Seismic Vulnerability Assessment and Retrofit Optimization of Non-Code Conforming Buildings with Consideration of Mainshock-Aftershock Earthquake

by

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Abstract

Many seismically active locations in Canada and around the world have structures built before modern seismic design practices were established. More recently, the effects of aftershocks have been included as an important consideration when assessing the seismic risk of non-code conforming structures. Often, the large stock of at-risk structures limits the available resources to mitigate seismic damage. In this work, modern simulation software was used to perform a seismic hazard analysis of a non-code conforming reinforced-concrete structure including the effects of mainshock-aftershock sequences. Structure performance was measured with both drift and damage accumulation methods. Additionally, optimization methods were integrated with the structural analysis to reduce the cost and improve the performance of seismic mitigation retrofit schemes. Modern parallel computing techniques were implemented to reduce the overall computation time, further improving the usability of the process. The resulting retrofit location schemes were presented and compared.
Preface

A version of Chapter 3 and relevant portions of Chapter 2 have been previously published. Duerr, K. and Tesfamariam. S. (2012) Damage accumulation in reinforced concrete structures due to mainshock and aftershock earthquakes. Proceedings of 3rd International Structural Specialty Conference. Canadian Society of Civil Engineers. Session: STRUCTURAL: Progressive Collapse of Buildings. I had performed all of the simulations and calculations, and written the manuscript.

Portions of Chapter 4 were previously presented. Duerr, K. and Tesfamariam. S. (2012) Comparison of optimization methods for retrofit placement in a non-code conforming reinforced-concrete structure. Proceedings of the 15th World Conference on Earthquake Engineering. Lisbon, Portugal. I had performed all of the simulations and calculations, and written the manuscript.
# Table of Contents

Abstract .................................................................................................................................... ii  
Preface ..................................................................................................................................... iii  
Table of Contents ................................................................................................................... iv  
List of Tables ........................................................................................................................ vi  
List of Figures ........................................................................................................................ vii  
List of Symbols ....................................................................................................................... ix  
List of Abbreviations ............................................................................................................. xi  
Acknowledgements ............................................................................................................... xii  
Dedication ............................................................................................................................. xiii  

1 Chapter: Introduction ....................................................................................................... 1  
  1.1 Background ....................................................................................................................... 1  
  1.2 Objective ............................................................................................................................ 4  
  1.3 Organization of the Thesis ............................................................................................... 5  

2 Chapter: Background and Review of Mainshock-Aftershock Earthquake ............... 7  
  2.1 Ground Motion Parameters .............................................................................................. 7  
  2.2 Ground Motion Record Selection ..................................................................................... 8  
  2.3 Ground Motion Scaling ................................................................................................... 9  
  2.4 Aftershocks ..................................................................................................................... 10  

3 Chapter: Damage Accumulation Due to Mainshock-Aftershock Earthquake Sequence .............................................................................................................................. 16  
  3.1 Seismic Structural Performance ....................................................................................... 16  
      3.1.1 Global Performance Measures ............................................................................... 17  
      3.1.2 Local Performance Measures ............................................................................... 18  
      3.1.2.1 Park and Ang Damage Index ........................................................................ 19  
      3.1.2.2 Kratzig Damage Index ................................................................................... 20  
      3.1.3 Application of Selected Damage Indices .............................................................. 22  
  3.2 Case Study: Van Nuys Hotel .......................................................................................... 23
List of Tables

Table 3.1: Inter-storey drift limits and corresponding performance levels (Adapted from FEMA 356) .......................................................... 18
Table 3.2: Comparison of DI value, MISDR, and damage level .................................................. 19
Table 3.3: Summary of damage indices and damage .................................................................. 19
Table 3.4: Summary of selected earthquake records .................................................................. 26
Table 4.1: Summary of optimization applications in structural analysis ........................................ 42
Table 4.2: Building loss estimates ............................................................................................... 54
Table A.1: Column reinforcement Schedule (Adapted from Krawinkler, 2005) ....................... 71
Table A.2: Spandrel beam reinforcement schedule, floors 3 through 7 (Adapted from Krawinkler, 2005) ........................................................................................................ 72
Table A.3: Roof and second-floor spandrel beam reinforcement schedule (Adapted from Krawinkler, 2005) ........................................................................................................ 72
List of Figures

Figure 1.1: Haiti Earthquake by Marco Dormino / The United Nations (http://en.wikipedia.org/wiki/File:Haiti_Earthquake_building_damage.jpg) CC BY 2.0 ................................................................................................................................. 1

Figure 1.2: Repairs in Tower O’Higgins, Concepcion by Ex-BGDA- (http://en.wikipedia.org/wiki/File:Torre_O’Higgins_conce.jpg) CC BY- SA 3.0 ......................................................................................................................... 2

Figure 1.3: Retrofit optimization flow chart ........................................................................ 6

Figure 2.1: Example UHS data for a selection of Canadian cities ........................................ 10

Figure 3.1: Hysteretic energy cycle nomenclature .............................................................. 21

Figure 3.2: South frame elevation of Van Nuys hotel ......................................................... 23

Figure 3.3: Schematic of the OpenSees "BeamWithHinges" Element and Fibre Model .............................................................................................................................. 25

Figure 3.4: 10%/50 years for the mainshock and 15%/50 year aftershock seismic hazard .................................................................................................................. 27

Figure 3.5: Example time history of a second storey drift ratio for a structure subject to mainshock and aftershock ................................................................. 29

Figure 3.6: Mainshock and mainshock-aftershock results for a) Residual ISD, b) Maximum ISD with performance levels shown .................................................. 30

Figure 3.7: Hysteretic curves for the exterior columns of the first 6 storeys (C1-C6) during a mainshock-aftershock earthquake sequence. ........................................ 31

Figure 3.8: Damage index results of a mainshock-aftershock earthquake sequence for single columns. The ISD results from Figure 3.5 are included for comparison ........................................................................................................ 32

Figure 3.9: Example progression of damage index values for a) Park and Ang and b) Kratzig damage indices ....................................................................................... 33

Figure 3.10: Cumulative distribution for the aggregated damage index values with data points and fitted log normal curves ................................................................. 34

Figure 4.1: Maximum ISD for a) the base structure results, b) GA optimization results, and c) Greedy optimization results. ........................................................................ 48
Figure 4.2: Van Nuys hotel schematic with Greedy Algorithm (Solid Lines) and Genetic Algorithm (Dashed Lines) optimized retrofit schemes with proportional objective function ........................................................................ 49

Figure 4.3: (a) Genetic Algorithm and (b) Greedy Algorithm improvement at each iteration ............................................................................................................. 49

Figure 4.4: Performance of retrofit schemes with proportional objective function values ................................................................................................................ 51

Figure 4.5: (a) Genetic Algorithm and (b) Greedy Algorithm improvement at each iteration for the cost-estimate objective function ............................................. 55

Figure 4.6: Van Nuys hotel schematic of genetic algorithm optimized retrofit schemes from cost-based objective function ............................................................ 56

Figure 4.7: Van Nuys hotel schematic of greedy algorithm optimized retrofit schemes from cost-based objective function ......................................................... 57

Figure 4.8: Performance of retrofit schemes with cost-estimate objective function values .................................................................................................................. 58
### List of Symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a, b, c$</td>
<td>site specific ground motion parameter</td>
</tr>
<tr>
<td>$b$</td>
<td>damage index scaling parameter</td>
</tr>
<tr>
<td>$C_R$</td>
<td>cost due to applied retrofits</td>
</tr>
<tr>
<td>$C_S$</td>
<td>cost due to structure damage</td>
</tr>
<tr>
<td>$dE$</td>
<td>incremental dissipated hysteretic energy</td>
</tr>
<tr>
<td>$D_i$</td>
<td>local damage index value;</td>
</tr>
<tr>
<td>$d_{ik}$</td>
<td>$k$ th discrete value for the $i$ th variable</td>
</tr>
<tr>
<td>$D_k$</td>
<td>Kratzig damage index value</td>
</tr>
<tr>
<td>$D_k^-$</td>
<td>Kratzig damage index value from negative cycles</td>
</tr>
<tr>
<td>$D_k^+$</td>
<td>Kratzig damage index value from positive cycles</td>
</tr>
<tr>
<td>$d_{max}$</td>
<td>maximum inter-storey drift ratio</td>
</tr>
<tr>
<td>$D_{PA}$</td>
<td>Park and Ang damage index value</td>
</tr>
<tr>
<td>$D_{set}$</td>
<td>global damage index value</td>
</tr>
<tr>
<td>$D_v$</td>
<td>set of discrete values for the $v$ th variable</td>
</tr>
<tr>
<td>$E_f$</td>
<td>total energy dissipated by an element in a test to failure</td>
</tr>
<tr>
<td>$E_i$</td>
<td>energy in a follower half-cycle</td>
</tr>
<tr>
<td>$E_{p,i}$</td>
<td>energy in a positive primary half-cycle</td>
</tr>
<tr>
<td>$F$</td>
<td>objective function value</td>
</tr>
<tr>
<td>$f(x)$</td>
<td>objective function</td>
</tr>
<tr>
<td>$F_y$</td>
<td>element yield strength</td>
</tr>
<tr>
<td>$g_j$</td>
<td>$j$ th constraint functions</td>
</tr>
<tr>
<td>$LN$</td>
<td>log normal distribution notation</td>
</tr>
<tr>
<td>$m$</td>
<td>number of total constraints of the objective function</td>
</tr>
<tr>
<td>$M_a$</td>
<td>moment magnitude of aftershock</td>
</tr>
<tr>
<td>$M_m$</td>
<td>moment magnitude of mainshock</td>
</tr>
</tbody>
</table>
$M_w$  moment magnitude

$n$  number of retrofit locations

$N_a$  number of aftershocks

$n_{EQ}$  number of earthquake records

$n_r$  number of maximum retrofits

$N_{aim}$  total number of required simulations

$n_v$  number of discrete design variables

$p$  site specific parameter

$p$  number of equality constraints

$P_{CP}$  collapse prevention penalty value

$P_{LS}$  life safety penalty value

$q_i$  number of available discrete values for the $i$th variable

$t$  time

$v$  number of design variables

$w_i$  weighting parameter

$x_i$  $i^{th}$ decision variable

$x_{iL}$  lower bound for $x_i$

$x_{iU}$  upper bound for $x_i$

$\alpha$  constant related to the mainshock magnitude

$\beta$  constant related to the mainshock magnitude

$\beta_e$  non-negative calibration parameter

$\delta_m$  maximum deformation response to the excitation

$\delta_u$  ultimate deformation capacity under monotonic loading

$\gamma$  mean daily rate of aftershocks

$\lambda_M$  mean occurrence rate of earthquakes

$\lambda_{M_a}$  mean occurrence rate of aftershocks
### List of Abbreviations

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>CISC</td>
<td>Canadian Institute of Steel Construction</td>
</tr>
<tr>
<td>CP</td>
<td>Collapse Prevention</td>
</tr>
<tr>
<td>DI</td>
<td>Damage Index</td>
</tr>
<tr>
<td>FHC</td>
<td>Follower Half-Cycle</td>
</tr>
<tr>
<td>FO</td>
<td>Fully Operational</td>
</tr>
<tr>
<td>FRP</td>
<td>Fiber-Reinforced Polymer</td>
</tr>
<tr>
<td>GA</td>
<td>Genetic Algorithm</td>
</tr>
<tr>
<td>IDA</td>
<td>Incremental Dynamic Analysis</td>
</tr>
<tr>
<td>IO</td>
<td>Immediate Occupancy</td>
</tr>
<tr>
<td>ISD</td>
<td>Inter-Storey Drift</td>
</tr>
<tr>
<td>ISDR</td>
<td>Inter-Storey Drift Ratio</td>
</tr>
<tr>
<td>LS</td>
<td>Life Safety</td>
</tr>
<tr>
<td>MISDR</td>
<td>Maximum Inter-Storey Drift Ratio</td>
</tr>
<tr>
<td>MRF</td>
<td>moment-Resisting Frame</td>
</tr>
<tr>
<td>NBCC</td>
<td>National Building Code of Canada</td>
</tr>
<tr>
<td>NEES</td>
<td>George E. Brown, Jr. Network for Earthquake Engineering Simulation</td>
</tr>
<tr>
<td>NGA</td>
<td>Next Generation of Ground-Motion Attenuation Models</td>
</tr>
<tr>
<td>PBEE</td>
<td>Performance-Based Earthquake Engineering</td>
</tr>
<tr>
<td>PEER</td>
<td>Pacific Earthquake Engineering Research center</td>
</tr>
<tr>
<td>PGA</td>
<td>Peak Ground Acceleration</td>
</tr>
<tr>
<td>PHC</td>
<td>Primary Half-Cycle</td>
</tr>
<tr>
<td>RC</td>
<td>Reinforced Concrete</td>
</tr>
<tr>
<td>SDOF</td>
<td>Single Degree-of-Freedom</td>
</tr>
<tr>
<td>UHS</td>
<td>Uniform Hazard Spectra</td>
</tr>
<tr>
<td>USGS</td>
<td>United States Geological Survey</td>
</tr>
</tbody>
</table>
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I would like to thank the faculty, staff and my fellow students at UBC and particularly the Okanagan Campus, who supported my learning and research. I would like to thank all the professors whose classes I attended, if occasionally only subconsciously.

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Special thanks to my supervisor Dr. Solomon Tesfamariam for his patience, support and encouragement through my undergraduate and graduate career.
Dedicated to my parents
1 Chapter: Introduction

1.1 Background

Earthquakes around the world continue to cause extensive loss of buildings, infrastructure, and human life. The 2010 Haiti earthquake resulted in billions of dollars of economic losses, millions of people displaced and thousands of lives lost (Cavallo, 2010). The 2010 Chilean earthquake, while resulting in large economic losses, had significantly fewer casualties (EERI, 2010).

The structural damage from the Chile earthquake was reduced because of the modern design standards used in that region. Modern design concepts include consideration of the load path to prevent catastrophic failure, proper detailing to resist cyclic loads, and elimination of stiffness irregularities. Conversely, the lack of proper construction practices in Haiti meant a much larger portion of the buildings collapsed despite a comparable earthquake magnitude with Chile (Kovacs, 2010). These differences highlight the improvements of modern seismic
design over past practices. Improved understanding of the behaviour and motion of structures in an earthquake had been applied to design more resilient structures thus improving its seismic performance. Old structures, however, in many locations, built before modern seismic design practices, are still vulnerable (Kovacs, 2010; Tesfamariam and Saatcioglu, 2008, 2010).

![Figure 1.2: Repairs in Tower O'Higgins, Concepcion by Ex-BGDA-](http://en.wikipedia.org/wiki/File:Torre_O'Higgins_conce.jpg) CC BY-SA 3.0

To reduce the seismic risk of deficient structures, different retrofit options are available. The structure could be torn down and replaced with a modern structure that follows the appropriate design standards. Full demolition and reconstruction could be very costly and the remaining lifetime value of the building would be lost. Also, demolition of a historically significant building would be a loss of value to society. One retrofit option is base isolation
By installing flexible connection between the foundation and the building, the motion affecting the structure can be altered or minimized to reduce damage. This retrofit method requires expensive excavation and foundation modification and so is often not feasible or required for most structures, so other options may be required (FEMA 547).

One of the most common methods to mitigate seismic damage is to retrofit the existing structure with additional columns, walls, braces, and other devices to improve the seismic resistance of the structure up to a desirable level (Thermou and Elnashai, 2006). Retrofit has the advantage of maintaining the existing value in the structure as only parts of the building would need to be changed. Also, it is often possible for the building to remain partially or wholly operational while the retrofits are applied. Thus, the retrofit is often a more cost effective seismic mitigation alternative. As there is a large stock of older, deficient structures, and a limited amount of resources, prioritizing retrofit spending is necessary (Tesfamariam and Saatcioglu, 2008). By analysing the performance of the retrofitted structure, decisions can be made as to the level of retrofit needed to meet a desired performance goal at the least cost.

The September 2010 Christchurch, New Zealand earthquake with a $M_w$ of 7.2 mainshock was followed by the February 2011, aftershock that had a $M_w$ of 6.2. The aftershock event was much weaker than the mainshock, but resulted in more losses because the buildings and infrastructure were damaged and/or weakened during the previous mainshock (Reyners, 2011). The March 2011 earthquakes in Japan had significant foreshock and aftershock events (Ide et al., 2011) that caused damage in addition to the mainshock event. When comparing these events, the building type, earthquake intensity, population density, and secondary effects (e.g. tsunami or landslide) are among the factors that contributed to the different levels of damage and loss that occurred.

Alongside the improvements to the field of structural dynamics, seismic hazard analysis has improved as well. From the Christchurch and Japan earthquakes, it is clear that multiple ground motion sequences are an important factor in structural seismic resistance (Goda and
Taylor, 2012; Goda et al., 2012; Goda 2012; Tesfamariam et al. 2014). If the goal of seismic analysis is to anticipate the real-world response of the structure then a significant part of the real-world seismic impact is not included if aftershocks are not considered. Including aftershocks necessitates a more thorough analysis as the motion of the structure and the effect on the structural components through multiple seismic events is a very complex process that cannot be accurately estimated.

The work of Tesfamariam et al. (2014) investigated the impact of aftershock sequences by simulating a number of reinforced-concrete (RC) frame configurations with one hundred mainshock-aftershocks sequences. The results of this work showed the increased drift demand on a structure when aftershocks sequences are included. When discussing non-code conforming structures, it is important to quantify the additional demand on a structure due to the aftershocks but applying this information to possible retrofit options is also important.

1.2 Objective

The goal of this thesis is to expand upon previous works to include the effect of mainshock-aftershock earthquake sequences on damage accumulation, and also to consider a retrofit optimization scheme to improve the structural performance. In order to achieve this goal, the study presented in this thesis applied and combined advancements in the fields of structural analysis, seismic hazard, retrofitting, optimization, and parallel computing. Modern structural simulation software was employed to analyze the performance of a non-code conforming structure under various circumstances. The areas of importance are summarized as follows:

- Seismic hazard analysis will include the effect of mainshock-aftershock sequences on drift and damage accumulation.
- Optimization methods will be integrated with the structural analysis methods to provide a retrofit scheme with a desired level of performance at a minimized cost.
- Parallel computing techniques will be applied to reduce the overall simulation and optimization computational time and improve the practicality of the presented methods.
Throughout this process, challenges and solutions to mitigate seismic induced damages of non-code conforming buildings will be highlighted with an example.

1.3 Organization of the Thesis

There are many fields of research involved in the work of this thesis. The literature review of these fields will be presented in sections based on the organization of the overall work as shown in Figure 1.3. Initially, the earthquake hazard (both mainshock and aftershock) is determined to apply appropriate ground motions for the structure simulation. The model of the structure is simulated and the response is recorded, which is used to determine the performance of the structure. Finally, an optimization algorithm is implemented to determine a set of optimized retrofit locations. Each step is further explained below.

In Chapter 2, a thorough literature review will be presented. The review covers the most important areas of the research separated into sections based on the steps in the analysis process. Chapter 3 presents a case study on the effect of mainshock-aftershock sequences on the damage accumulation that occurred in a simulation of a deficient RC structure. Chapter 4 expands the case study by applying an optimization method to determine the optimized retrofit locations in the structure with the objective function of cost minimization. Two optimization methods will be compared, namely a genetic algorithm and greedy algorithm. Finally, Chapter 5 will present some discussion of the results of the work, some conclusions that can be drawn from the results, and the direction of future research.
Figure 1.3: Retrofit optimization flow chart
Before assessing the performance of a structure, it is necessary to understand the earthquake hazard at the building site. In this section, the core parameters of the ground motion, evaluation, selection, and modification will be discussed.

2.1 Ground Motion Parameters

When describing the seismic input to an analysis procedure, a number of concepts are generally considered. FEMA 440 (2005) discusses the complexities of ground motion behavior. The first concept involves the seismic fault conditions; different movement and contact types at the fault affect the resulting properties of the earthquake. The contact at the fault often generates lateral movement but, depending on the fault type, one- or two-dimensional lateral movements and a vertical movement can be generated. The characteristics of the energy released by the seismic event are also an important concept. Generally, the moment magnitude is used as a measure of the earthquake intensity and corresponds to the amount of energy released. Also, the intensity and duration of the earthquake affects the energy imparted to the structure. The final concept to be discussed is the frequency content of the earthquake. The soil properties between the source of the earthquake and the building site affect the seismic wave attenuation due to the frequency transmission properties of different soil materials. All structures have a natural frequency due to the design and materials. Resonance between the natural frequency of the structure and the frequency content of the earthquake has a significant impact on the intensity of shaking experienced by the structure. The combinations of the many factors that affect the concepts discussed here have a significant effect on the resulting earthquake hazard and, as they are significant for the behavior of the structure, these concepts must be considered when comparing ground motion records.
2.2 Ground Motion Record Selection

Often, the representative seismic parameters for a given site are not adequately reflected in only one ground motion record. This deficiency is remedied by the use of multiple ground motion records that collectively represent the desired parameters. Also, the robustness of the analysis is increased when multiple records are tested against the system. The challenge in this is selecting the most representative set of earthquake records.

FEMA P695 (2009) addressed the need for a set of relevant earthquakes by setting the following criteria:

- **Code Consistent** – The records should be consistent with the ground motion requirements of ASCE/SEI 7-05 for three-dimensional analysis of structures.

- **Very Strong Ground Motions** – The records should represent very strong ground motions. In high seismic regions where buildings are at greatest risk, few recorded ground motions are intense enough, and significant upward scaling of the record is often required.

- **Large Number of Records** – The number of records in the set should be “statistically” significant such that the resulting collapse evaluations adequately describe both the median value and record-to-record variability of collapse capacity.

- **Structure Type Independent** – Records should be broadly applicable to collapse evaluation of a variety of structural systems. Accordingly, records should not depend on period, or other building-specific properties of the structure.

- **Site Hazard Independent** – Records should be broadly applicable to collapse evaluation of structures located at different sites, such as sites with different ground motion hazard functions, site and source conditions. Accordingly, records should not depend on hazard de-aggregation, or other site- or hazard-dependent properties.

The Pacific Earthquake Engineering Research (PEER) group Next Generation Attenuation (NGA) ground motion database contains records for 3551 three-component recordings from 173 earthquakes and 1456 recording stations from all around the world. Based on the
previously stated objectives, the PEER NGA was examined for the appropriate records. From this collection, FEMA P695 presents two sets of twenty ground motion records to represent near-field earthquakes, where the epicenter is within 10km of the building site, and far-field earthquakes, where the epicenter is beyond 10km. These records can be used to perform an analysis that will provide results for illustration. Site-specific results would require hazard analysis specific to a location.

2.3 Ground Motion Scaling

In order to test a specific structure, it is necessary to adjust the ground motion records to represent the hazard where the structure would be located as closely as possible. The most common ground motion parameter that needs to be adjusted is the intensity. In Canada, the method of ground record scaling is prescribed by the National Building Code of Canada (NBCC). The 2010 NBCC used a combination of the site location, soil classification, and structure period to determine the scaled level of the ground motion record. The site location is used to determine the seismic hazard based on the probable intensity of an earthquake. This information is given in the form to Uniform Hazard Spectra (UHS) to provide a standard way of determining the intensity of shaking for an area. The values given in a UHS selected based on the fundamental period of the structure and scaled according to the site soil classification. Based on different hazard levels, the UHS provides expected intensity of shaking for different spectral periods (Error! Reference source not found.). The different hazard levels for earthquakes are typically given in terms of probability of exceedence or return period. For example, a rare event is generally given as 2% in 50 year probability of exceedence or approximately equivalent to a 1 in 2500 years return period event.
There are generally two ways to scale the ground motion record to the UHS. The first method is described in more detail by Atkinson (2009). Initially, the UHS is compared to the spectral period content of the ground motion record over a relevant period range. The ground motion intensity is scaled by a factor that matches the ground motion spectrum to the UHS. This method is to ensure the ground motion applied to the test structure is the same intensity as an earthquake the structure would be expected to experience. The second method again compares the ground motion spectral period content to that of the UHS but instead is based on a method presented by Abrahamson (1992) and Hancock et al. (2006). This method generates wavelets to modify the earthquake record to more closely match the UHS. Compared to the first method, the second method does not provide as accurate matching at the specific structure period but does include a wider range of periods in the matching.

2.4 Aftershocks
Including aftershocks in seismic performance analysis presents a number of challenges similar to those included with mainshocks. Both mainshock and aftershock hazard are based
on site properties and available records. Additional challenges with aftershocks are to include the effect of the preceding mainshock and quantifying the series of aftershock events. Quantifying the seismic response of structures due to mainshocks and aftershocks is also more difficult as it requires multiple seismic events within a relatively short time frame. In this section, a literature review is performed that presents a number of different solutions to the challenges faced by aftershock seismic response analysis.

Luco et al. (2004) compared dynamic and static methods to compute the residual capacity of a mainshock damaged building. The comparison had shown that the static computation method underestimated the response of the structure due to the additional strength degradation imposed by cyclic loading during an earthquake. The authors used the dynamic results to propose a calibration of the static results to reflect the more accurate analysis methods. This has the effect of allowing a much faster static analysis to be more accurately used for post-mainshock residual capacity estimation. The residual capacity is important for estimation of aftershock performance.

Bazzurro et al. (2004) presented a method to estimate the damage level of a structure after an aftershock to assist with the post-earthquake damage assessment and inspection that typically occurs. The authors suggest that incremental dynamic analyses (IDA) are preformed prior to the seismic event to determine the relationship between the earthquake spectral acceleration and the structure roof drift. This information can be used to determine the occupancy level of the structure based on the magnitude of a given event. Additionally, this information was used to create fragility curves for the structures that can be used, with the mainshock and aftershock hazard, to estimate the probable damage states of a structure after one or more seismic events.

Yeo and Cornell (2005) produced a report on the stochastic characterization and time-dependent aftershock risk for performance-based earthquake engineering (PBEE). This report addresses many challenges of aftershocks in PBEE, such as assessing aftershock hazard and post-mainshock collapse probability. To address aftershock hazard, the report includes a method to estimate the probability of ground motion intensity. Yeo and Cornell (2005)
reported a method from previous research known as the “generic California” aftershock model that is based on a modified Omori law and the Gutenberg-Richter relationship of mainshock and aftershock magnitudes. This model is shown in equation (2.1):

\[
\gamma(t,M_w;M_m) = \frac{10^{a+b(M_w-M_m)}}{(t+c)^p}
\]  

(2.1)

where \( \gamma \) is the mean daily rate of aftershocks with magnitude \( M_w \), \( t \) is days after the main shock, \( M_m \) is the mainshock magnitude, and \( a, b, c, \) and \( p \) are site specific factors. Yeo and Cornell expanded upon this model to calculate the mean number of aftershocks in a given time span after a mainshock event. This information was used to generate aftershock hazard curves. Comparisons were performed of the mainshock and aftershock hazards in terms of annual rates of exceeding PGA and the change in mainshock-aftershock magnitude ratios over time. The effect of mainshock magnitude on aftershock hazard was also presented.

To quantify the performance of mainshock damaged buildings, Yeo and Cornell (2005) used methods based on the previously mentioned work of Luco et al. (2004) and Bazzurro et al. (2004). To re-state the method, generally, IDA was used to find the magnitude of event that would cause a specific damage state and this information was used to generate fragility curves and transition probabilities of the different damage states. The fragility curves provide a cumulative probability of the structure being in a certain damage state given the intensity of the earthquake. The transition probabilities describe the probability that the structure will enter a more damaged state after subsequent seismic events.

The report by Yeo and Cornell (2005) also included a chapter with a formulation of the future life-cycle financial loss analysis. A number of methods were included to model the life-cycle cost. A Poisson loss model was presented as a simple and well known method to determine a loss event. This method assumed the building was returned to an un-damaged state immediately following a loss event. This assumption was relaxed in a Markov process framework that was subsequently presented. The Markov process framework extended the Poisson model in a number of ways. The previously mentioned assumption of immediate
repair was relaxed and time-dependant repair was included. Additionally, discounted losses and non-homogeneous aftershock processes were included. A final method of using a renewal process for mainshock occurrences was presented.

Li and Ellingwood (2007) investigated the damage to a steel moment frame due to mainshock-aftershock sequences. The topic of the work is similar to Yeo and Cornell (2005), in that aftershocks were included in the performance evaluation of the structure, but the methods used are different. The properties of the aftershock were generated from the mainshock properties. The authors used the Gutenberg-Richter formula (shown in equation (2.2)) as a starting point for their method.

\[ N_a(M_a) = \alpha \exp(-\beta M_a) \]  

(2.2)

where \( N_a(M_a) \) is the number of aftershocks with magnitudes at or exceeding \( M_a \), \( \alpha \) and \( \beta \) are constants related to the mainshock magnitude. The authors included the work of Sunasaka and Kiremidjian (1993) which developed the relationship between the mainshock magnitude and the \( \beta \) factor for the California area. The work resulted in equation (2.3) describing the expected number of aftershocks with magnitude greater than or equal to 3.0.

\[ E(N_a(3.0)) = \exp(-0.647 + 0.684M_m) \]  

(2.3)

where \( M_m \) is the magnitude of the mainshock event.

Another important point highlighted by Li and Ellingwood (2007) is the observation of Fragiacomo et al. (2004) that the first aftershock causes greater damage to buildings than do later aftershocks of similar magnitude. Also, Lee and Foutch (2004) found that repeated identical earthquake ground motions do only slightly more damage than one occurrence of the same event. From this, Li and Ellingwood assume that the aftershock sequence can be modeled for damage assessment purposes with a single representative aftershock described probabilistically as the maximum of a series of \( N_a(M_m) \) aftershocks.
For their analysis, Li and Ellingwood (2007) used 30 natural earthquake records for the mainshocks. The aftershocks records were scaled using equation (2.4) based on the work of Sunasaka and Kiremidjian (1993).

\[
\frac{\lambda_{M_a}}{\lambda_{M_m}} = 10^{-b(M_m - M_a)}
\]

A typical reported value for \( b \) was 1.14 for the Western United states. The mean occurrence rates of earthquakes, \( \lambda_{M_m} \), with hazard levels of 10%/50yr and 2%/50yr chance of exceedence were presented by Li and Ellingwood (2007). The factors to scale the mainshock records to aftershock records were generated from the previous equations and the hazard curves from the U.S Geological Survey website. The results of Li and Ellingwood (2007) have shown probabilistic damage levels for mainshock only and mainshock-aftershock sequences showing significant increases in damage level with the inclusion of aftershocks.

Jalayer et al. (2010) presented a method that is similar to that presented by Yeo and Cornell (2005). The aftershocks were modeled with a modified Omori law with the Gutenberg-Richter law given in equation (2.4), similar to Yeo and Cornell (2005). Jalayer et al. also included a Bayesian framework to update the aftershock probability model as new data became available after an earthquake. As seismic events are rare, including the most recent data with Bayesian updating allows prediction of future events to be more accurate. This framework could be used for post-earthquake decision making between actions such as evacuation, shut-down, repair and re-occupancy.

\[
\gamma(t, M_w, M_m) = b \ln(10) 10^{\alpha + b(M_m - M_w)} \left( t + c \right)^\beta
\]

Ryu et al. (2011) proposed a method to develop fragility curves with aftershocks included. This method used mainshock-aftershock sequences with IDA to generate the fragility curves. Additionally, this method was able to include uncertainty in the mainshock response and in the damage state thresholds. Uma et al. (2011) applied the method of Ryu et al. to compare fragility curves generated by New Zealand and US models for single-degree-of-freedom
(SDOF) systems. Uma et al. suggest that, due to differences in engineering judgment and empirical expressions, the New Zealand models are more fragile than the US models, but further studies are needed for this to be conclusive.
3 Chapter: Damage Accumulation Due to Mainshock-Aftershock Earthquake Sequence

The effect of mainshock-aftershock sequences on the damage accumulation is illustrated with a case study. A simulation of a typical seismic deficient RC structure will be used to show the level of damage caused by mainshock only and mainshock-aftershocks earthquakes.

3.1 Seismic Structural Performance

During an earthquake, a structure can exhibit many complex behaviors. The interaction of all structural components affects the motion and resulting damage of the structure. The nonlinear nature of structural materials results in dynamic time-varying stiffness and strength degradation, and consequent damage accumulation. SEAOC Vision 2000 (1995); ATC 40 (1996); and FEMA 273 and 274 (1996) established the basis for much of the principles behind performance based design and implementation. These documents suggest a number of different methods to quantify the seismic performance of a structure. Many performance measures rely on some knowledge of the structure capacity (often calculated from original material and design information) to determine a performance level. Often these measures require information that is only available through structural simulations, such as in FEMA P695 (2009), where plastic rotation was suggested as a measure of the performance. An updated review of the mechanisms, advantages, and disadvantages of appropriate methods was presented by Ghobarah (2000).

The concept of structure performance is often presented as a level of damage. There are different ways of describing the general concept of damage. One method is based on the structural element’s capacity to resist the imposed loads. When the element deflects more under load, or fails to resist the load entirely, the element is said to be more damaged. Another interpretation of damage is based on the element’s residual capacity after the event and its ability to be repaired. In the following sections of this chapter, more quantitative measures of the structure performance will be discussed.
3.1.1 Global Performance Measures

Global performance measures are used to assess performance of the structure on a macro scale. The performance of individual structural components is not recorded; instead, the performance is measured of combined floor systems or of the whole structure. One of the most common global performance measures is the maximum inter-storey drift ratio. SEAOC Vision 2000 (1995) presented drift as a measure of performance and has become one of the most commonly used methods due to its simplicity. Maximum inter-storey drift ratio (MISDR) is a measure of the relative displacement of different floor levels within a building, and is expanded by dividing it by the floor height (i.e. distance between the floors) to obtain the drift ratio. Ghobarah (2001) summarized and combined the suggested performance levels, damage states, and MISDR limits for reinforced-concrete structures to clearly show the limits that are expected for each performance level. Additionally, FEMA 356 correlated the building ISD limits, structure performance levels, and earthquake hazard levels with prescribed safety objectives as shown in Table 3.1. Each combination of hazard and performance (‘a’ to ‘p’) represents a discreet objective that is a component of the overall safety objective. This method is often used because no knowledge of the structure capacity is required, though more complex structure behavior common to cyclic loading is not represented in the result.
Table 3.1: Inter-storey drift limits and corresponding performance levels
(Adapted from FEMA 356)

<table>
<thead>
<tr>
<th>Earthquake Hazard Level</th>
<th>ISD Limit</th>
<th>FO 0.2%</th>
<th>IO 0.5%</th>
<th>LS 1.5%</th>
<th>CP 2.5%</th>
<th>Safety Objective</th>
</tr>
</thead>
<tbody>
<tr>
<td>50%/50 year</td>
<td>a</td>
<td>b</td>
<td>c</td>
<td>d</td>
<td></td>
<td>Limited</td>
</tr>
<tr>
<td>20%/50 year</td>
<td>e</td>
<td>f</td>
<td>g</td>
<td>h</td>
<td></td>
<td>Basic</td>
</tr>
<tr>
<td>10%/50 year</td>
<td>i</td>
<td>j</td>
<td>k</td>
<td>l</td>
<td></td>
<td>Enhanced</td>
</tr>
<tr>
<td>2%/50 year</td>
<td>m</td>
<td>n</td>
<td>o</td>
<td>p</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

FO = Fully operational; IO = Immediate Occupancy; LS = Life Safety; CP = Collapse Prevention

3.1.2 Local Performance Measures

Local performance measures describe the behaviour of individual structural elements separate from the performance of the system. Many local performance measures have been reported by Williams and Sexsmith (1995). Often called ‘damage indices’ (DIs), these methods use hysteretic stress-strain behaviour of the structural member as a performance measure. DIs are reported as a unit-less value between zero and one with some variation on which specific DI values correspond to what damage state. A comparison of the DI values and the corresponding MISDR and damage states is shown in Table 3.2. A summary of the many available DI methods are shown in Table 3.3. Non-cumulative mechanisms are similar to inter-storey drift as they only represent single aspects of structure behaviour, such as the maximum or end value for the whole cyclic loading event. Cumulative measures include the behaviour of the structural members, in some form, through all of the loading cycles. Deformation, stiffness and energy based methods rely on assessing different aspects of the structural members’ behaviour through the loading cycles.

This thesis focuses on the approaches proposed by Park et al. (1984) and Kratzig et al. (1989). The Park and Ang DI was selected as it is a commonly referenced and implemented method. In contrast to the Park and Ang DI, the Kratzig DI is based on the energy dissipation...
of the structural member. Together, these damage indices represent commonly applied methods and include differing methodologies.

Table 3.2: Comparison of DI value, MISDR, and damage level

<table>
<thead>
<tr>
<th>DI value range</th>
<th>MISDR</th>
<th>Damage level</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;0.1</td>
<td>&lt;0.2%</td>
<td>Fully operational</td>
</tr>
<tr>
<td>0.1 – 0.25</td>
<td>&lt;0.5%</td>
<td>Immediate occupancy</td>
</tr>
<tr>
<td>0.25 – 0.4</td>
<td>&lt;1.5%</td>
<td>Life safety</td>
</tr>
<tr>
<td>0.4 – 0.8</td>
<td>&lt;2.5%</td>
<td>Collapse Prevention</td>
</tr>
<tr>
<td>0.8 – 1.0</td>
<td>&gt;2.5%</td>
<td>Collapse</td>
</tr>
</tbody>
</table>

Table 3.3: Summary of damage indices and damage

<table>
<thead>
<tr>
<th>Damage mechanism</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deformation based</td>
<td></td>
</tr>
<tr>
<td>Non-Cumulative</td>
<td>Fajfar (1992)</td>
</tr>
<tr>
<td>Cumulative</td>
<td>Bannon et al. (1981)</td>
</tr>
<tr>
<td>Stiffness Based</td>
<td>McCabe and Hall (1989)</td>
</tr>
<tr>
<td>Energy Based</td>
<td>Roufaiel and Meyer (1987)</td>
</tr>
<tr>
<td>Combined (Deformation and Energy)</td>
<td>Stephens and Yao (1987)</td>
</tr>
<tr>
<td>Low-Cycle Fatigue</td>
<td></td>
</tr>
<tr>
<td>Repeated Cumulative Effect</td>
<td>Calado and Castiglioni (1996)</td>
</tr>
<tr>
<td></td>
<td>McCabe and Hall (1989)</td>
</tr>
<tr>
<td></td>
<td>Chung et al. (1989)</td>
</tr>
</tbody>
</table>

3.1.2.1 Park and Ang Damage Index

The Park and Ang damage index $D_{PA}$ (Park et al. 1984) was developed as a hybrid method, which includes both displacement and dissipated hysteretic energy of the structural member being assessed. This is a commonly used damage quantification method that is computed using equation (3.1).

$$D_{PA} = \frac{\bar{\delta}_u}{\delta_u} + \beta \frac{\int dE}{F \delta_u}$$  (3.1)
where $\delta_m$ = maximum deformation response to the excitation; $\delta_u$ = ultimate deformation capacity at rupture under monotonic loading; $F_y$ = element yield strength; $\beta_e$ = non-negative calibration parameter; and $dE$ = incremental dissipated hysteretic energy.

The first term measures the displacement based damage where the maximum displacement $\delta_m$ is compared to its ultimate displacement $\delta_u$. The second term uses the dissipated energy of the element to account for the accumulated damage and it is compared to the ultimate energy capacity to measure failure. In subsequent work, this method has been expanded upon to use moment and rotation instead of force and deflection as the basis for the damage but in this work the original method is used. The resultant value of $D_{PA}$ represents the approximate damage level to the structural element.

While the Park and Ang DI provides a reasonable measure of the structural element damage, there are some limitation to the method. For RC structural members, the ultimate deformation ($\delta_u$) and yield force ($F_y$) parameters can be determined from a physical test to failure for an accurate value, or knowledge of the member design from which the parameter could be estimated or simulated. Furthermore, the calibration parameter ($\beta_e$) is generally based on test data of similar structures, but values have been reported from below 0.1 to 0.5 (Williams and Sexsmith 1995) resulting in significant changes to the calculated damage levels.

### 3.1.2.2 Kratzig Damage Index

Kratzig et al. (1989) presented a formulation using the energy dissipation of the structural members. The method is based on the definition of primary and follower half-cycles shown in Figure 3.1. A primary half-cycle (PHC) is defined as the first half cycle of loading at a given amplitude. Subsequent half-cycles are follower half-cycles (FHC) unless they exceed the previous maximum amplitude. For positive element deformation, the positive damage parameter, $D_{k}^{+}$, is:
where \( E_{p,i}^+ \) = the energy in a PHC; \( E_i \) = the energy in a FHC; \( E_f \) = the total energy absorbed by the element in a test to rupture. Equation (3.2) can be used for a similar parameter for the negative displacement case. The positive and negative parameters are combined for the overall damage index as shown in Equation (3.3).

\[
D_k^+ = \frac{\sum E_{p,i}^+ + \sum E_i^+}{E_f^+ + \sum E_i^+}
\]

\[
D_k = D_k^+ + D_k^- - D_k^+ D_k^-
\]  

This method accounts for the energy dissipated by a structural member and thus includes the force and displacement experienced by that member. In summary, the energy of the strongest cycles is compared to the energy capacity at failure, while the energy from other cycles is included to account for the continuous accumulated energy dissipation. The combination of PHC and FHC energy has been shown to approach unity at failure (Williams and Sexsmith 1995).
The Kratzig DI is able to include the effects of both single high-amplitude cycles, and repeated low-amplitude cycles but faces some limitations for the overall method. Similar to the Park and Ang DI, the Kratzig DI contains a parameter based on the behavior of the element when loaded to failure thus some knowledge of this element design would be required. While studies had shown that the Kratzig DI converged to unity at failure (Williams and Sexsmith 1995), little research had been done on the linearity of the values between 0 and 1.

3.1.3 Application of Selected Damage Indices

To apply the damage indices to a structure, some parameters must first be determined. In the case of the PA DI used in this thesis, the calibration parameter was selected based on the values presented in the literature shown previously. For both the PA and Kratzig DIs parameters based on the behavior of the individual sections were required. These parameters were calculated based on results of monotonic loading simulations of the individual members. From these simulations, the relevant information was extracted to include in the DI models.

With the parameters for the models determined, the results of the DI methods still only provide the damage values for individual structural elements. Bracci et al. (1989) presented a generalized method to aggregate the local damage indices, $D_i$, into a global damage index $D_{set}$, and is shown in equation (3.3). Where $w_i$ is a weighting parameter that allows other parameters to be introduced into the calculation. Increasing values of the $b$ parameter result in greater emphasis of the more severely damaged members. For a more simple equation, the parameters $w_i$ and $b$ can be arbitrarily set to 1.0, resulting in the formulation shown in equation (3.4). This simpler formulation was the equation used in this thesis to aggregate the individual DI values for the whole structure.

$$D_{set} = \sum w_i D_i^{(b+1)} \left/ \sum w_i D_i^b \right.$$  
(3.3)

$$D_{set} = \sum \frac{D_i^2}{D_i}$$  
(3.4)
3.2 Case Study: Van Nuys Hotel

The simulated structure used throughout this thesis is modeled after the Van Nuys Hotel building reported by Krawinkler (2005). The Krawinkler report includes many of the design and material properties of the building such as reinforcement detailing and concrete strength. The structure is a hotel built in Van Nuys, California, USA (Figure 3.2) and was constructed in 1965 according to the 1964 Los Angeles City Building Code. The building is a RC moment resisting frame with 7 storeys and 8 bays in the east-west direction and 3 bays in the north-south direction. Krawinkler reports the concrete compressive strength as 5 ksi (34 MPa) for the first storey, 4 ksi (27 MPa) for the second storey, and 3 ksi (20 MPa) for the third to seventh storeys. Column reinforcing steel is reported as ASTM Grade 60, beam and slab reinforcement steel is reported as ASTM Grade 40. Additional information on the building design can be found in Appendix A. This building was chosen because it represented current practise at the time of construction and is seismically deficient according to current design codes. Additionally, this structure is used as an example in a number of different PEER reports.

![Figure 3.2: South frame elevation of Van Nuys hotel](image-url)
3.2.1 Structure Modeling

The simulation of the Van Nuys hotel was completed in the Open System for Earthquake Engineering Simulation (OpenSees) software platform (http://opensees.berkeley.edu). This open source software is supported by the Pacific Earthquake Engineering Research Center (PEER) and the George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES) and is designed specifically to model a structure’s performance during an earthquake and provide meaningful results (McKenna et al., 2011). The hotel building was modeled as a 2D frame based on the reinforcement detailing provided by Krawinkler (2005). The reduced 2D frame includes the majority of the seismic force resisting system while significantly reducing the computational time to model. In this chapter, a modified version of OpenSees 1.6.2 provided by Koduru and Haukaas (2010) was used to perform the simulation. The modifications correct some problems with the damage index commands in the default software.

The structural elements were modeled as elastic elements with plastic behaviour concentrated at plastic hinge lengths at the element ends. The plastic behaviour of the hinge lengths was simulated using a built-in fibre model method and the plastic hinge length was set at 0.3 times the length of the structural member. This length was chosen based on documentation of the OpenSees software as it is a value that typically contains enough of the plastic behaviour of a reinforce-concrete member for meaningful simulation results without needlessly increasing the simulation time. The element portions that were not designated as plastic hinges were modeled as linearly elastic under the assumption that these element portions would not experience loads beyond yield strength. One advantage of the fibre model is that the force and deflection of the material sections within the element are known and thus data about the damage of each element can be recorded at each time step within the simulation. The main disadvantage of the fibre model is that as the number of fibres increase, so does the simulation time. The level of accuracy required must be balanced with any time limitations of the simulation.

The beam-column connections for this simulation were modeled as rigid connections. The rigid connections reduce the accuracy of the model as real-world behaviour would be
different but would have added a further layer of complexity to the modeling and simulation. Non-rigid connection would have added a further category of structural members to assess as the connections may have dissipated energy in addition to the beams and columns but would have also been a potential source of structural failure. Adequate results were obtained by only modeling the beams and column behaviour without the increase in computation time that would have been added by including beam-column connection behaviour.

3.2.2 Ground Motion Selection and Scaling

The ground motion records applied to the structure model were based on the FEMA P695 record set. The specific properties of the earthquake records used are shown in Table 3.4. Mainshock records were scaled to the USGS UHS for California while the aftershock records were scaled relative to the mainshock UHS based on the previously reported equation (2.4).
from Sunasaka and Kiremidjian (1993). SeismoMatch software (http://www.seismosoft.com) was used to scale the records to the generated UHS spectra using the methods presented by Abrahamson (1992) and Hancock et al. (2006). When scaling the records, a damping value of 5% was used as it represents a generally applicable value for most buildings. The damping value does influence the acceleration values of the UHS but refinement of the damping value was beyond the scope of this thesis. To illustrate the damage progression in the test structure, a hazard level of 10%/50 years and 15%/50 years probability of exceedence was chosen for the mainshock and aftershock, respectively. This lower hazard level was selected due to the vulnerability of the test structure, as a higher intensity earthquake could cause collapse of the structure early in the simulation and would not provide a meaningful comparison of the aftershock damage. A plot of the scaled mainshock and aftershock spectra with the respective target UHS is shown in Figure 3.4. Between the mainshock and aftershock events, a delay was included to allow the building to reduce its motion to an at-rest state. This delay has the effect of separating the two ground motion records into discrete mainshock and aftershock events.

Table 3.4: Summary of selected earthquake records

<table>
<thead>
<tr>
<th>FEMA P695</th>
<th>Earthquake</th>
<th>PEER_NGA Record Information</th>
</tr>
</thead>
<tbody>
<tr>
<td>ID</td>
<td>M</td>
<td>Year</td>
</tr>
<tr>
<td>3</td>
<td>7.1</td>
<td>1999</td>
</tr>
<tr>
<td>4</td>
<td>7.1</td>
<td>1999</td>
</tr>
<tr>
<td>5</td>
<td>6.5</td>
<td>1979</td>
</tr>
<tr>
<td>6</td>
<td>6.5</td>
<td>1979</td>
</tr>
<tr>
<td>7</td>
<td>6.9</td>
<td>1995</td>
</tr>
<tr>
<td>8</td>
<td>6.9</td>
<td>1995</td>
</tr>
<tr>
<td>9</td>
<td>7.5</td>
<td>1995</td>
</tr>
<tr>
<td>10</td>
<td>7.5</td>
<td>1999</td>
</tr>
<tr>
<td>13</td>
<td>6.9</td>
<td>1989</td>
</tr>
<tr>
<td>14</td>
<td>6.9</td>
<td>1989</td>
</tr>
<tr>
<td>15</td>
<td>7.4</td>
<td>1990</td>
</tr>
<tr>
<td>16</td>
<td>6.5</td>
<td>1987</td>
</tr>
</tbody>
</table>
3.3 Experimental Procedure

A number of disparate systems were combined to complete the simulation procedure and obtain the required results. After the first step of determining an appropriate structure to model, a two-dimensional frame of the structure was programmed using the OpenSees Software. The properties of the structural elements were based on the mechanical properties of the building presented by Krawinkler (2005). Additionally, a number of simulation parameters had to be adjusted based on OpenSees documentation to obtain a stable model simulation. Ground motion input was based on the seismic records shown in Table 3.4 and were manipulated to represent the desired building location and intensity. The modified records were saved in a consistent format to be read by the simulation software.

Before the simulation could produce the desired results, a number of DI specific parameters had to be calculated. The individual structural elements were separately modeled and subjected to a monotonic loading simulation to determine the properties of the sections. The resulting parameters were saved to text files to be read by the simulation software when calculating the DI value within the specific DI software functions.
With the separate components completed, the simulation was run through to completion. The required inputs were either programmed into the simulation software or read from text files and the desired output data was written to separate text files for analysis. All of the simulation results were loading into MATLAB to perform further calculations and plotting of the data.

### 3.4 Analysis of Results

From the simulation analysis, many different data were recorded. The inter-storey drift and damage index values were recorded from the simulation software for all of the mainshock and aftershock simulations. Figure 3.5 shows typical ISD variations for the second storey subjected to one of the earthquake records and its aftershock. In this figure, one can clearly see some important stages of the simulation. The initial mainshock event in the figure is a scaled record of the FEMA P695 ID#13 and the following aftershock event is a scaled ID#9 record. The typical features during the simulation are shown in Figure 3.5. This figure displays the ISD results for the second storey when subjected to the scaled 1989 Loma Prieta (Rec. No. 752) as a mainshock followed by the scaled 1995 Kocaeli, Turkey Rec. No. 1158) aftershock. The peak ISD for the mainshock (shown just before the 10 second time mark) displays the period of most intense motion of the structure. A delay between the mainshock and aftershock (shown between the 35 and 55 second times) displays typical damping behaviour nearly to a resting state. Finally, the incidence of most intense motion of the structure during the aftershock (shown just before the 60 second mark) is the maximum ISD for that portion of the ground motion sequence.
The ISD results of the simulation are shown in Figure 3.6. These results show each individual mainshock record, mainshock-aftershock sequence, and average values for the residual and maximum ISD. For comparison, the mainshock record results in an average MISD value of 1.3% while the mainshock-aftershock sequence results in a MISD of 1.45%. When compared to the performance levels given in Table 3.1, these results represent a shift from an ‘Immediate Occupancy’ performance level to a ‘Life Safety’ level.

In all cases, the first storey displayed the largest ISDR of all the storeys. This was likely the result of a soft-storey effect created by the taller first-storey columns. The soft-storey effect is typical of this design layout and commonly results in concentrations of the seismic damage at the first storey columns. Furthermore, the next largest ISDR was shown at the fourth storey, followed by the third. One possible cause of this behaviour was that a higher degree of freedom mode effect was the dominant case. This multi-degree-of-freedom behaviour displays displacement effects that may not have been shown in a pushover analysis.
The hysteretic response of each ground motion was used to compute the corresponding Park & Ang ($D_{PA}$) and Kratzig ($D_K$) damage indices for the structural elements. An example of the hysteretic motion of building columns for a given mainshock-aftershock sequence is given in Figure 3.7 with the cycles due to aftershock highlighted. For the displayed mainshock-aftershock sequence, the majority of the increased displacement occurs during the mainshock but the aftershock causes a significant number of additional cycles. The additional cycles increase the energy dissipated by the column and contribute to the level of damage as calculated by a DI. Similar to Figure 3.6, these results show the first storey has the highest displacement but also show the load incurred on the columns. As the columns at different floors have different dimensions and reinforcement, further analysis is necessary to determine a level of damage.
Figure 3.7: Hysteretic curves for the exterior columns of the first 6 storeys (C1-C6) during a mainshock-aftershock earthquake sequence.

Figure 3.8a and Figure 3.8b present the evolution of the calculated damage index results ($D_{PA}$ and $D_K$, respectively) for columns C1-C6 for the simulation of the mainshock-aftershock record sequence shown in Figure 3.5. As expected from the results presented in Figure 3.6 and Figure 3.7, the first floor columns experienced the highest level of damage but the calculated DI results allow a more direct relative assessment. The vast majority of the damage was caused by the mainshock motion with only a small portion of the damage was a result of the aftershock motion. The individual structural element DI results allow for a more detailed assessment of the structure. The element DI results could be used to identify weak points in the structure such as critical load path members or local irregularities to be remedied through retrofit or other means. The higher DI value for the first storey column is representative of the hysteretic curve shown for that column. The first storey location resulted in much higher axial loads on the column while the soft-storey effect resulted in a
higher displacement. These combined effects resulted in higher forces and energies, thus increasing the DI values for both methods. Similar to the inter-storey drift results, the third and fourth storey columns had a higher DI value than the second storey.

![Figure 3.8: Damage index results of a mainshock-aftershock earthquake sequence for single columns. The ISD results from Figure 3.5 are included for comparison.](image)

After analysis of the individual member results, further analysis can be done to assess the performance of the structure as a whole. The desired focus and results of the analysis can influence the choice of aggregation method. In this analysis, equation (3.4) was applied to obtain the results shown in Figure 3.9a and Figure 3.9b for the progression of both damage indices ($D_{PA}$ and $D_{K}$, respectively) aggregated for the whole structure. The different lines in each figure represent the different mainshock-aftershock earthquake sequences. The variation can be attributed to the differences in the original earthquake records. The generalized PEER record set contains a variety of different fault-slip types, magnitudes, and frequencies. Even though the records were scaled to the UHS, the inherent characteristics of the records remain
and cause different damage profiles. The earthquake sequences variation in maximum intensity and duration impact the resulting DI value.

Figure 3.9: Example progression of damage index values for a) Park and Ang and b) Kratzig damage indices

To show the variability in the results provided by the different ground motions, the damage index values presented in Figure 3.9a and Figure 3.9b were fit to a normal distribution, $N(\text{mean}, \text{standard deviation})$. By fitting the results shown in Figure 3.9 to a statistical distribution, further trends in the results can be observed. The cumulative distribution of the resulting DI values aggregated for all earthquake sequences is shown in Figure 3.10. This figure shows the general increase in expected damage level when aftershocks are added to the earthquake sequence. When fit to a log normal distribution, the $D_{PA}$ distribution parameters were calculated as $\ln(-2.00,0.39)$ and $\ln(-1.54,0.47)$ for the post-mainshock and post-aftershock results, respectively. The change from MS to MS-AS with $D_{PA}$ represents a 60% increase in the DI mean value and would change the building from an ‘Immediate Occupancy’ state to nearly a ‘Life Safety’ state. With the mainshock-aftershock results, both mean and standard deviation values have increased. The $D_K$ distribution parameters when fit to a normal distribution were calculated as $\ln(-1.53,0.20)$ and $\ln(-1.34,0.15)$ for the post-mainshock and post-aftershock results, respectively. The difference in the mean values for
$D_K$ represents a 20% increase and would just push the building from the top end of an ‘Immediate Occupancy’ state into a ‘Life Safety’ state. Given the large variation of the $D_{PA}$ values, the $D_K$ could potentially be a better indicator of damage as it offers more consistent results for the simulated structure. However, this has to be verified as some parameters of the DI models were based on simulated inputs for the member capacity and not experimental test data. It is important to note that these results represent the damage level after the MS-AS sequence and this

![Graph](image)

*Figure 3.10: Cumulative distribution for the aggregated damage index values with data points and fitted log normal curves*

### 3.5 Summary

In this chapter, a case study was introduced to illustrate the behaviour of a seismically deficient RC structure when subjected to 12 MS-AS earthquake sequences. The seismic performance was quantified through use of damage indices. The damage indices include maximum and residual inter-storey drift, hysteretic and damage index for a selection building columns, aggregated damage index values for all mainshock-aftershock sequences, and the
overall cumulative distribution of the damage index values. The results generally show the 
first storey columns as being the most damaged when averaged for all earthquake sequences. 
This result is consistent with the potential soft-storey irregularity identified from past 
earthquake reconnaissance report studies (e.g. Tesfamariam and Saatcioglu, 2008). 
Comparing the resulting performance as shown by the damage measures shows that the two 
damage indices
4 Chapter: Retrofit Location Optimization

This chapter presents the retrofit configuration that was applied to the RC building and the optimization methods that were implemented to determine the location of the retrofit within the structure. The retrofit and optimization methods are illustrated through implementation on the previously presented Van Nuys hotel model (Figure 3.2).

4.1 Retrofit

Upgrading non-code conforming buildings to mitigate seismic induced damages is an important and complex task. As a starting point, it is important to understand the deficiencies that are common in structures. FEMA 547 (2006) provides generalized categories of seismic deficiencies:

- Global strength
- Global stiffness
- Configuration
- Load path
- Component detailing
- Diaphragms
- Foundations
- Other deficiencies

Global strength and stiffness deficiencies are based on the capacity of the building elements. Strength and stiffness are linked in the element behaviour but are considered separately. For example, a steel column and RC column may have the same axial load capacities but the steel column will experience much larger displacements from a lateral load due to the reduced stiffness of steel. These differences in behaviour require consideration of different designs or retrofits to remedy. Configuration deficiencies are most commonly plan or vertical irregularities that create torsional response or concentrated demand at certain locations. Load path deficiencies occur when critical members would fail well before the overall system capacity is reached. Component detailing involves elements failing due to inadequate connections or poor confinement, preventing much of the nonlinear element capacity. Diaphragms are important to distribute the forces between the force-resisting elements.
Foundation deficiencies occur when the main point of failure is the foundation’s inability to transfer forces between the soil and the structure. There exist numerous retrofit techniques to mitigate the categories of seismic deficiency listed above (e.g. Housner, 1997; Thermou and Elnash, 2006; FEMA 547). Retrofit techniques can be generally grouped into classes as proposed by FEMA 547:

- Add structural elements
- Enhance performance of existing elements
- Improve connections between components
- Reduce demand on the structure
- Remove selected components

Different classes of retrofit are recommended depending on the deficiencies present in the structure. Adding (e.g. steel braces) or strengthening (e.g. fibre-reinforced polymer wrapping) structural elements improves the local and global capacity of the structure to withstand additional force or deflection. In some cases, the improvement of the member is done by weakening the member (thus reducing local performance) in order to balance irregularities in the structure and improve global performance. Improving the element connections allows the full strength and displacement capacity to be developed. More complex evaluations can reduce the demand on the structure by removing mass (e.g. removing equipment or entire upper floors), adding dampers, or isolating the base of the structure. Removing brittle components can improve the behavior of the whole structure or mitigate the effects of irregularities.

4.2 Structure Modeling with Parallel Computing

Modern software is increasingly integrating parallel computing techniques to improve simulation and optimization time requirements. Parallel computing techniques have been a research topic in the field of computer science for many years. Almasi and Gottlieb (1989) provide a base for understanding many of the core concepts of parallel computing. With modern advances in hardware, parallel computing capabilities have become more common and easier to apply. Though the field has developed for many years, Asanovic et al. (2009) state that there are still many challenges in connecting parallel computing methods to benefit
other fields of research. Additionally, Braun et al. (2001) have shown that algorithm type and hardware configuration can greatly impact the degree of benefit from parallel computing.

Parallel computing has been applied to a number of topics in civil engineering. Adeli and Kumar (1995) and Theirauf and Cai (1997) have presented parallel computing techniques for improving optimization algorithms applied to structural optimization. Kandil and El-Rayes (2005) have applied parallel optimization methods to complex multi-objective construction planning problems. When applied to improving optimization algorithms, parallel computing is often used to evaluate multiple solutions simultaneously. Alonso et al. (2007) have separated the analysis of a large 3D structure model such that sections of the structure are analyzed on different computers in order to reduce the overall computation time. Additionally, when analysis requires results from multiple earthquake records, simulation of the different records can be completed simultaneously. From these works it is clear that many areas of civil structural engineering can benefit from applications of parallel computing.

### 4.3 Optimization

An optimization problem entails consideration of objective functions, decision variables, and constraints. The decision variables are any property that can be changed at the discretion of the designer while the constraints limit the possible decisions in regards to the decision variables. The objective function is an overall measure of the success of the design. In most problem definitions, the objective function is developed as a cost function and is optimal when at a minimum. The general form of an optimization problem is shown in equation (4.1).

\[
\begin{align*}
\text{minimize} & \quad f(x) \\
\text{subject to} & \quad \begin{cases} 
  g_j = 0, & j = 1, \ldots, p \\
  g_j \leq 0, & j = p + 1, \ldots, m \\
  x_i \in V_i, & V_i = \{v_{i_1}, v_{i_2}, \ldots, v_{i_{n_i}}\}; i = 1, \ldots, n_v \\
  x_{i_{n_v}} \leq x_i \leq x_{i_{n_v}}'; i = n_v + 1, \ldots, n
\end{cases}
\end{align*}
\] (4.1)

where:
The constraint functions can be applied to the optimization definition such that undesirable variables or solution values are penalized or explicitly disallowed. When applied to structural performance problems, the nature of the system and desired outputs affect how the optimization functions should be developed and which method would be best suited to complete the optimisation. To show the challenges of seismic retrofit optimization, a simple case will be expanded upon. The simplest case of structural optimization is one where the structure is elastic, the applied force is static, and the decision variables are continuous.

This case would be easy to optimize as there are numerous available methods, but the system does not realistically or accurately represent the real-world behaviour of a structure. To improve the accuracy of the system behaviour, the non-linear nature of the materials would have to be included and would result in many complex non-linear interactions between the structural components. Due to this, any optimization methods that require linearity could not be used, and non-linear methods that are much more complex and calculation intensive would be required.

To simulate the dynamic behaviour of the structure, the external load and displacements, and the individual member behaviour must be calculated for each time step in sequence. This process significantly increases the time and computational power needed to calculate the structure’s state at the end of the simulation. The increased time does not explicitly eliminate
certain optimization methods, but many methods (such as brute force enumeration of all possible solutions) would require an impractical amount of time to generate an optimized solution.

Finally, the decision variables of the structure system are generally not continuous. In many engineering decisions, the available materials are limited by the market; e.g. when choosing the size of a beam, the optimal size may not be available and, thus, a smaller or larger one would need to be selected, affecting the optimization method. In some cases, such as the one for seismic retrofit, many retrofit locations are possible but the retrofit must be fully applied, or not at all; a half-retrofit is not possible. This causes the system to have discrete and discontinuous variables. This added difficulty of the decision variables eliminates another set of possible optimization methods.

While many optimization methods are eliminated, there remain a number of possible methods. Burns (2002) provides extensive discussion on different types of problems in structural optimization and a number of optimization methods appropriate for each case. A review of the methods that have been applied to structure and retrofit optimization is presented in the following paragraphs.

Agrawal and Yang (1999) applied three search methods to a problem of passive damper placement for seismic and wind loads. A number of different objective equations (including terms for energy, damping ratio, and inter-storey drift) were used to generate optimized damper locations. Additionally, it was shown that the optimal locations obtained by minimizing peak ISD of buildings depend on the specific design earthquake. Ganzerli et al. (2000) included the concepts of performance-based design in the optimization of a simplified structure subject to earthquake loading. Uncertainties in the structural period and in the earthquake excitation were also taken into account. Abdullah et al. (2001) used genetic algorithm (GA) to select placement of sensor/actuator pairs in a 40-storey elastic structure with three input earthquake records. The proposed optimization method was able to reduce structure motion at a reduced actuation force when compared to previous optimization methods. Yan and Yam (2002) applied GA to select actuator locations in a 72-bar space truss
for vibration control. The results show that the disturbance acting on a structure is a key factor in determining the optimal number and locations of actuators in active structural vibration control, and a global and efficient optimization solution of multiple actuator locations can be obtained using the GAs. Martinez-Rodrigo and Romero (2003) developed linear and non-linear performance indices to be used to evaluate viscous damper retrofit strategies and select the best strategy. The non-linear optimization method reduced the maximum damper force by 35% when compared to the linear method.

Dargush and Sant (2005) presented a GA for optimizing retrofitting type and location for structures with vertical stiffness irregularities. The proposed method was able to design robust damper system for a three different structures. Lavan and Levy (2005) used a gradient based method to determine the optimized damping coefficients of a 10-storey elastic structure with dampers placed on every floor. The optimized design yielded maximum drifts that were smaller than or equal to the allowable drift. Perez and Behdinan (2007) applied a particle swarm optimization method to three benchmark truss design problems. The proposed particle swarm method resulting is similar or better results when compared to other optimization methods. Apostolakis and Dargush (2009) used a GA to select placement of passive dampers in a non-linear three-storey steel moment-resisting frame structure subject to seismic time-history analysis. Sung and Su (2010) augmented the GA method with the use of fuzzy logic control to adapt the GA. An optimized design was successfully generated from the proposed method. Bigdeli et al. (2012) applied a variety of optimization methods to select optimal damper locations for the pounding problem in two adjacent structures. A retrofit insertion method was shown to be the most efficient and reliable method, especially for tall structures. A summary of the presented works is shown in Table 4.1.
### Table 4.1: Summary of optimization applications in structural analysis

<table>
<thead>
<tr>
<th>Reference</th>
<th>System</th>
<th>Goal</th>
<th>Optimization Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rajeev et al. (1992)</td>
<td>Steel Truss</td>
<td>Member weight†</td>
<td>Genetic Algorithm</td>
</tr>
<tr>
<td>Agrawal and Yang (1999)</td>
<td>Shear frame</td>
<td>Multiple†</td>
<td>Sequential Search, others</td>
</tr>
<tr>
<td>Ganzerli et al. (2000)</td>
<td>RC frame</td>
<td>Material cost†</td>
<td>D.O.T software</td>
</tr>
<tr>
<td>Abdullah et al. (2001)</td>
<td>Elastic MRF</td>
<td>Cost†</td>
<td>GA and Gradient-based</td>
</tr>
<tr>
<td>Li et al. (2001)</td>
<td>Shear frame</td>
<td>No. of actuators, control cost, acceleration†</td>
<td>Genetic Algorithm</td>
</tr>
<tr>
<td>Nanakorn et al. (2001)</td>
<td>Steel Frame</td>
<td>Member weight†</td>
<td>Adaptive Genetic Algorithm</td>
</tr>
<tr>
<td>Yan and Yam (2002)</td>
<td>Space truss</td>
<td>Vibrations and control energy†</td>
<td>Genetic Algorithm</td>
</tr>
<tr>
<td>Liu et al. (2003)</td>
<td>Steel Frame</td>
<td>ISD and material costs†</td>
<td>Genetic Algorithm</td>
</tr>
<tr>
<td>Martinez-Rodrigo and Romero (2003)</td>
<td>Steel MRF</td>
<td>Actuator forces†</td>
<td>Equal energy disipation</td>
</tr>
<tr>
<td>Chan and Zou (2004)</td>
<td>RC frame</td>
<td>Material cost†</td>
<td>Optimality Criterion</td>
</tr>
<tr>
<td>Park et al. (2004)</td>
<td>Shear frame</td>
<td>Damper size and brace stiffness†</td>
<td>Sequential quadratic</td>
</tr>
<tr>
<td>Dargush and Sant (2005)</td>
<td>Steel Frame</td>
<td>Utility function‡</td>
<td>Genetic Algorithm</td>
</tr>
<tr>
<td>Lavan and Levy (2005)</td>
<td>RC frame</td>
<td>Damping strength†</td>
<td>Cutting planes method</td>
</tr>
<tr>
<td>Alimoradi et al. (2007)</td>
<td>Steel Frame</td>
<td>ISD and weight†</td>
<td>Pareto Front</td>
</tr>
<tr>
<td>Zou et al. (2007)</td>
<td>RC Frame</td>
<td>Initial cost and loss‡</td>
<td>ε-constraint</td>
</tr>
<tr>
<td>Perez and Behdinan (2007)</td>
<td>Steel Truss</td>
<td>Member weight†</td>
<td>Particle Swarm</td>
</tr>
<tr>
<td>Fragiadakis and Papadrakakis (2008)</td>
<td>RC frame</td>
<td>Direct cost†</td>
<td>Evolution Strategies</td>
</tr>
<tr>
<td>Apostolakis and Dargush (2009)</td>
<td>Steel MRF</td>
<td>Relative Performance Index‡</td>
<td>Genetic Algorithm</td>
</tr>
<tr>
<td>Liu (2010)</td>
<td>Steel Frame</td>
<td>Member weight†</td>
<td>Genetic Algorithm</td>
</tr>
<tr>
<td>Sung and Su (2010)</td>
<td>Reinforced concrete piers</td>
<td>Cost and difference from target performance†</td>
<td>Fuzzy logic Genetic Algorithm</td>
</tr>
<tr>
<td>Bigdeli et al. (2012)</td>
<td>Adjacent Elastic Structures</td>
<td>Inter-storey drift†</td>
<td>Exhaustive search, Greedy, Max. Velocity location, Genetic Algorithm</td>
</tr>
</tbody>
</table>

†Minimize; ‡Maximize;

#### 4.3.1 Genetic Algorithm

GA is a commonly used optimization technique used to solve a wide variety of engineering problems (e.g. Rajeev and Krishnamoorthy, 1992; Abdullah et al. 2001; Burns 2002; Yan and Yam 2002; Apostolakis and Dargush 2009). This method is based on the behaviour of natural evolution. A general description of GA is given below:

- Randomly generate an initial set of possible solutions.
• Test each solution determine the objective function score.
• Rank solutions based on the objective function.
• Combine and modify the most successful solutions in a semi-random manner to create a new set of solutions.
• Test the new set of solutions and repeat the process.

This method has been used, partly, because of its ability to provide an optimized solution in a relatively short computational time. Due to the randomized nature of the GA, there is no guarantee that a global, or even local, optimum solution will ever be found (Burns 2002). While GA is not guaranteed to find an optimum solution, the ability to provide a much improved solution in a reasonable amount of time is the main benefit of the GA. For the problem of retrofit location, the possible locations can be coded into a binary string with each variable in the string representing a single location.

GA has many parameters that must be selected such as population size, generation size, mutation probability, and combination and generation methods. Goldberg et al. (1991) and Burns (2002) provide discussion on the appropriate population and generation sizes. The choice of parameters greatly impacts the number of simulations required and how quickly an optimal solution is selected.

4.3.2 Greedy Algorithm

A greedy algorithm is a simple optimization algorithm that makes the locally optimal decision at any given step. This method has been used for simplified systems similar to retrofit location selection problems (Agrawal and Yang, 1999; Bigdeli et al. 2011). For the specific retrofit location optimization problem, with \( n \) retrofit locations and \( n_r \) maximum retrofits, a greedy algorithm would proceed as follows:

• Given the base test structure, the \( n \) locations are tested with a single retrofit in each.
• The \( n \) test retrofit locations are scored and ranked.
• A retrofit is placed at the best location and this new structure is taken as the starting point.
• The process is repeated until a given goal is reached such as: structure performance, total cost, or maximum retrofit limit.

This method provides a much more systematic and deterministic approach to finding an optimized solution as opposed to the randomness inherent in the genetic algorithm. Given this deterministic nature, the algorithm is guaranteed to find a locally optimal solution as the main stopping case occurs when all possible results of the next retrofit placement are less optimal than the current results. A disadvantage of the structured nature of algorithm is that the starting conditions of the simulation heavily influence the results. The starting case of a bare structure defines the result that will occur when no other parameters of the simulation are changed. An advantage, in the simplest case, is that there are no optimization method parameters to select or adjust. Due to the structured nature of this method, the total number of simulations required \((N_{\text{sim}})\) can be determined. The maximum number of simulations can be calculated as shown in equation (4.2).

\[
N_{\text{sim}} = n + (n-1) + \ldots + (n-(n_r-1))
= nn_r + n_r - \left(\frac{n_r^2 + n_r}{2}\right)
\]  

(4.2)

4.4 Illustrative Example

This illustrative example uses the same structure model as presented in section 3.2. This example uses the mainshock earthquake records presented in Chapter 3 and extends the system to include the aforementioned optimization methods.

4.4.1 Common Experimental Procedure

In this study, the structure was simplified for simulation to eliminate a number of possible deficiency categories. The two-dimensional nature of the model eliminates much of the diaphragm effects and the element connections and structure foundations are not modeled. Neglect of the diaphragm and connections alters the interaction of the beams with the columns and the load transfer mechanisms of the building as a whole, possibly reducing the
stiffness of the floor system and increasing the local differences in the columns behavior within a storey. Based on the remaining deficiencies of global strength and stiffness, configuration, and load path, a common chevron bracing configuration as described in FEMA 547 was chosen as the desired retrofit option for placement optimization. This retrofit type was chosen as it addresses the global deficiency concerns by adding force-resisting elements to the structure and can address configuration and load path deficiencies with appropriate location selection. Additionally, the chevron bracing allows a doorway through the braced bay which is more important for the architectural considerations for the intended use of the hotel structure that is simulated in this thesis.

For the braced frame members, steel wide-flange sections were chosen. Generally, steel members are easier to apply as retrofit for an RC structure as they can be directly bolted to the existing structure with much less work than would be required to properly integrate new RC elements. For this study, a CISC standard W310×74 section was chosen as the braced frame member throughout the structure. As the cost of steel is a significant factor in the design, the size of the members is often adjusted depending on the force in the members, typically reducing in size when the member is placed higher in the structure. Varying the size of steel sections would have added an additional set of variables to the optimization procedure and so it was not performed to reduce complexity and computation time.

The example problem in this section provided different options for the implementation of parallel computing techniques to improve the overall computation time. The OpenSees software is able to assign portions of the structure model to different computational threads, allowing large structures to be simulated more quickly. As the model constructed for this test was relatively small, this option was not selected. The optimization methods chosen require evaluation of multiple retrofit locations for determination and ranking of score. In this case, the method of information transfer between the optimization method in MATLAB and the structure simulation in OpenSees meant there could have been conflicts when multiple simulations would attempt to read or write data at the same time and thus this option was not selected. The multiple earthquake records that are used to test the simulation model provided another means of parallelization. The developed model was tested with each earthquake
record in a separate parallel thread. Each thread wrote the results of each record simulation and the MALTAB optimization script was able to combine the results for a more robust objective function score. This method was selected as it was able to be implemented more easily with the previously developed simulation and it avoided the challenges of the other parallelization options. Additionally, the reduction in computation time was still significant for this method.

To begin the analysis, the structure model and scaled mainshock records used in the previous section were further developed to incorporate parallel computation methods. As an initial test, the base structure with no applied retrofits was tested with the mainshock records to provide a base performance level for comparison. A first objective function was developed for use by the optimization methods. The Greedy algorithm and GA were implemented in MATLAB to communicate with the OpenSees software to analyze the structures. After analysis of the results from the first objective function, a second objective function was developed to address the issues presented by the results. The second objective function was implemented with the optimization methods and the results were analyzed.

4.4.2 Case 1: Proportional Performance Measure

For this case, an objective function was selected that incorporated both the structure performance and retrofit cost parameters as ratios. The objective function (see equation (4.3)) quantifies the performance by comparing the maximum inter-storey drift ratio (MISDR), $d_{\text{max}}$ (%), to a defined required performance level. Both the MISDR and number of retrofit variables represent competing factors within the objective function. The variables were normalized to values representing a practical maximum. This normalization has the effect of weighting the two competing variables differently. The performance is averaged over a set of earthquake records selected from FEMA P695 that are numbered from 1 to $n_{\text{EQ}}$. The ‘life safety’ and ‘collapse prevention’ performance levels ($P_{\text{LS}}$ and $P_{\text{CP}}$, respectively) correspond to a MISDR of 1.5% and 2.5%, respectively, and if the drift is beyond these limits, the objective function is penalized. The penalty factors shown in equations (4.4), and (4.5) were selected based on the FEMA 356 performance-based design principles. The number of
applied retrofits, \( (n_r) \), is used as a proportional cost measure, comparing number of applied retrofits to a defined maximum number.

\[
\min F = \left( \frac{\sum_{i=1}^{n_{EQ}} d_{\max,i}}{n_{EQ}} \right) \cdot \frac{n_r}{2.5} \cdot p_{LS} \cdot p_{CP}
\]

\[
p_{LS} = \begin{cases} 
1, & \text{if } d_{\max} < 1.5 \\
1.5, & \text{if } d_{\max} \geq 1.5 
\end{cases}
\]

\[
p_{CP} = \begin{cases} 
1, & \text{if } d_{\max} < 2.5 \\
2, & \text{if } d_{\max} \geq 2.5 
\end{cases}
\]

As a baseline study, the structure with no applied retrofits was simulated. This resulted in the max ISDR results shown in Figure 4.1(a) and an objective function value \( F = 1.16 \). Based on the MISDR results of 1.9\%, the structure is in the ‘Life Safety’ performance category (i.e. 1.5\% < MISDR < 2.5\%). These numbers are provided as a means of comparison to the following results.
The optimized solution produced by the GA is shown in Figure 4.2. This solution contains 18 retrofitted locations, significantly more than the greedy algorithm, after performing 900 (30 individuals over 30 generations) function evaluations during the algorithm. This solution resulted in the max ISDR results shown in Figure 4.1(b) and an objective function value of 0.48. The improvements in structure MISDR are consistent with the expected behaviour given the GA optimized retrofit layout. The retrofits are distributed regularly through the whole structure, with two or three retrofits per floor. This layout improves the strength and stiffness of every floor, and thus the MISDR of each floor is significantly reduced.
Figure 4.2: Van Nuys hotel schematic with Greedy Algorithm (Solid Lines) and Genetic Algorithm (Dashed Lines) optimized retrofit schemes with proportional objective function

Figure 4.3(a) presents the objective function value of the best solution for each generation. The improvement profile is similar to most GA results where significant improvement occurs in early generations followed by much slower (or in this case, no) progress in the later generations. While the GA calculated the maximum 30 generations, the best scoring solution did not improve past the 14th generation.

Figure 4.3: (a) Genetic Algorithm and (b) Greedy Algorithm improvement at each iteration
It is also interesting to note how the results of the genetic algorithm correspond to the established engineering knowledge. In the third bay from the right of Figure 4.2, the applied retrofits are nearly vertically continuous; this matches the idea that vertically continuous retrofits limit any stiffness irregularities and reduces the vulnerability of the structure (FEMA 356). The GA optimized result contains similar numbers of retrofits on each storey. This layout does not conform to the general understanding that fewer retrofits are needed at the upper storeys due to reduced shear and axial loads, given the structure does not have significant irregularities past the first storey. Increasing the number of generations in the algorithm could increase the chance that these complex factors are accounted for in the final solution. Alternatively, this can also be introduced as one of the constraints.

The GA solution can be compared to some standard retrofit schemes to see the impact of using this particular algorithm. The retrofit schemes and the results are shown in Figure 4.4. The figure shows the MISDR for all earthquake records and the objective function value for each retrofit configuration. The central bay retrofit scheme provides strong improvement over the base structure, reducing the drift to meet the ‘immediate occupancy’ (i.e. 0.2% < MISDR < 0.5%) performance objective and the lowest objective function value of 0.44. The exterior bay retrofit scheme improved the MISDR over the central bay scheme but required twice as many retrofits to do so, resulting in a higher objective function value of 0.56. A retrofit scheme was also applied that combined the central and exterior bay schemes, resulting in a structure that is almost entirely retrofitted. This full retrofit scheme resulted in significantly reduced MISDR for all earthquake records and reaching the ‘fully operational’ (i.e. MISDR < 0.2%) performance objective. Due to the large number of retrofits required to reach this level of performance, the objective function value is the highest of the five schemes shown. The scheme generated by the GA is similar to the central bay scheme but with slightly lower MISDR due to slightly more retrofit used.
During initial optimization procedures, the greedy algorithm completed with only two retrofit locations, resulting in an objective function value $F = 0.46$ and an average MISDR $= 1.08\%$. Upon investigation, it was clear the algorithm had stopped at a local minimum. The algorithm was modified by removing the stopping case when no improved solution could be found in the next step. This allowed the algorithm to continue past the local minimum and possibly find a better solution with subsequent retrofit placements up to a stopping case defined by the maximum number of retrofit placed. The algorithm was allowed to proceed to 20 placed retrofits (930 function evaluations). The best solution that had been found is shown in Figure 4.2. While the modified algorithm searched beyond the original local minimum, it was unable to find a solution better than the result with two applied retrofits. Compared to the baseline structure, this is a significant improvement as it was able to bring the structure below the life safety performance limit (i.e. MISDR $<1.5\%$) with a minimum number of retrofit locations. Figure 4.3(b) shows the improvement of the objective function value as subsequent retrofits are placed. Additionally, the MISDR improvements shown in Figure
4.1(c) can be compared to the Greedy optimized retrofit layout. The two applied retrofits were placed on the two storeys that had the highest MISDR in the base structure and thus significant improvements in the MISDR were shown for those two storeys. The retrofit placement on the most critical storeys is intuitive as those locations should be addressed first, before assessing less critical locations. It is also interesting that the MISDR of the storeys adjacent to the retrofit storeys showed some reduction in MISDR as well.

Implementing a simple algorithm like the greedy algorithm has a drawback of also having simple stopping cases. In this case, the algorithm initially stopped after it failed to find any immediate improvement. Additional retrofits did not improve the objective function for several additional retrofit placements. This simple greedy algorithm implementation, while reasonably effective in this case, could miss a better solution because it quickly cuts-off any branching searches other than the most immediate and best current solution, resulting in a single sequence of retrofit placements.

For both optimization algorithms, the results presented varied due to the process of the algorithm and the interaction with the objective function. The MISDR performance results of the two algorithms were logically apparent given the optimized retrofit location schemes that were generated. While the objective function values resulting from both optimization methods were similar, the resulting retrofit layouts were very different. The two sets of results have similar objective function values but with widely varying numbers of retrofits placed. This highlights the importance of selecting an objective function that accurately represents the goal of the structural optimization.

**4.4.3 Case 2: Cost-Estimate Performance Measure**

For the second case, the goal was to determine a performance measure that improved upon the scoring method of the proportional performance objective function case. In the proportional scoring, the ISD and retrofit number are normalized to somewhat arbitrary scales that were set in an attempt to balance the requirements of reducing both the MISDR and the number of retrofit locations. This normalization has a significant impact on the
relative impact of one variable over the other. To counter this problem, a scale common to both competing factors was needed that could directly compare the MISD and number of retrofits. The developed solution was an objective function that incorporated both the structure performance and applied retrofit parameters as cost estimates.

The cost due to structure damage \((C_S)\) was estimated from the work of Krawinkler (2005) and correlated to the previously defined performance levels, resulting in the values shown in Table 4.2. Values not shown in the table were linearly interpolated from the defined points. This cost parameter influences the objective function in favour of reduced damage to the structure.

The cost of each retrofit was based on an approximate material and installation cost of $8 per pound of steel (Patterson, 2013). This was a high estimate based on the type of structure and the assumption that no significant foundation modification would be necessary and was applied to a retrofit scheme based on W310×74 beams applied to in a chevron configuration. The cost of each retrofit was multiplied by 4 to bring the estimate for the 2D single-frame simulation into agreement with the cost estimate of the whole four-frame structure. The resulting cost of each retrofitted bay was $44800. When this value is multiplied for each retrofit in the structure \((n_r)\), the total cost of retrofit \((C_R)\) is given by equation (4.6). This cost parameter influences the objective function in favour of reducing the amount of retrofit locations in the structure. The two competing cost values are then added together to provide the objective function value for the optimization methods. This is shown in equation (4.7).
Table 4.2: Building loss estimates

<table>
<thead>
<tr>
<th>Performance level</th>
<th>MISDR</th>
<th>Cost Estimate, $C_S \ ( \text{ $ Million})$</th>
</tr>
</thead>
<tbody>
<tr>
<td>No damage</td>
<td>0.0%</td>
<td>0</td>
</tr>
<tr>
<td>Fully operational</td>
<td>0.2%</td>
<td>2</td>
</tr>
<tr>
<td>Immediate occupancy</td>
<td>0.5%</td>
<td>4</td>
</tr>
<tr>
<td>Lift safety</td>
<td>1.5%</td>
<td>6</td>
</tr>
<tr>
<td>Collapse Prevention</td>
<td>&lt;2.5%</td>
<td>6.5</td>
</tr>
<tr>
<td>Collapse</td>
<td>&gt;2.5%</td>
<td>9</td>
</tr>
</tbody>
</table>

\[ C_S = (n_r \cdot 44800) \] (4.6)

\[ \min F = (C_S + C_R) \] (4.7)

In this case, the penalty factors that were present in case 1 were not included as the costs more directly reflect the combined performance and cost of the structure. The structure cost and retrofit costs were weighted equally but the weighting could be altered to represent different relative values. Additionally, placing hard limits on the either of the variables would reflect the available resource limits. To further improve the second test case, the Greedy Algorithm was modified to a slightly more realistic method. In case 1, when the \( n^{th} \) retrofit placement resulted in multiple retrofits of the same performance value, the algorithm would place the retrofit in the first available retrofit location to continue with the analysis. Due to the data structure used in the simulation and optimization methods, the retrofit locations were enumerated beginning from the roof. In Case 2, this was changed so that when resulting in equivalent performance retrofit locations, locations closer to the ground would be chosen.

A set of results similar to Case 1 above are used to illustrate the effect that the modified performance measure had on the results. Figure 4.5 presents the progression of the objective function value from the start of the optimization method to termination with the GA results shown in Figure 4.5 (a) and the Greedy Algorithm results shown in Figure 4.5 (b).
The Greedy algorithm was able to improve the building from MISDR = 1.9% in the baseline study, and MISDR = 1.1% in the Case 1 study to an MISDR = 0.51%, correlating to nearly an immediate occupancy performance level. This optimized retrofit scheme is shown in Figure 4.6. In this case the optimization method did not encounter a local minimum as early in the process and was able to continue to find improved solutions.

The Genetic Algorithm was again allowed to complete the maximum number of 900 function evaluations. The GA was able to improve the building from MISDR = 1.9% in the baseline study, and MISDR = 0.4% in the Case 1 study to an MISDR = 0.13%, correlating to a fully operational performance level. Case 2 resulted in 31 retrofit locations (as shown in Figure 4.6) that are very different from Case 1, but the GA again steadily found improved retrofit schemes as the method proceeded.

The resulting retrofit location schemes for Genetic algorithm and Greedy optimization are shown in Figure 4.6 and Figure 4.7, respectively. Additionally, the results of both optimization methods compared to a selection of three standard retrofit schemes in Figure

Figure 4.5: (a) Genetic Algorithm and (b) Greedy Algorithm improvement at each iteration for the cost-estimate objective function
4.8. The results of all the schemes include both the MISDR of each earthquake record and the resulting cost-performance objective function value. The changes to the objective function have resulted in clear differences in the correlation of MISDR and the performance measure when compared to Case 1. These results show the GA result has the best objective function value while achieving similar or better MISDR results as the other retrofit schemes.

Figure 4.6: Van Nuys hotel schematic of genetic algorithm optimized retrofit schemes from cost-based objective function
Figure 4.7: Van Nuys hotel schematic of greedy algorithm optimized retrofit schemes from cost-based objective function
In this chapter, the building presented in Chapter 3 was used as a platform to demonstrate two methods to optimize retrofit locations. Both optimization methods were implemented with a simplified proportional objective function and with a more sophisticated cost-estimate objective function. The results of both methods and objective functions were presented and compared to a selection of standard retrofit schemes.

Figure 4.8: Performance of retrofit schemes with cost-estimate objective function values

4.5 Summary

In this chapter, the building presented in Chapter 3 was used as a platform to demonstrate two methods to optimize retrofit locations. Both optimization methods were implemented with a simplified proportional objective function and with a more sophisticated cost-estimate objective function. The results of both methods and objective functions were presented and compared to a selection of standard retrofit schemes.
5 Chapter: Discussion and Conclusion

In this thesis, a variety of different components and methods are applied. This chapter will focus on some of the challenges and benefits faced in the different components of the study and for the method as a whole.

5.1 Hazard

The first focus of this thesis is the inclusion of aftershock earthquake records in the hazard implementation of the structural simulation. The works presented in Chapter 2 demonstrate the importance of considering aftershocks in a hazard analysis and the results shown in Chapter 3 display the significant effect aftershock events can have on a mainshock-damaged building.

The challenges with aftershock records are similar to the challenges of mainshock record events. The method of selection and scaling of mainshock and aftershock records can alter the relevance the chosen records when applied to a structure. In this study, the records were chosen to be a generalized set of records for analysis as the goal was to demonstrate the effect of including aftershock records. In a case where information is needed for a more specific structure and location, additional effort would be required to select a set of mainshock and aftershock records that would provide results relevant to that location. As a further complication, aftershock records would need to be selected to match the type of earth crust motion to remain representative of a possible future earthquake event. Mainshock and aftershock sequences that meet all of the requirements for a specific location may not be available so alternate methods of creating appropriate sequences may be needed.

5.2 Damage Accumulation

The second focus of this thesis is the effect of aftershock motion on mainshock damaged structures. While it is clear aftershocks will have an effect on the performance of the structure, quantifying the level of impact is a challenge. The residual ISDR and the maximum ISDR (as shown in Figure 3.6) were both increased in the post-aftershock case but the relative increase is an imprecise measure of a change in structure performance.
The damage indices presented in section 3.1.2 provide a more accurate measure of the individual member and whole structure performance but result in trade-offs. Damage indices rely on a damage model to calculate the member performance based on simulation data but the damage model chosen may not reflect all of the potential behavior of the member. More complete damage models are available but require more resources both to record more complex member behavior and to perform more complex calculations based on the expanded information. During a damage assessment simulation, the project scale, building importance and available resources will all impact the decision of which methods to implement.

5.3 Retrofit Location Optimization

The third focus of this thesis is the ability to optimize the retrofit location to reduce costs and increase performance of a structure. To implement an optimization method an appropriate objective should to be selected. The objective function defines what properties of the model will be optimized or scored and, if multiple variables are included, the relative importance of the variables. In this work, two objective functions were chosen to contrast some of the possible performance measures that can be used as a basis for the objective score.

The first objective function, the proportional performance measure, was chosen because it was very simple to implement and is based on data directly from the model’s behavior. The disadvantage of this method was that the ISDR and number of retrofit measures cannot be combined without a selection of a relative importance value. In the first case shown, the scaled measures were weighted but another weight could have easily been chosen, potentially changing the results.

The second objective function, the cost-estimate measure, is more complex but allows a more relevant comparison of performance measures. By incorporating additional information, the ISDR and retrofit measures could be related to cost estimates of loss for the building, and cost for the retrofits. The combined values can be directly compared as they both represent a dollar value in expected costs. The downside is that the additional resources would be needed to determine an estimate for the structure’s cost of damage.
Depending on what information is available and the goals of a given project, different objective functions could be selected to alter the performance of the optimization method. Furthermore, modern multi-objective optimization methods could incorporate the conflicting needs of any stakeholders and decision makers. Multi-objective methods offer a more complete result but require additional information of the conflicting factors and likely would require additional computation time.

With a defined objective function, the selection of an optimization algorithm further alters the resulting retrofit locations. The algorithm interprets the results of the simulation to determine how best to place the retrofits. The two algorithms selected in this work have different features. The Genetic algorithm is well research and generally can find an optimized solution in a reasonable amount of time, however, their random nature of the algorithm means results can vary widely and the optimal solution is not guaranteed. The Greedy algorithm is much simpler to implement and understand, and provides much more consistent results. The determinant algorithms, as opposed to the random algorithms, can be shown to guarantee a solution at a local minimum would be found given enough time. While the optimal solution is important, a nearly optimal solution that can be found much faster may be better depending on the goals of the project. Due to the non-linear discreet-variable nature of structure simulation, the available optimization algorithms are limited, thus trade-offs are often necessary.

5.4 Overall Method

When combining all of the components of the project, certain aspects become important for the overall function of the method. Primarily, the components have been separated into modular components wherever possible. Assuming the method of information transfer between the modules remains consistent, the different components can be altered with equivalent or improved modules.

Customization may still be necessary to produce relevant results depending on the details of the project. The structural model would need to be developed for the building to be studied; the hazard would need to reflect the location of the building; the objective function would
need to reflect the goals of the project; and the optimization method would need to accommodate any constraints on results.

An additional improvement in the overall method was the implementation of multi-threaded computation capabilities for the structure simulation. Time-series seismic simulation of a structure is a process that requires each time-step to be computed before proceeding to the next. Computer hardware can be underutilized in such situations as the bottleneck becomes the simulation method rather than the hardware limitations. When the hardware is underutilized by a single simulation process, additional simulations can be run simultaneously to take full advantage of resources available. This can greatly reduce the time needed to complete a study and complex problems can better scale when additional resources are available. Only in recent years has both multi-threaded hardware and software been readily available, even on consumer-grade hardware. While this has reduced the cost in implementing such a multi-threaded system, a certain level of knowledge is still necessary for proper implementation as the formulation of some problems is not conducive to the advantages provided my multi-threading. Additionally, there can be multiple ways in which to spit the problem onto different processes, which can have varying impacts on performance depending on where the computational bottle-neck occurs. Finally, extra care must be taken to ensure the results of the separated threads are properly integrated before the next step in the process to avoid errors.

5.5 Summary
The different methods implemented in this work were all intended to improve the process to obtain an optimized retrofit location scheme. When comparing alternative methods, it is apparent that the result is often a trade-off between that of accuracy and speed. The time required to calculate the optimal solution of larger and more complex structures becomes prohibitive even though the quality of the result is important.
References


## Appendix A

Table A.1: Column reinforcement Schedule (Adapted from Krawinkler, 2005)

<table>
<thead>
<tr>
<th>Level</th>
<th>Col size</th>
<th>Vert. bars</th>
<th>Ties</th>
<th>Vert. bars</th>
<th>Ties</th>
<th>Vert. bars</th>
<th>Ties</th>
<th>Vert. bars</th>
<th>Ties</th>
<th>Vert. bars</th>
<th>Ties</th>
</tr>
</thead>
<tbody>
<tr>
<td>7th floor</td>
<td>C-13 to C-17, C-21 to C-26</td>
<td>18”x18”</td>
<td>6-#7</td>
<td>#2@12”</td>
<td>6-#7</td>
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<td>6-#7</td>
<td>#2@12”</td>
<td>#2@12”</td>
<td>4-#5</td>
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<tr>
<td></td>
<td>C-11, C-12, C-20</td>
<td>18”x18”</td>
<td>6-#7</td>
<td>#2@12”</td>
<td>6-#7</td>
<td>#2@12”</td>
<td>#2@12”</td>
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<td>#2@12”</td>
<td>#2@12”</td>
<td>4-#5</td>
</tr>
<tr>
<td></td>
<td>C-30 to C-34</td>
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<td>6-#7</td>
<td>#2@12”</td>
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<td>#2@12”</td>
<td>4-#5</td>
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<td>#2@12”</td>
<td>4-#5</td>
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<td>4-#5</td>
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<td>#2@12”</td>
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<td>#2@10”</td>
<td>4-#5</td>
<td>#2@10”</td>
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<td>#2@10”</td>
</tr>
<tr>
<td></td>
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<td>#2@10”</td>
<td>#2@10”</td>
<td>#2@10”</td>
<td>#2@10”</td>
</tr>
</tbody>
</table>

## 7th floor

- Vert. bars: 6-#7, 6-#7, 6-#7, 6-#7, 6-#7, 6-#7, 4-#5, 4-#5
- Ties: #2@12”, #2@12”, #2@12”, #2@12”, #2@12”, #2@12”, #2@10”, #2@10”

## 6th floor

- Vert. bars: 6-#7, 6-#7, 6-#7, 6-#7, 6-#7, 6-#7, 4-#5, 4-#5
- Ties: #2@12”, #2@12”, #2@12”, #2@12”, #2@12”, #2@12”, #2@10”, #2@10”

## 5th floor

- Vert. bars: 6-#7, 6-#8, 6-#7, 6-#7, 6-#7, 6-#7, 4-#5, 4-#5
- Ties: #2@12”, #2@12”, #2@12”, #2@12”, #2@12”, #2@12”, #2@10”, #2@10”

## 4th floor

- Vert. bars: 6-#8, 8-#9, 6-#7, 6-#9, 6-#7, 6-#7, 4-#5, 4-#5
- Ties: #3@12”, #3@12”, #3@12”, #3@12”, #3@12”, #3@12”, #3@10”, #3@10”

## 3rd floor

- Vert. bars: 8-#9, 12-#9, 6-#9, 8-#9, 8-#9, 6-#7, 4-#6, 4-#5
- Ties: #3@12”, #3@12”, #3@12”, #3@12”, #3@12”, #3@12”, #3@10”, #3@10”

## 2nd floor

- Vert. bars: 10-#9, 12-#9, 6-#9, 8-#9, 8-#9, 6-#7, 4-#6, 4-#5
- Ties: #3@12”, #3@12”, #3@12”, #3@12”, #3@12”, #3@12”, #3@10”, #3@10”

## 1st floor

- Col size: 20”x20”, 20”x20”
- Vert. bars: 10-#9, 12-#9, 10-#9, 12-#9, 10-#9, 8-#9, 4-#8, 4-#6
- Ties: #3@12”, #3@12”, #3@12”, #3@12”, #3@12”, #3@12”, #3@10”, #3@10”
**Table A.2: Spandrel beam reinforcement schedule, floors 3 through 7 (Adapted from Krawinkler, 2005)**

<table>
<thead>
<tr>
<th>Beam mark</th>
<th>Width</th>
<th>Height</th>
<th>Gridline</th>
<th>Top bars</th>
<th>Bottom bars</th>
<th>#3 ties</th>
</tr>
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<tbody>
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<td>16&quot;</td>
<td>22-1/2&quot;</td>
<td>1,9</td>
<td>2#7</td>
<td>3@5&quot;, 5@6&quot;, rest at 10&quot;</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
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<td>2#9</td>
<td>6@4&quot;, 5@6&quot;</td>
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<td>16&quot;</td>
<td>22-1/2&quot;</td>
<td>2,8</td>
<td>2#9</td>
<td>3@5&quot;, 5@6&quot; ea end</td>
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</tr>
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<td>16&quot;</td>
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<td>2#8</td>
<td>3#8</td>
<td>3@5&quot;, 5@6&quot; ea end</td>
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**Table A.3: Roof and second-floor spandrel beam reinforcement schedule (Adapted from Krawinkler, 2005)**

<table>
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<th>Beam mark</th>
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<tr>
<td>2FSB-7</td>
<td>16&quot;</td>
<td>30&quot;</td>
<td>2#9</td>
<td>2FSB-3 top bars</td>
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<tr>
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<td>3</td>
<td>2FSB-8 top bars</td>
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<tr>
<td>2FSB-8</td>
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<td>30&quot;</td>
<td>2#9</td>
<td>2#9</td>
<td>2#8</td>
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72