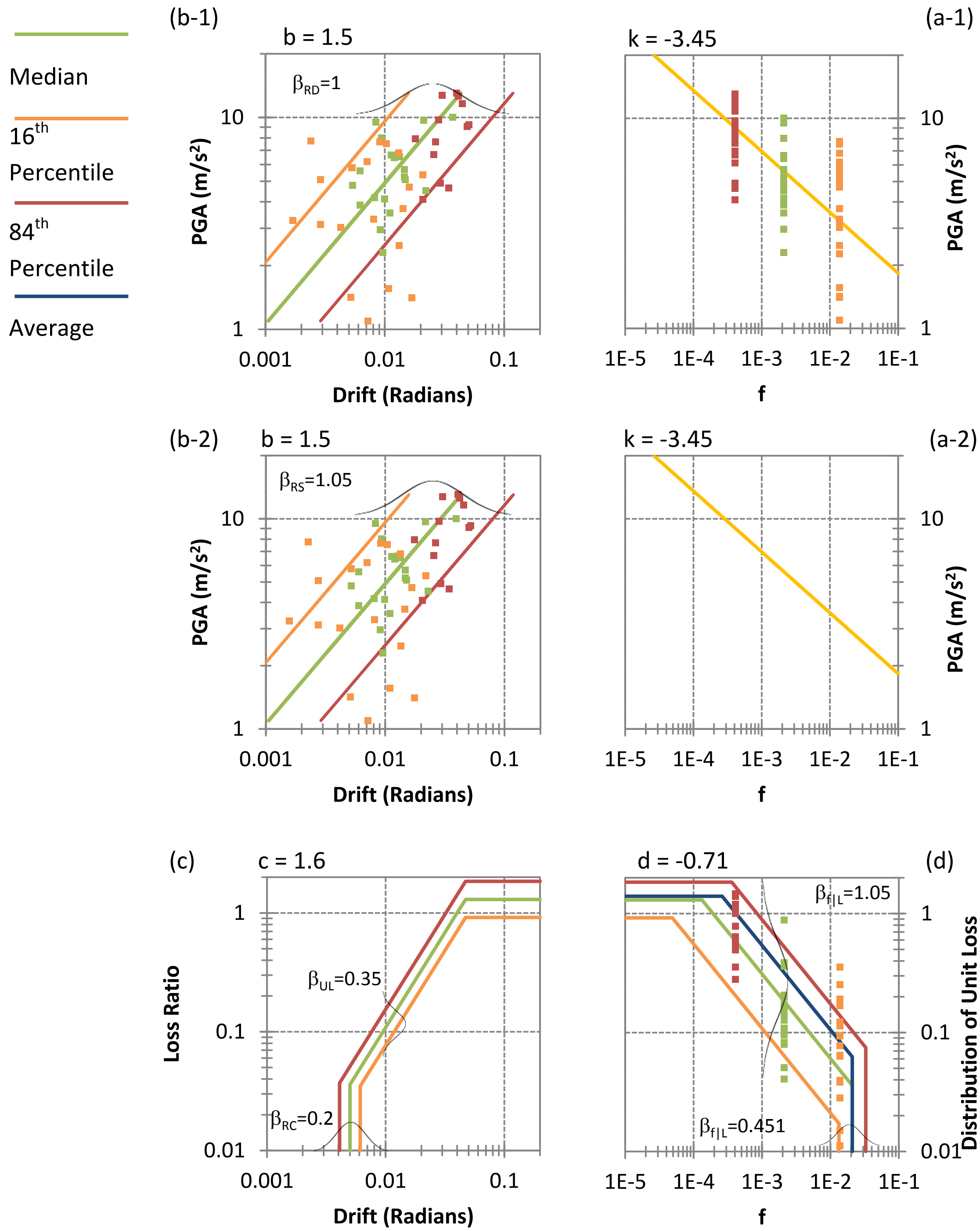


Figure 14: SAC-LA3 Brittle structure (Maximum Loss Model). (a) Hazard recurrence relation; (b-1) drifts obtained from structural analysis; (b-2) drifts with combined uncertainty of modeling and capacity of the structure; (c) loss ratio obtained from story drifts; (d) loss model after accounting for different variabilities in the loss model.

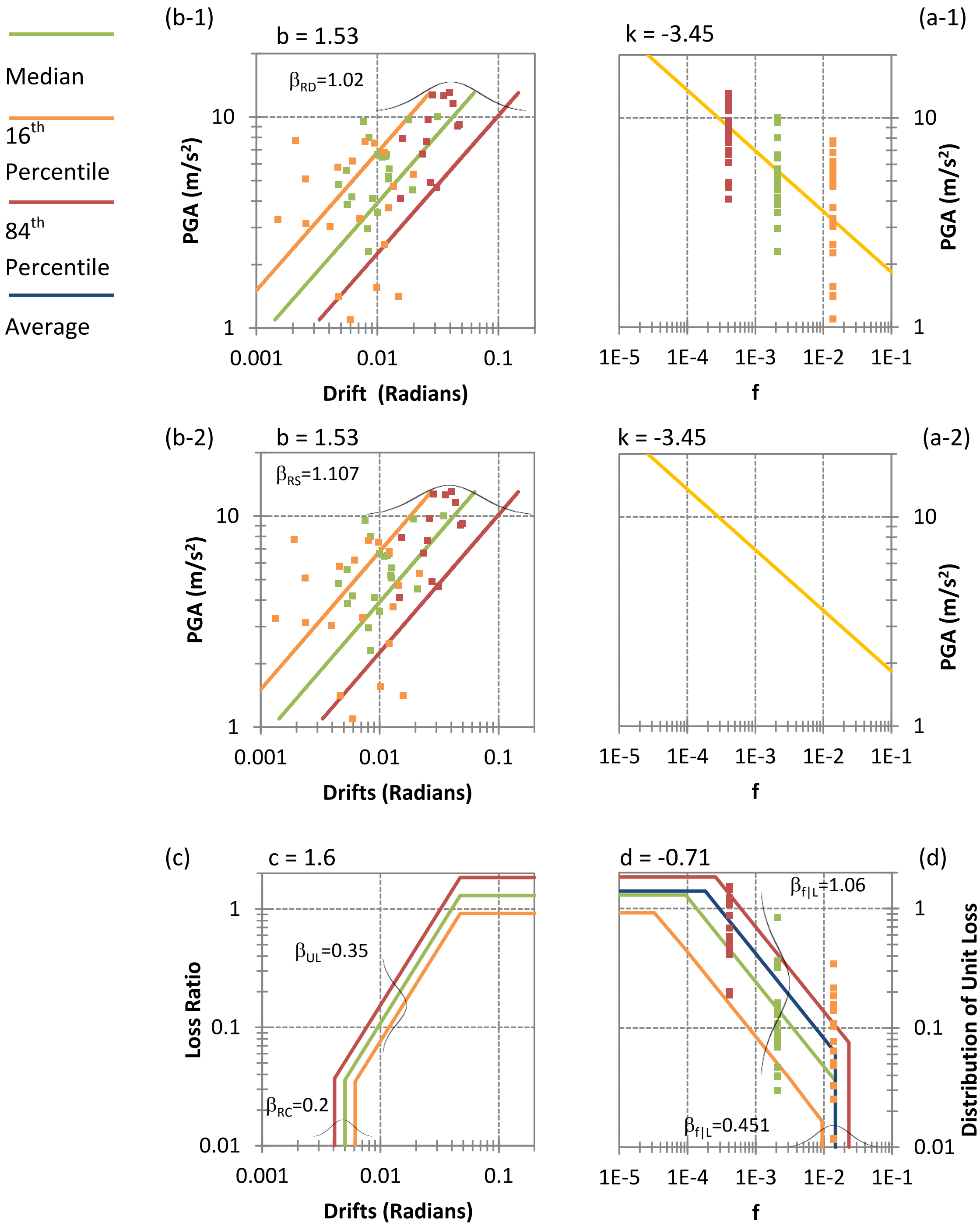


Figure 15: SAC-LA3 Brittle structure (Average Loss Model). (a) Hazard recurrence relation; (b-1) drifts obtained from structural analysis; (b-2) drifts with combined uncertainty of modeling and capacity of the structure; (c) loss ratio obtained from story drifts; (d) loss model after accounting for different variabilities in the loss model.

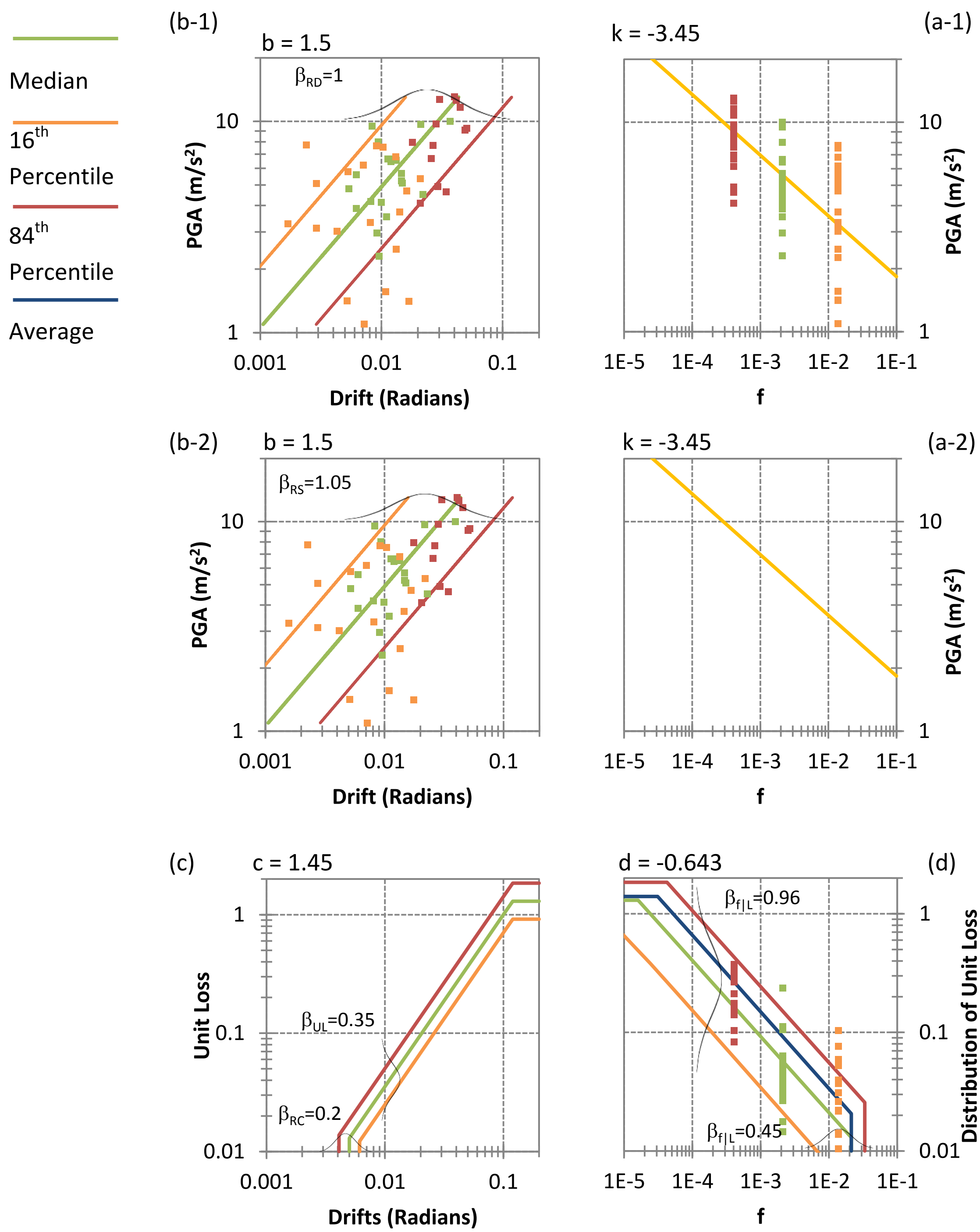


Figure 16: SAC-LA3 Ductile structure (Maximum Loss Model). (a) Hazard recurrence relation; (b-1) drifts obtained from structural analysis; (b-2) drifts with combined uncertainty of modeling and capacity of the structure; (c) loss ratio obtained from story drifts; (d) loss model after accounting for different variabilities in the loss model.

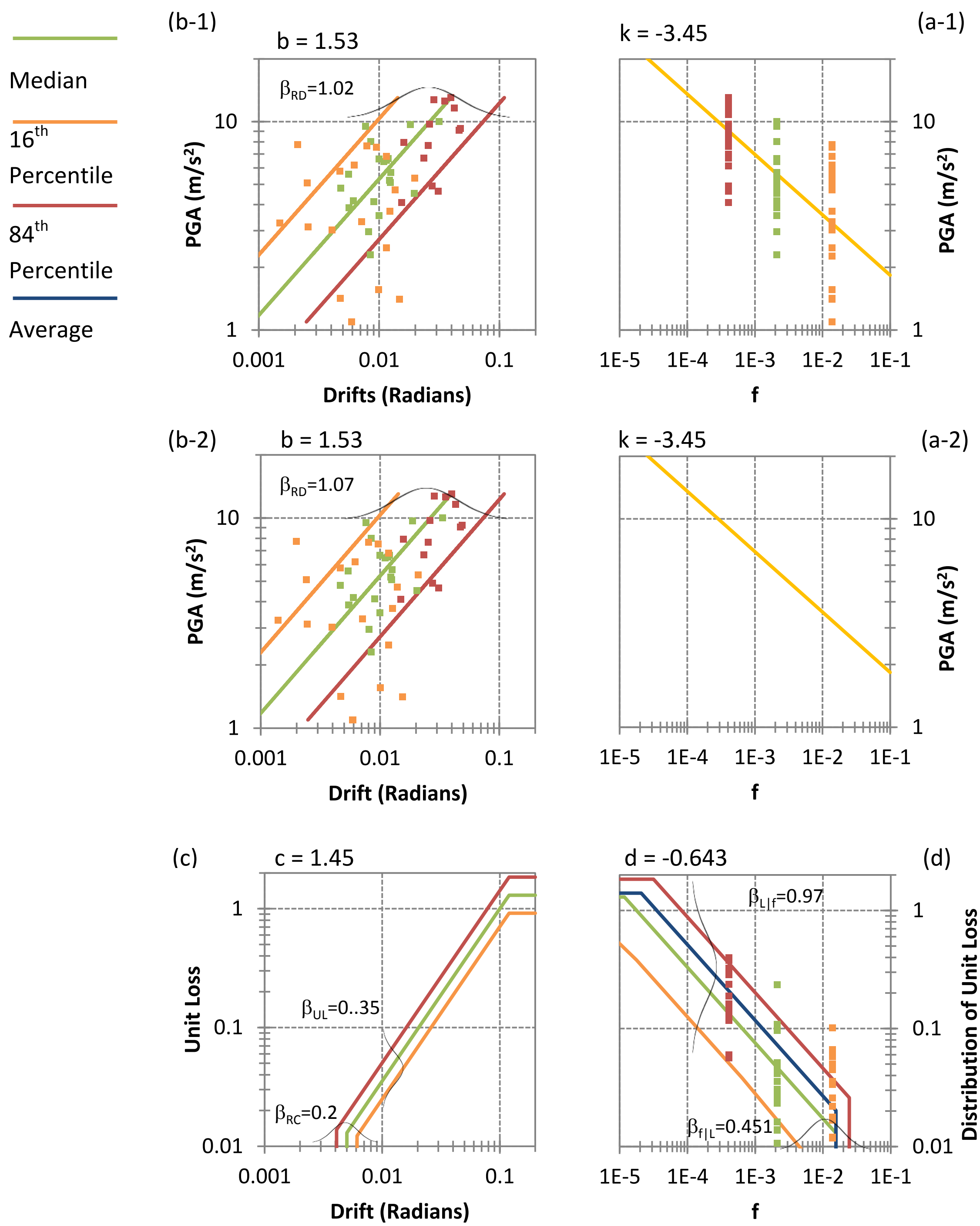


Figure 17: SAC-LA3 Ductile structure (Average Loss Model). (a) Hazard recurrence relation; (b-1) drifts obtained from structural analysis; (b-2) drifts with combined uncertainty of modeling and capacity of the structure; (c) loss ratio obtained from story drifts; (d) loss model after accounting for different variabilities in the loss model.

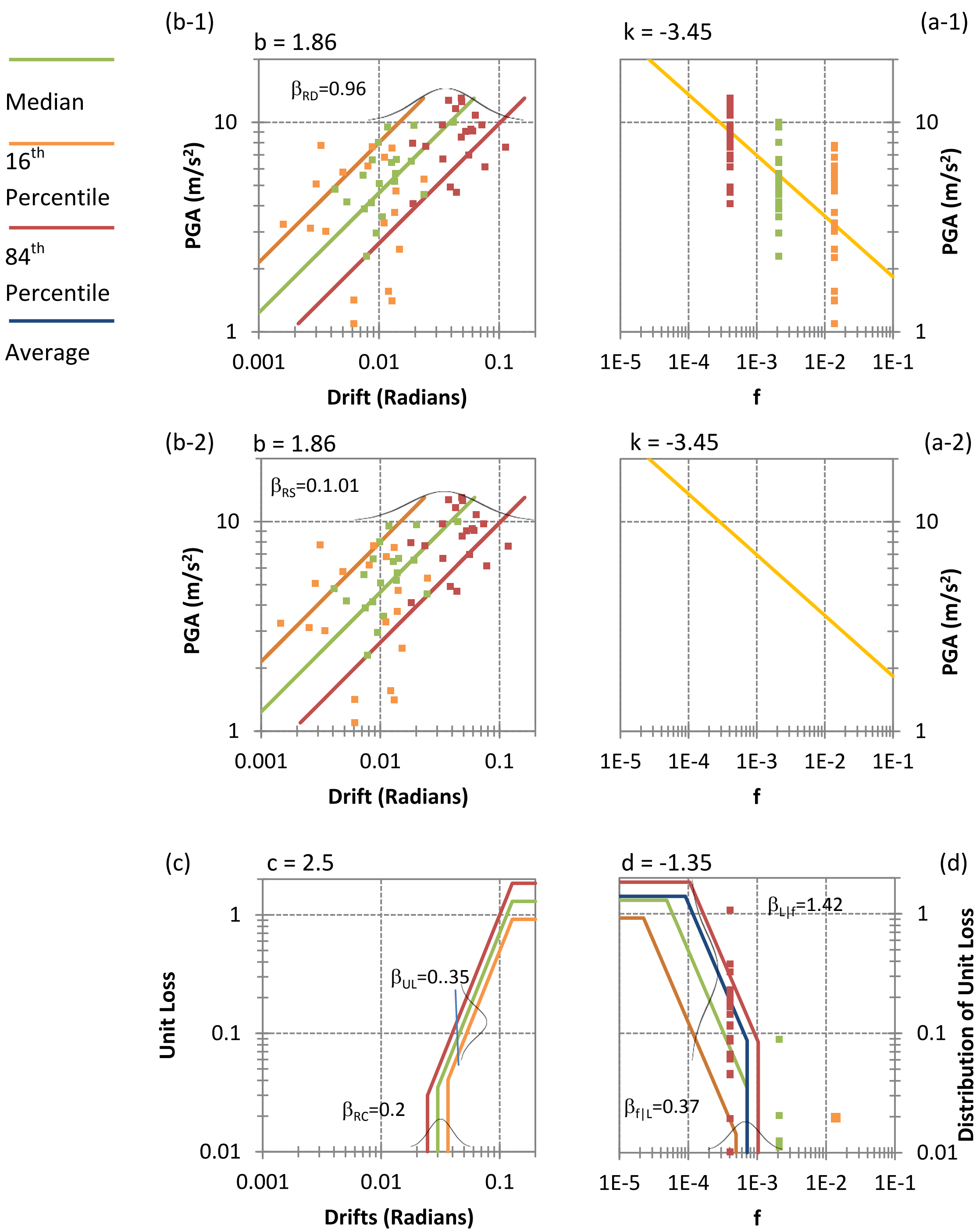


Figure 18: SAC-LA3 HF2V structure (Maximum Loss Model). (a) Hazard recurrence relation; (b-1) drifts obtained from structural analysis; (b-2) drifts with combined uncertainty of modeling and capacity of the structure; (c) loss ratio obtained from story drifts; (d) loss model after accounting for different variabilities in the loss model.

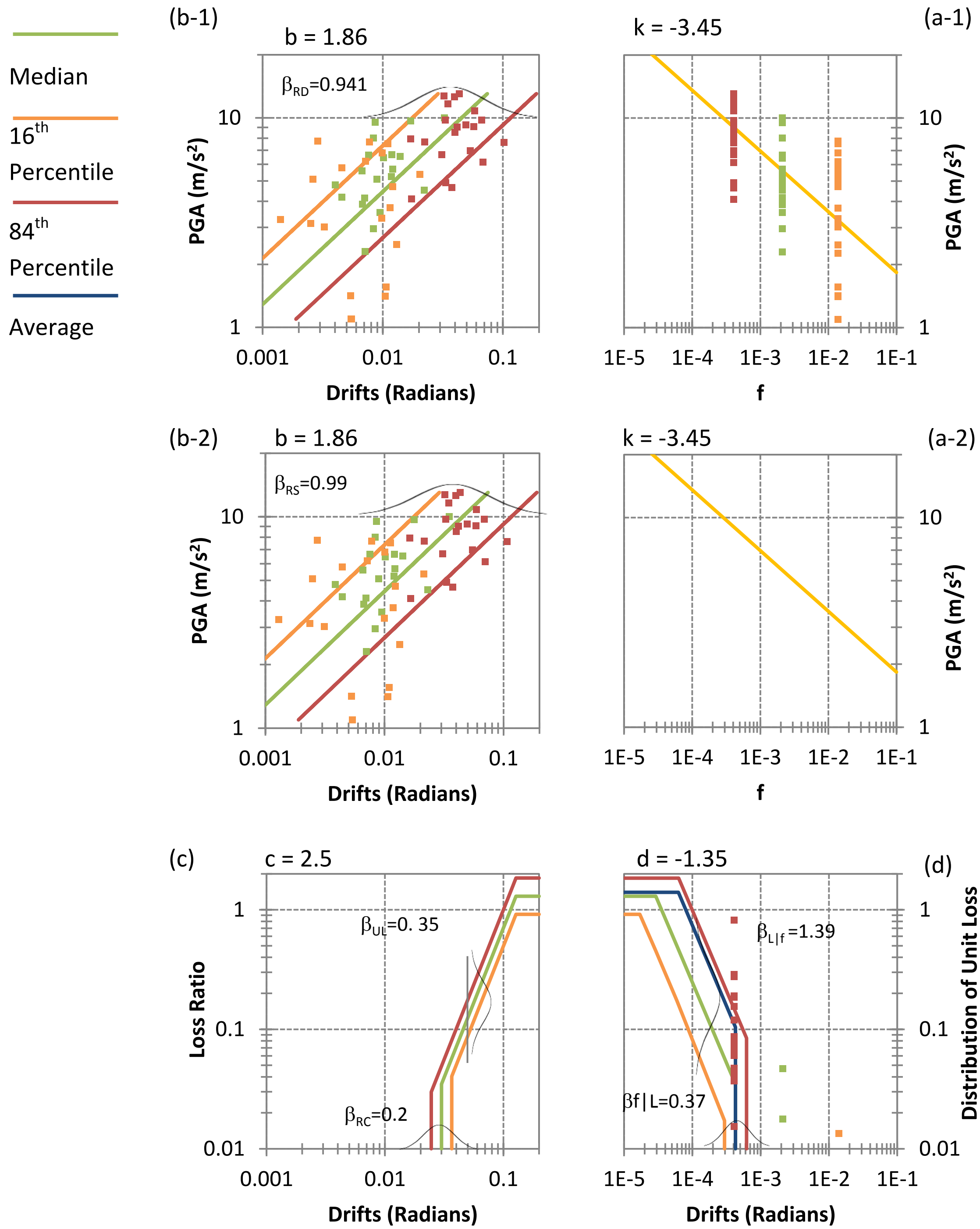


Figure 19: SAC-LA3 HF2V structure (Average Loss Model). (a) Hazard recurrence relation; (b-1) drifts obtained from structural analysis; (b-2) drifts with combined uncertainty of modeling and capacity of the structure; (c) loss ratio obtained from story drifts; (d) loss model after accounting for different variabilities in loss model.

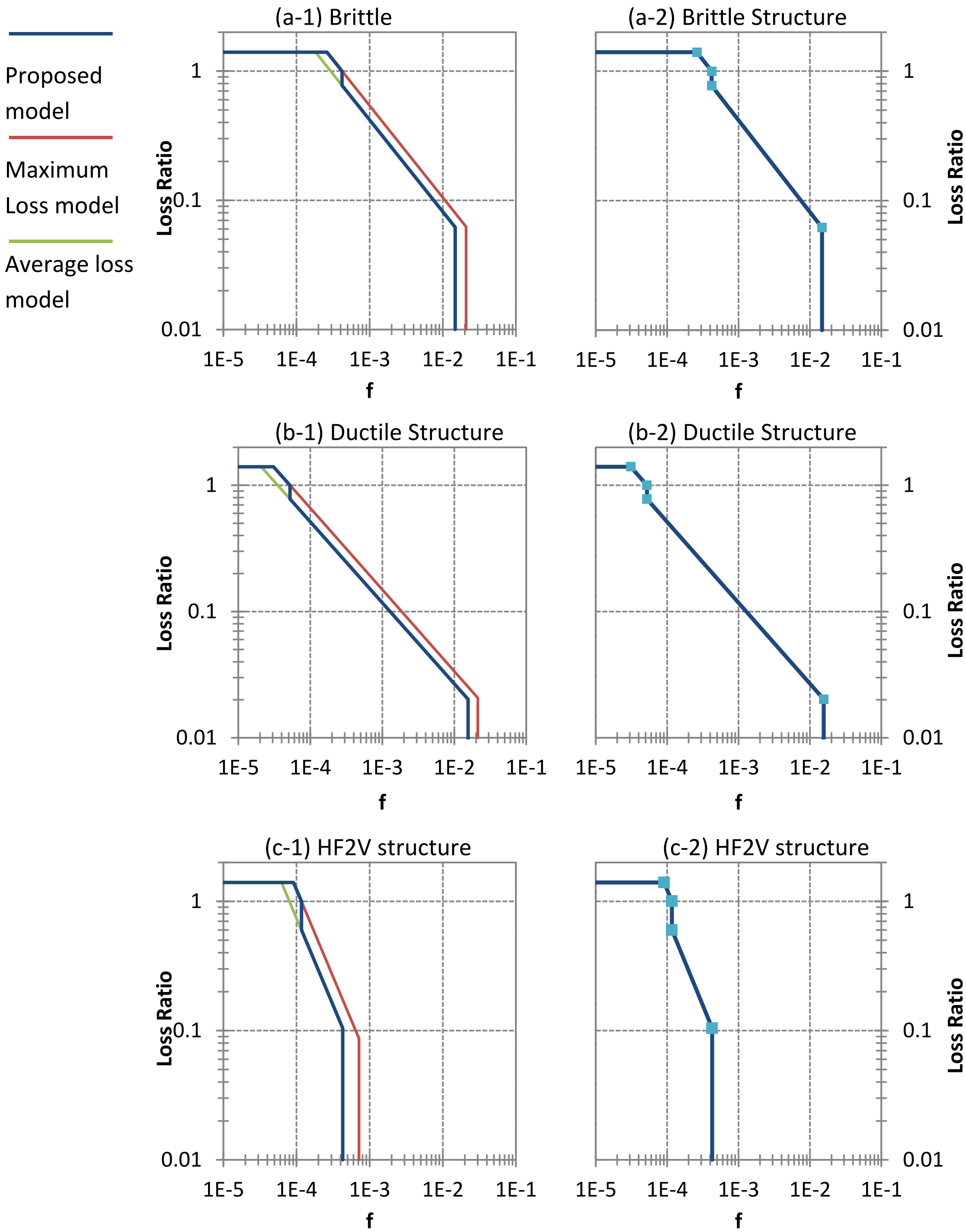


Figure 20: Construction of proposed model from the Average and Maximum Loss Model; (a) brittle structure (b) ductile structure (c) HF2V structure.

Table 10: Summary of parameters used in loss estimation of SAC LA3 building.

Parameter	Brittle Structure		Ductile Structure		HF2V Structure	
	Maximum	**Average**	**Maximum**	**Average**	**Maximum**	**Average**
b	1.51	1.53	1.53	1.53	1.86	1.86
c	1.45	1.45	1.6	1.6	2.5	2.5
d	-0.643	-0.643	-0.71	-0.71	-1.35	-1.35
k	-3.45	-3.45	-3.45	-3.45	-3.45	-3.45
θ_{on} *(Rad)*	0.005	0.005	0.005	0.005	0.03	0.03
θ_c *(Rad)*	0.04	0.04	0.1	0.1	0.115	0.115
$\tilde{L}_{on}$	0.035	0.04	0.012	0.01	0.034	0.03
$\tilde{L}_u$	1.3	1.3	1.3	1.3	1.3	1.3
$\tilde{f}_{on}$	$2.09\text{x}10^{-2}$	$1.48\text{x}10^{-2}$	$2.13\text{x}10^{-2}$	$1.55\text{x}10^{-2}$	$7.15\text{x}10^{-4}$	$4.28\text{x}10^{-4}$
$\tilde{f}_u$	$1.33\text{x}10^{-4}$	$9.41\text{x}10^{-5}$	$1.61\text{x}10^{-5}$	$1.81\text{x}10^{-5}$	$4.87\text{x}10^{-5}$	$2.91\text{x}10^{-5}$
β_{RD}	1	1.02	1	1.02	0.96	0.94
β_U	0.25	0.25	0.25	0.25	0.25	0.25
β_{RC}	0.2	0.2	0.2	0.2	0.2	0.2
β_{RS}	1.05	1.1	1.05	1.07	1.01	0.99
$\beta_{f\|L}$	0.45	0.45	0.45	0.45	0.37	0.37
β_{UL}	0.35	0.35	0.35	0.35	0.35	0.35
$\beta_{L\|f}$	1.05	1.06	0.96	0.97	1.41	1.41
$\bar{f}_{on}$	$2.09\text{x}10^{-2}$	$1.48\text{x}10^{-2}$	$2.13\text{x}10^{-3}$	$1.55\text{x}10^{-2}$	$7.15\text{x}10^{-4}$	$4.28\text{x}10^{-4}$
$\bar{L}_{on}$	0.06	0.06	0.02	0.02	0.09	0.1
$\bar{L}_u$	1.4	1.4	1.4	1.4	1.4	1.4
$\bar{f}_u$	$2.6\text{x}10^{-4}$	$1\text{x}10^{-5}$	$3.09\text{x}10^{-5}$	$2.09\text{x}10^{-5}$	$9.1\text{x}10^{-5}$	$6.23\text{x}10^{-5}$
EAL/$ Million	**$ 3595**	**$2523**	**$1146**	**$822**	**$315**	**$210**

Table 11: Implementation of proposed composite loss model based on the results presented in the Table 10.

Parameter	Brittle Structure Proposed Loss Model	Ductile Structure Proposed Loss Model	HF2V Structure Proposed Loss Model
$\overline{f}_{on}$	$1.48\text{x}10^{-2}$	$1.55\text{x}10^{-3}$	$4.25\text{x}10^{-4}$
$\overline{L}_{on}$	0.06	0.02	0.7
$\overline{f}_{u}$	$2.61\text{x}10^{-4}$	$3.09\text{x}10^{-5}$	$9.1\text{x}10^{-4}$
$\overline{L}_{u}$	1.4	1.4	1.4
L_{rr}	1	1	1
f_{rr}	$4.19\text{x}10^{-4}$	$5.22\text{x}10^{-5}$	$1.17\text{x}10^{-4}$
L_{rl}	0.78	0.78	0.6
EAL / $ Million	**$ 2578**	**$ 829**	**$231**

Table 12: Scenario results for the SAC-LA3 structure.

Parameter	Brittle Structure		Ductile Structure		HF2V Structure	
	Maximum	**Proposed**	**Maximum**	**Proposed**	**Maximum**	**Proposed**
L_{DBE}	0.31	0.25	0.09	0.07	0.02	0.01
L_{MCE}	1.03	0.80	0.27	0.21	0.19	0.11
Return Period at Collapse	--	2384 yrs	--	19140 yrs	--	8560 yrs
EAL/$ Million	**$ 3595**	**$2578**	**$1146**	**$829**	**$315**	**$231**

3.7 Conclusions

Based on the results of this section the following conclusions are drawn:

1 Suites of earthquakes with different probabilities of occurrence can be used to establish demands from which story damage and financial losses can be computed directly.

2 It was observed that for the three story SAC-LA3 steel frame building investigated herein there was a variation of some 30% less loss when considering spatial distribution irrespective of the connection type.

3 The model helps in discriminating between the damaged structures which can be repaired to structures which need to be completely rebuilt.

4. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

4.1 Summary

Tall building frames that respond to large seismic ground motions may have significant spatial variability of damage over their height, often with the concentration of that damage in the lower stories. In spite of this spatial variability in the damage, the existing damage models tend to focus on calculating the maximum story drift and then assuming the same drift over the entire height. A new approach was presented towards loss analysis after incorporating the effects of spatial distribution of earthquake induced damage in frame structures. Incremental dynamic analysis using suites of earthquakes was used to establish demands from which story damage and financial losses were computed directly and aggregated for the entire structure. Different methods were explored to aggregate losses arising from individual story drifts. Based on comparison between these distinct approaches, a simple algorithm was recommended.

4.2 Conclusions

Based on the study presented in this thesis, the following conclusions can be drawn:

1. Suites of earthquakes and incremental dynamic analysis along with a commercial software SAP2000 was used to establish demands from which story damage and financial losses were computed directly and then aggregated over the entire structure.
2. It is observed that for moment frame structures damage in concentrated in the lower stories. Severe damage in a lower story may be such that the entire building requires reconstruction even though the upper stories could be virtually

unscathed. There was about 50% reduction in damage for the 10-story concrete building and 30% reduction for the 3-story steel structure due to spatial variability with respect to the maximum loss.

3. The loss model which was based on assessing both maximum story and average story losses, enables the engineer to discriminate between those cases when the structure has to be completely rebuilt versus those structures where only repairs to damaged components are needed.

4.3 Recommendations

The following areas may be considered suitable for future research:

(a) Exploring the response of three dimensional model of a reinforced concrete structure due to a bidirectional earthquake effects. This may lead to some interesting outcomes especially for structures with mass eccentricity.

(b) While the study presented herein was on moment frame building, it would be interesting to investigate the outcomes of frames versus concrete shear wall structures and also moment frame versus braced frame steel structures.

(c) Numerical experiments with the use of different intensity measures like peak ground acceleration and peak ground velocity to capture the acceleration and velocity sensitive damage in the structure and examine its effect specifically on non-structural components and contents.

(d) As might be expected the two buildings investigated herein tended to show there was a height effect in concentration of damage in the lower stories. Thus an investigation of the damage pattern of much taller buildings would be of interest.

For example, when buildings exceed some 20 stories seismic demands, and thus damage patterns can be markedly affected by higher mode effects.

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