Optimizing Resilience:
Performance Based Assessment of Retrofits for Wood-Frame Housing in San Francisco

by

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Abstract:

Prevalent seismic hazards in the San Francisco Bay Area region require the building stock to be able to withstand frequent ground accelerations while maintaining life-safety standards. Most housing in the City of San Francisco is more than a century old and unable to comply with the increasingly stringent code requirements. Wood-frame multi-unit structures in the City are especially vulnerable to ground movement due to soft-stories located on the first level, which makes structural retrofit a necessary means to safeguard lives and prevent building collapse. This study conducts a performance based seismic assessment to an eight-unit, four-story wood-frame structure typical in San Francisco to appraise the building's response to incremental seismic mitigation strategies. The goal of the research is to optimize retrofit based on minimization of lifetime damage costs due to seismic impacts. The investigation seeks to evaluate performance of existing urban housing through the application of a seismic assessment methodology typically used for new construction or high stake buildings. The performance assessment uses a simplified procedure delineated by Ghisbain,[1] decreasing the computational intensity of the simulations and affording more retrofit iterations for design optimization. Results show that lifetime damage can be significantly reduced for the considered wood-frame building with applied retrofits, and that minimization of lifetime losses corresponds to augmented retrofit costs. The research concludes with a discussion on prescribing retrofits for a building's longevity versus recommendations for short-term solutions that only call for the structure to fall within code-required life-safety limits. Long-term cost implications can vary greatly, which has necessitated this comprehensive evaluation of the wood-frame structure's behavior and lifetime damages such that it can be conveyed to stakeholders and decision-makers for meaningful planning towards the City's resilience.

Thesis Supervisor: Jerome J. Connor
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1 Ghisbain, Pierre. *Seismic Performance Assessment for Structural Optimization* (Cambridge, MIT; Doctoral Thesis, 2013).

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Finally, to my wonderful parents- thank you for being here for me even when you are miles away. This thesis is dedicated to you, and your tenacious faith in me to be able to accomplish anything I have set my mind to.

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SUMMARY

Soft-story housing in the City and County of San Francisco has long been a source of concern for officials and engineers who understand the repercussions of mid- to large- intensity earthquakes on this particular building typology. Wood-frame structures are a quintessential part of the urban fabric in San Francisco, many built over a century ago, and comprising a bulk of the building stock in the City. After policies for retrofitting un-reinforced masonry structures in the Bay Area proved beneficial in reducing seismic risk, multi-unit wood-frame buildings are now the region's top concern as a vulnerable structure. This research investigates the wood-frame building's response to various earthquake scenarios typical in the Bay Area, and also seeks to optimize potential retrofits to safeguard lives and reduce lifetime damage costs of the building.

Realizing the considerable economic impacts that are caused by seismic events, the move toward performance based evaluation of a building has become increasingly valuable for engineers and designers. Using lifetime damage as a measure to quantify a building's performance is a means to be able to communicate risk and risk mitigation to a broader audience. It also provides basis for prescience in undertaking various retrofit recommendations. This research is also a critique on the typical application of performance based design, commonly employed on new or high stake construction only. This trend seeks to exclude a bulk of the City's most vulnerable building types, such as existing residential housing, which have high financial implications for San Francisco.

The research applies a performance based assessment methodology that has been described by Ghisbain to fully understand the impacts of an earthquake to an unretrofitted wood-frame housing structure. This seismic assessment is framed by the Performance Engineering Research Institute's methodology for Performance Based Earthquake Engineering, and has been "linearized" by allowing for certain assumptions which then reduce the runtime of the assessment process. In this way, Ghisbain has found that the accuracy of the linear process is between 50-100% of the more precise non-linear analysis, but with 1000 times the computation power required.[1] Given the speed and relative veracity of Ghisbain's interpretation of performance estimation, this research is able to provide 14 different retrofit cases to then be compared to the base case wood-frame structure.

Results of this study find that incremental improvements to the soft-story can significantly reduce lifetime damage dollar loss of the building, but that a more robust retrofit may be a large percentage of the building's total value. Simple retrofits such as adding shear walls to the weak axis of the soft-story can surely minimize lifetime damage, however, the longevity of the retrofit may be delimited and must be accounted for. In such circumstances, it may well be that a more extensive retrofit option can provide better returns as it does not need frequent replacement or repair as lower-level retrofits may require.

In conclusion, when life-safety parameters are met by retrofits, the next pressing dilemma an engineer is faced with is one of recommending retrofits which account for both the health and cost implications for the structure. Results will show that those retrofits that improve the building's response to seismic action have the highest cost of retrofit, but which will allow for the building's occupiable lifetime to be extended. Only an expansive assessment such as one conducted in this study can provide an engineer with information and language such that the most appropriate retrofit be suggested to homeowners, city officials and decision makers to mitigate large scale risks in the City.

1 Ghisbain, Pierre. Seismic Performance Assessment for Structural Optimization (Cambridge, MIT; Doctoral Thesis, 2013).

01
INTRODUCTION

1.0 Introduction

1.1 Housing in San Francisco

The City and County of San Francisco abuts the Pacific Rim of Fire, which is home to many of the world's active and dormant earthquakes. The Pacific Rim spans from the western American coast to Japan to the Austral-Asia, with fault slips in this region that have recently caused devastating crises in Haiti, Japan and New Zealand. Urban cities within this seismic area are susceptible to high levels of hazards and damage, mainly caused by densification, vulnerable housing stocks, liquefaction and conflagration after an earthquake. San Francisco is home to some of the most disastrous seismic events in the past century, and is especially familiar with discussions of risk and resiliency in the greater Bay Area.

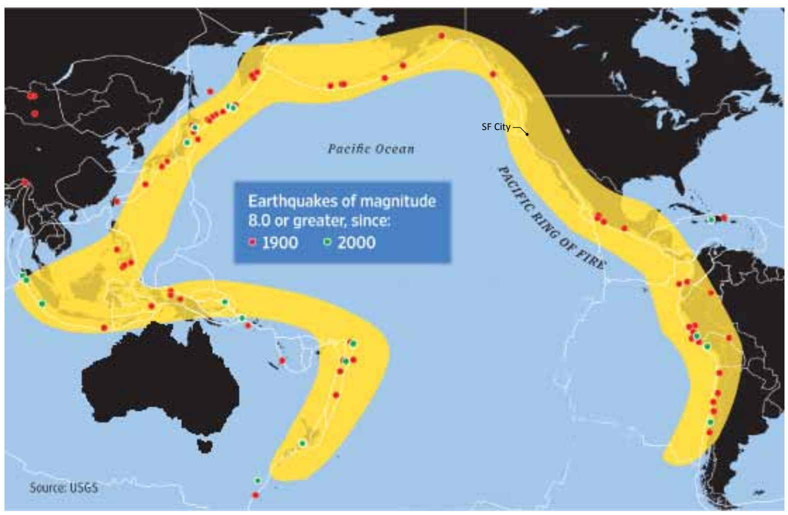

Figure 1.1.1. Pacific Rim of Fire. Source: United States Geological Survey.

San Francisco has approximately 160,000 buildings, ranging from downtown high rises to small, single family homes built over one hundred years ago. The value of these buildings vary based upon the district in which it is located, with residences accounting for 63% of the $190 billion building replacement value in the City.[1] The California Action Plan for Seismic Safety (CAPSS) project of the San Francisco Department of Building Inspection has designated the Applied Technology Council (ATC) to investigate the impacts of several large and overdue seismic events. The reports that these studies have generated have provided significant data on the impacts of city-wide and building damage, on businesses and downtime, and other effects in the San Francisco following a catastrophic event. This becomes increasingly important for engineers and designers who are concerned about the capacities of

1 Applied Technology Council. *Here Today- Here Tomorrow: The Road to Earthquake Resilience in San Francisco; Potential Earthquake Impacts: Technology Documentation 52-1A* (Redwood City, CA; Applied Technology Council, 2010).

buildings in San Francisco that were built much prior to current design code. After intensive field surveys, the ATC has tabulated the number and value of buildings of various structural types in the City of San Francisco. The material, construction and configuration of these buildings attribute to their behavior in an earthquake. The number of individuals within each also becomes a key concern in relation to the building's resilience against such forces. Therefore, wood-frame buildings that account for much of the City's building stock can be further categorized by inherent vulnerabilities, such as the soft-story characteristic of many San Franciscan homes. The table below from the ATC reports details wood-frame and other vulnerable structures in the City.

Structural Type	Estimated Number of Buildings[a]	Estimated Replacement Value of Buildings[b] ($ Billions)
Wood-frame single-family soft-story	60,000	$29
Wood-frame two unit residential soft-story	10,000	$12
Wood-frame three or more unit residential soft-story[c]	13,000	$26
Wood-frame single-family not soft-story	52,000	$24
Wood-frame two unit residential not soft-story	9,000	$10
Wood-frame three or more unit residential not soft-story[c]	6,000	$12
Concrete built before 1980[d]	3,000	$19
Tilt up concrete	200	$0.8
Modern concrete[e]	600	$4
Steel moment and braced frame	1,500	$21
Unreinforced masonry, retrofitted[f]	1,500	$5
Unreinforced masonry, unretrofitted[g]	400	$1
Other[h]	4,200	$27
Total[i]	160,000	$190

a. The numbers of buildings are estimates for 2009 based on available studies and engineering estimates.

b. These figures represent an estimate of the cost to replace or reconstruct a building in 2009. They do not include the value of the land the building sits on or a building's contents. Replacement values are significantly different than real estate prices or assessed valuation. Building value is based on square footage from San Francisco Assessor's Tax Roll, not the estimated number of buildings.

c. The City is currently discussing a program to require evaluation and possible retrofit of residential wood-frame buildings with 3 or more stories and 5 or more residential units. Some but not all of these buildings have a soft-story. There are an estimated 4,400 of these buildings with an estimated replacement value of $14 billion.

d. Concrete built before 1980 includes concrete shear wall buildings and concrete frames with masonry infill walls. The 1980 date was chosen to be consistent with the survey work of the Concrete Coalition (see footnote, next page, for a description of the Concrete Coalition).

e. Modern concrete buildings include concrete moment frame and shear wall buildings built after 1980.

f. This includes buildings retrofitted under the City's program.

g. This includes buildings in the City's retrofit program that have not yet received their certificate of completion, and buildings not included in the City's retrofit program, such as buildings with fewer than five residential units.

h. Other includes steel frame with cast in place concrete walls or masonry infill walls, reinforced masonry buildings, and non-residential wood-frame buildings.

i. Numbers in table have been rounded, which can make totals differ from sum of columns or rows.

Sources: This study, Concrete Coalition, and San Francisco Department of Building Inspection.

Table 1.1.1. Estimated Number and Value of Buildings of Various Structure Types. Table 4, ATC 52-1.

1.2 Vulnerabilities in Housing Stock

Soft-story buildings are described as having a first floor that is significantly weaker or more flexible than the stories above it. The ATC states:

> "The weakness at the ground level usually comes from large openings in perimeter walls, due to garage doors or store windows, and/or few interior partition walls. During strong earthquake shaking, the ground level walls cannot support the stiff and heavy mass of the stories above them as they move back and forth. The ground level walls could shift sideways until the building collapses, crushing the ground floor. This type of weakness, called a soft-story, can be found in many types of buildings. It is common in single-family houses, where the dwelling space sits over a garage, and multi-family buildings, which may have parking or large and open commercial space at the ground level. Corner buildings are believed to have the highest risk, because mid-block buildings are often supported by their neighbor buildings. Soft-stories also occur in commercial buildings constructed from concrete or steel, often with retail space at the ground level and offices above."[2]

This research is mainly interested in understanding behavior and opportunities for improvement for large, wood-frame soft-story buildings. Given that this typology accounts for a significant portion of the City's housing stock, is residence to many of the City's inhabitants, and is greatly impacted by ground motion, it is considered highly vulnerable and warrants immediate actions for retrofit. Figure 1.2.1 is an example of a soft-story wood frame structure common in San Francisco.

Figure 1.2.1. Typical Multi-Family Woodframe Soft-Story Building in San Francisco. Figure 4, ATC 52-1.

Soft-story buildings are commonly found throughout San Francisco, and Figure 1.2.2 shows the distribution of open first story buildings in the City. Research from the ATC has explored MCE events that are commonly projected for the San Francisco Bay Area. These are generally a San Andreas fault and Hayward fault scenario, with varying magnitudes between 6.5 and 7.9. Figure 1.2.3 shows the hazard

2 Applied Technology Council. *Here Today- Here Tomorrow: The Road to Earthquake Resilience in San Francisco; Potential Earthquake Impacts: Technology Documentation 52-1A* (Redwood City, CA; Applied Technology Council, 2010). 8-10.

maps of these scenarios on the aforementioned faults. Comparing this to vulnerable housing locations, one notices the western areas of the City are most susceptible to high intensity seismic events, especially in the Sunset, Ingleside, Exelsor and Merced districts.

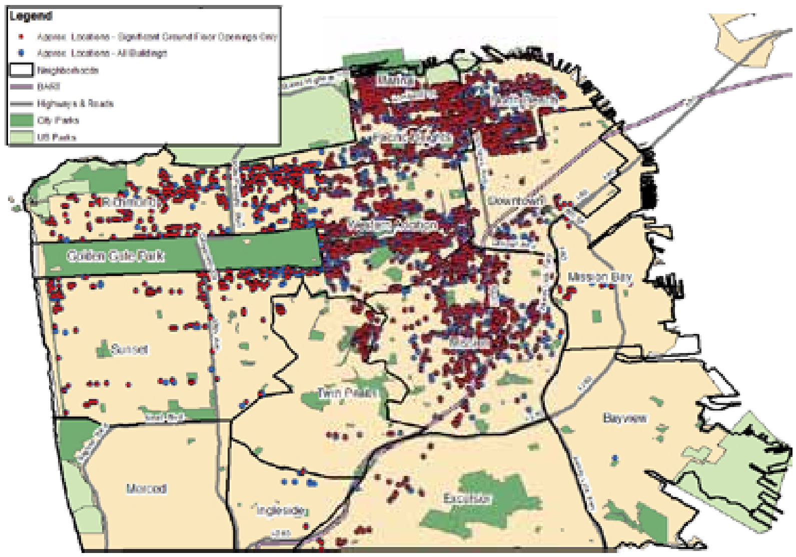

Figure 1.2.2. Wood Frame Buildings with 3+ Stories and 5+ Residential Units. ATC 53-2A.

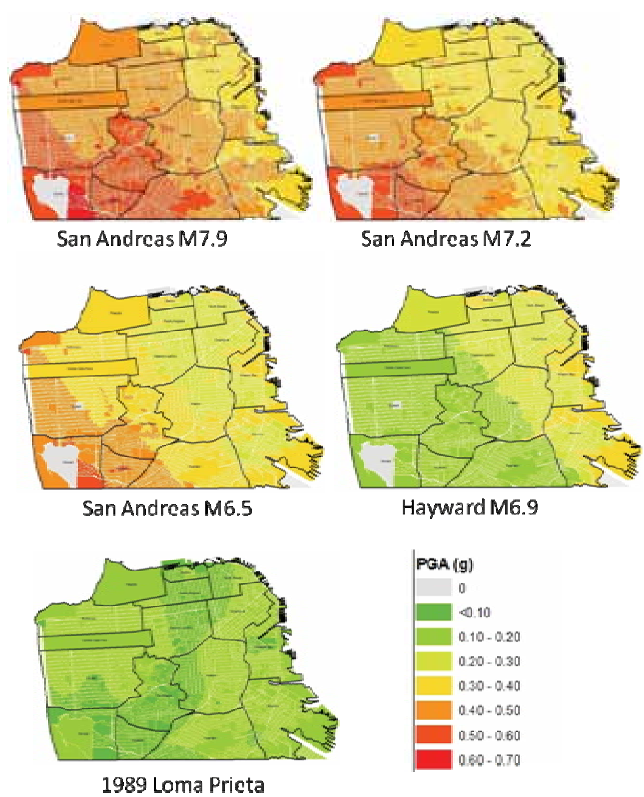

Figure 1.2.3. Hazard Scenarios Analyzed in CAPSS. ATC 52-1.

The Applied Technology Council has found that three or more unit residence woodframe houses will suffer the greatest amount of damage as compared to other building types, averaging about $7.9 billion USD in current dollar amounts. Table 1.2.1 categorizes the potential damages to common buildings in San Francisco and under the scenario earthquakes.

Building Use	Cost of Building Damage in Four Scenario Earthquakes ($ billions)[a]			
	Hayward Magnitude 6.9	San Andreas Magnitude 6.5	San Andreas Magnitude 7.2	San Andreas Magnitude 7.9
Single-family Houses	$2.3	$6.0	$8.8	$13
Two unit residences	$1.4	$2.4	$3.6	$5.4
Three or more unit residences	$4.2	$5.2	$7.8	$12
Other Residences[b]	$0.8	$0.7	$1.3	$2.6
Commercial Buildings	$4.5	$4.2	$6.6	$11
Industrial Buildings	$0.9	$1.0	$1.4	$2.2
Other[c]	$0.1	$0.2	$0.3	$0.7
Total[d]	$14	$20	$30	$48

a. Estimates are in 2009 dollars.
b. Other Residences includes hotels, motels, nursing homes, and temporary lodging.
c. Other includes religious and educational buildings listed in San Francisco Assessor's Tax Roll.
d. Numbers in table have been rounded, which can make totals differ from sum of columns or rows.

Table 1.2.1. Estimated Cost to Repair and Replace Buildings Damaged from Shaking and Ground Failure in Four Scenario Earthquakes. Table 5, ATC 52-1.

1.3 Motivation

Given the extreme hazards in the region and the high susceptibility to building damage, one of the foremost interests in this research is to assess seismic mitigation options for wood-frame structures. In particular, multi-story, wood-frame buildings with soft-stories are of primary concern, as these account for the largest losses and damage costs. In addition, the means to such assessment is equally important for this investigation. In other words, the manner in which a wood-frame building is evaluated for vulnerability and damage greatly affects the types of retrofits that could and should be recommended. Alternatives to traditional code-based design have evolved a performance-based earthquake engineering (PBEE) approach to understanding damage cost and losses from a building or set of buildings. This research seeks to apply an interpretation of the PBEE as described by Ghisbain in

Seismic Performance Assessment for Structural Optimization that simplifies current procedures.[3] In his research, Ghisbain interprets a means to evaluate buildings in a discretized fashion, and finds that more damage is caused to buildings over their lifetime by low intensity, high frequency events rather than high intensity, low occurring events. Figure 1.3.1. is a diagram of the Pacific Earthquake Engineering Research group's description of a performance based approach that is widely accepted, applied, and is an important step away from simple strength-based structural design optimization. Finally, this research seeks to critique the typical applications of Performance Based Earthquake Engineering which tend to focus on strengthening conceptual or new buildings. It is a common practice by many engineering firms and researchers to employ performance assessments on buildings to be designed or for retrofits on high-stake buildings, including hospitals, city halls, and fire stations. However, few research studies provide such performance assessment on existing buildings, especially housing in places of high density. In addition, when such assessments are carried out, they tend to use MCE events as seismic inputs, which do not fully capture the damage induced more frequently on buildings by low magnitude events. A performance based approach is typically probabilistic and can capture the reality of seismic scenarios to understand building damage in its lifetime rather than worst-case scenarios which tend to favor implications of life-safety alone.

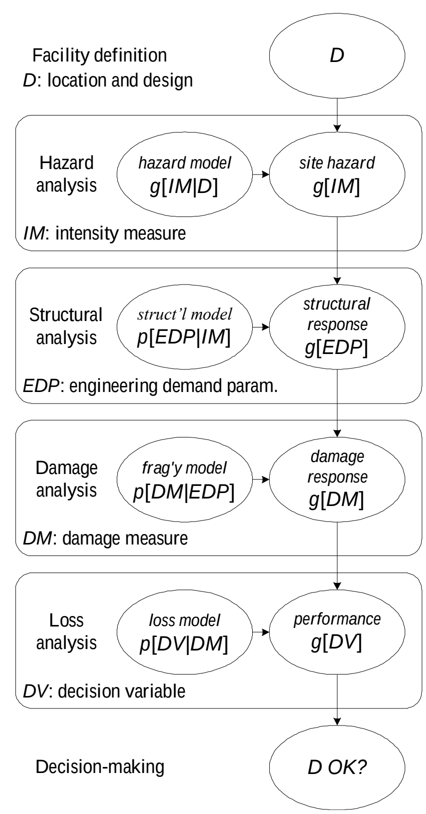

Figure 1.3.1. PEER Performance Based Earthquake Engineering Method. Courtesy, Porter 2003.

3 Ghisbain, Pierre. *Seismic Performance Assessment for Structural Optimization* (Cambridge, MIT; Doctoral Thesis, 2013).

Therefore, this research takes advantage of a streamlined performance assessment approach as defined by Ghisbain to test a typical wood-frame building and 14 applied retrofits to evaluate the costs and benefits of seismic mitigation.

The overall objectives of this research include:

- Conducting performance based assessments to existing multi-story, wood-frame structures in San Francisco using a range of seismic events,
- Providing bottom line estimates of annual damage and lifetime damage costs of the base scenario and with retrofit options applied,
- Applying Ghisbain's strategy for performance assessment such that damages to the overall structure can be parsed to comprehensively understand its behavior and contributions to overall loss,
- Recommending structural retrofits that are able to more than satisfy life-safety requirements, as well as to reduce overall lifetime damage costs to the woodframe structures with soft-stories.

The major goal of this thesis is to investigate the range of vulnerabilities for housing in the San Francisco Bay Area. Since these make up the greatest percentage of San Francisco's building stock, it is critical that they be given due attention in terms of the resiliency against seismic events prevalent in the region. The study hopes to apply powerful assessment and optimization strategies to find appropriate structural solutions for retrofitting such buildings. In doing so, the research also hopes to offer better understanding of how a building's nuanced behaviors can affect overall damage and loss values. Overall, the report seeks to forward the risk and resilience discussions beyond the engineering realm such that the results of this study can be understood by policy makers, home owners, and others that are interested in safeguarding lives and minimizing losses suffered by inevitable seismic events in the Bay Area. The following chapter describes methodologies that are guidelines for assessing a building's performance and provide a basis for the methods of seismic assessment conducted in this thesis.

02

PERFORMANCE BASED EARTHQUAKE ENGINEERING

2.0 Performance Based Earthquake Engineering (PBEE)

2.1 State of Structural Design Practice

In structural design, two important criteria must be satisfied in order that the building can be occupied. These relate to safety and serviceability requirements. Safety relates to the extreme loadings that have a low occurrence probability during a structure's lifetime. Of high concern for safety requirements are potential of collapse of the structure, major damage to the structure and its contents, and mostly, the loss of life. Serviceability describes the moderate loadings that may occur several times during the structure's lifetime. For service loadings, the building should remain fully operational, which means that it should suffer little damage, and the motion experienced by the structure should not exceed specified comfort limits for humans and motion sensitive equipment. In structural terms, the human comfort limit is provided by a restriction on the acceleration, as humans feel uncomfortable when the acceleration reaches about 0.02 g.[1] Safety requirements are generally satisfied by requiring the resistance or strength of the structural elements to be greater than the demand that comes from the extreme loading case. Traditional structural design proportion the structure based on strength requirements, to then establish the needed stiffness properties, and finally to check if the building meets serviceability limits for satisfaction. This approach is referred to as *strength-based design* since elements are proportioned according to strength factors.[2]

Strength-based design has limitations that have provided reason to explore new approaches in assessing a structure's behavior. "The trend towards flexible structures such as tall buildings and longer span horizontal structures has resulted in more structural motion under service loading, which shifts the emphasis from safety toward serviceability. In addition, some new facilities such as micro-structure manufacturing centers have more severe design constraints on motion than the typical civil structure. Advances in material science and engineering have resulted in significant increases in strength of traditional engineering materials such as steel and concrete. However, the lag in material stiffness versus material strength has led to problems in satisfying serviceability requirements on motion parameters. The alternative, which is a subset of performance based design, is motion-based structural design which employs structural motion control methods to deal with motion issues. It is an emerging engineering discipline concerned with the broad range of issues associated with motion of structural systems, which can then be used to understand the building's performance given various criteria such as dollar loss and business interruption", to name a few. [3]

A major critique of current code-based design practice is that codes only account for strength and safety requirements, but do not consider building damage requirements. The emphasis has moved

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1 Connor, J.J. *Introduction to Structural Motion Control* (Prentice Hall, 1st Edition; 2002).
2 *Ibid*
3 *Ibid*

toward performance based design mainly due to "the high cost of seismic retrofit, if done to comply with the prescriptive code requirements developed for design of new buildings, may be too high for economic feasibility and may not provide the performance intended. [In addition,] economic losses [are] higher than expected by owners due to both damage repair costs and business interruption."[4] Current codes provide a *limit states* framework for design, while future directions needed are those that provide reliability based approaches, such as performance based evaluation strategies.

Model codes used in California today are the California Building Code (CBC), International Building Code (IBC) and Uniform Building Code (UBC). The stated purpose of these codes are to provide minimum provisions for design and construction of structures to resist effects of seismic ground motions. In addition, the codes are to "safeguard against major structural failures and loss of life, not to limit damage or maintain function."[5]

The following chart is from the SEAOC Blue Book recommendations, which provide guidelines on prescribed structural performance for seismically active regions. These three tiers of performance criteria are generally ambiguous, since definitions are non-quantitative (for example, *limited damage*, or one *or more* times, etc.). Three tiers exist, but only one design earthquake is considered and provisions are not specifically associated with any particular performance level. The issue in such prescriptions and language is that it leads to a wide variation of interpretation and performance.[6]

	Earthquake Intensity	Frequency of Occurrence	Desired Performance
1	Minor	Several times during service life	No damage to structure or nonstructural contents
2	Moderate	One or more times during service live	Limited damage to nonstructural components and no significant damage to structure
3	Major (Catastrophic) (10% exceedence in 50 years)	Rare and unusual event as large as any experienced in vicinity of site.	No collapse of structure or other damage that would create a life safety hazard.

(After: Lateral Force Recommendations and Commentary, SEAOC.)

Table 2.1.1. SEAOC Blue Book Recommendations.

2.2 Trends Toward Performance Based Earthquake Engineering

One of the first approaches to quantifying performance rather than strength in determining the quality of the structure was through Vision2000. The seminal document provided new concepts

4 Holmes, W.T. *Motivation and Development of PBEE for Existing and New Buildings* (IRCC Workshop, 2006).
5 UBC 1997 ed., Section 1626
6 UC Berkeley. *Lecture: Basic Concepts of PBEE* (CEE 227, UCB, 2003)

that incorporated performance states in the appendices of the SEAOC "Recommended Lateral Force Requirements and Commentary." It focuses on defining what constitutes a frequent, rare or very rare earthquake, and describes in detail the provisions of performance states are which are then categorized by types of events and structures. The basic approach of Vision 2000 is to determine relationships between performance objective, type of facility and probability of earthquakes. It also seeks to relate response parameters to each performance objective to then provide initial acceptance criteria. Vision 2000 categorized performance states in the following manner:

- *Fully Operational*: Continuous service. Negligible structural and non-structural damage.
- *Operational:* Most operations and functions can resume immediately. Structure safe for occupancy. Essential operations protected, non essential operations disrupted. Repair required to restore some non-essential services. Damage is light.
- *Life Safe:* Damage is moderate, but structure remains stable. Selected building systems, features or contents may be protected from damage. Life safety is generally protected. Building may be evacuated following earthquake. Repair possible, but may be economically impractical.
- *Near Collapse:* Damage severe, but structural collapse prevented. Non-structural elements may fall.

The following figure is an iconic schematic of the relationship between performance objectives and earthquake probability. The diagram illustrates that a building would be expected to suffer more damage if it were subjected to a more severe, less frequent earthquake. A more critical building would be expected to have less damage for the same earthquake probability. Although Vision2000 offers provisions in relating engineering response parameters to limit states, the criteria were predominantly based on consensus, rather than on test data or quantitative field observation.

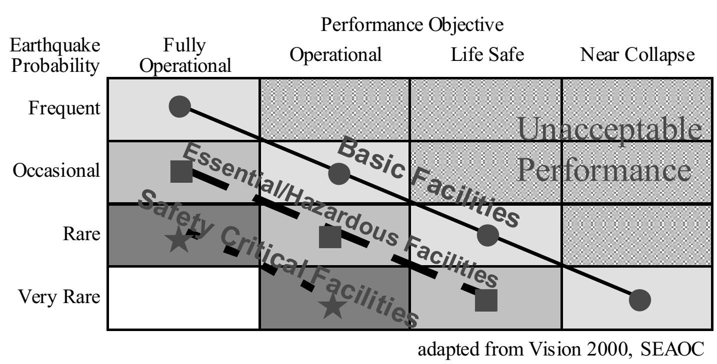

Figure 2.2.1. Vision2000 Schematic Diagram.

In extending the Vision2000 approach, FEMA-273/353 provided guidelines for seismic rehabilitation of buildings. The method suggests four performance goals, including Collapse Prevention, Life Safe, Continued Occupancy and Operational. National seismic hazard maps developed by USGS assist in fully understanding the range of events possible on a site and are included in this method, as is a non-linear dynamic and static pushover method in addition to conventional elastic methods. Finally, FEMA 273/353 uses a displacement based approach with subjective factors to assess uncertainty.

Figure 2.2.2 shows the basic FEMA -HAZUS method, in which probabilities of exceedance of seismic events is related to damage states.

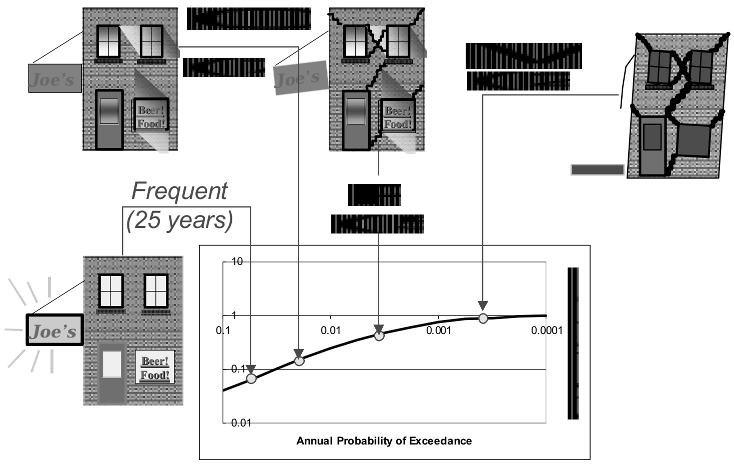

Figure 2.2.2. FEMA Seismic Assessment Methodology.

However, some limitations do exist in the FEMA 356 method. As ground motions are defined in probabilistic terms in this method, the uncertainty and randomness not considered are related to structural demands and capacities. Evaluations are made on a member by member basis, in which case the failure of a few elements might not lead to the failure of the system. Finally, the performance goals are defined in absolute, but subjective terms, for example, whether a structure is either life safe or it is not. Nevertheless, FEMA and Vision2000 took a large step toward Performance Based Earthquake Engineering that have since been evolving for the last decade, changing the way we are able to understand and design structures.

2.3 Performance Based Approach

To address some limitations in FEMA's previous approach, the organization sponsored a performance based design technique called the SAC approach. This approach uses an LRFD-type format to divide demand uncertainty into several parts, including ground motion randomness, structural response, analysis method and modelling. It also related seismic capacity to three main components, including element level effects (stress, plastic hinge rotations), global behavior (drifts, static and dynamic instability) and brittle failure modes (premature column fracture or buckling). This assessment method was further improved by the Pacific Earthquake Engineering Research group, that moved beyond the performance requirements delineated by FEMA and related engineering demand parameters to replacement costs, casualty risk and business downtime in days. The PEER PBEE approach is "aimed at improving decision-making about seismic risk by making the choice of performance goals and the tradeoffs that they entail apparent to facility owners and society at large."[7]

Given the statistical and probabilistic nature of the seismic assessment process, nomenclature becomes increasingly important as various stakeholders must communicate risk to then be able to refine building codes and design. An ideal means of describing the vulnerability or performance of a structure would be as "x% chance of exceeding performance level for an earthquake with a z% probability of occurrence in y years." As such, the ground motion and structure are treated separately, as are the inherent probabilities of the respective quantification. In this way, the performance objective has three parts which include the definition of the performance level, the statement of associated seismic hazard, and a statement of desired confidence. This allows dense engineering and geotechnical data to be communicated with clients and owners in a more effective manner.

Figure 2.3.1 and Figure 2.3.2 illustrate the PEER PBEE approach in terms of the evolution from previous methods and the general procedure for seismic damage analysis.

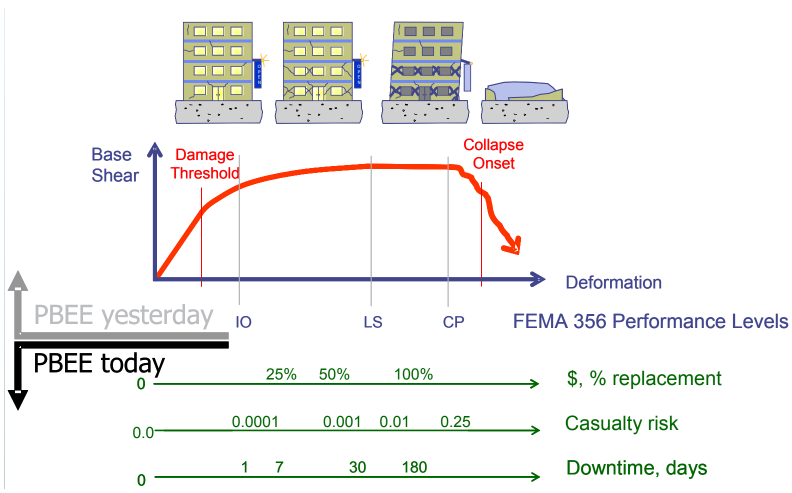

Figure 2.3.1. Evolution of PBEE.

The approach has four levels of assessment, starting with the intensity measure and the effects on particular engineering demand parameters, such as peak drift and floor acceleration. This follows with a damage measure typically conducted via use of fragility functions provided by HAZUS or ATC-58. Fragility functions relate the engineering demand parameter's sensitivity to increasing ground motions, and are described in detail in subsequent chapters. Finally, a decision variable is evaluated, and can include direct financial loss, business downtime and collapse and casualties.

7 Krawinkler, H., Deierlein, G. *Framework for Performance-Based Earthquake Engineering (PBEE)* (PEER, poster).

Figure 2.3.2. PEER PBEE Assessment Procedure.

The next chapter describes an interpretation to traditional Performance Based Earthquake Engineering that has been used as the driving methodology for this research. It is important to emphasize that Performance Based Engineering is a framework that has been delineated over a decade ago, and from which practitioners and researchers have devised interpretations to implement the overarching assessment procedure.

2.4 Communicating Resilience

A key issue in providing new means of assessing a building, such as performance based methodologies, is how those new measures become embedded in codes and the manner in which they are to be implemented. The San Francisco Urban Planning and Research Association (SPUR) has outlined in a document entitled *Building It Right the First Time*[8] an alternative set of performance objectives that allow the general public to understand to what degree their building is safe in a mid- to large-scale earthquake situation. This is to address issues of unclear or complex codes that are provided by engineering research groups or code-designing organizations.

SPUR states that “the level of seismic performance provided by the current building code is not explicitly defined. In general language, the Structural Engineers Association of California[9] states that the intention of the building code with respect to seismic performance is that a building designed to the code should be able to

- ‘Resist a minor level of earthquake ground motion without damage.’
- ‘Resist a moderate level of earthquake ground motion without structural damage, but possibly experience some nonstructural damage.’

8 Maffei, J. *Building It Right the First Time* (San Francisco, SPUR, Degenkolb Engineers, 2009).
9 SEAOC 1999 Blue Book

- ‘Resist a major earthquake of earthquake ground motion...without collapse, but possibly with some structural as well as nonstructural damage.’ “[10]

The language of these performance objectives is rather vague, with words like “possibly” and “some” nonstructural damage. This terminology stems from the inherent randomness and statistical nature of earthquakes, building response and structural damage that are part and parcel of the performance based assessment method. For these reasons, SPUR has created a set of performance goals with nomenclature that is easily understood by most people that are stakeholders in the resilience of the built environment for San Francisco. In the report, SPUR states that “in parallel with improving seismic performance, [the organization] advocates a clearer communication of what seismic performance is expected from each building in our community.” The following is a table from the report that describes these performance standards.

CATEGORY	BUILDINGS
A	**Safe and Operational.** This describes the performance now expected of new essential facilities such as hospitals and emergency operations centers. Buildings will experience only very minor damage and have energy, water, wastewater and telecommunications systems to back up any disruption to the normal utility services.
B	**Safe and usable during repair.** This describes the performance needed for buildings that will be used to shelter in place and for some emergency operations. Buildings will experience damage and disruption to their utility services, but no significant damage to the structural system. They may be occupied without restriction and are expected to receive a green tag after the “expected” earthquake.
C	**Safe and usable after repair.** This describes the current expectation for new, non-essential buildings. Buildings may experience significant structural damage that will require repairs prior to resuming unrestricted occupancy, and therefore are expected to receive a yellow tag after the “expected” earthquake. Time required for repair will likely vary from four months to three years or more.
D	**Safe but not repairable.** This level of performance represents the low end of acceptability for new, non-essential buildings, and is often used as a performance goal for existing buildings undergoing rehabilitation. Buildings may experience extensive structural damage and may be near collapse. Even if repair is technically feasible, it might not be financially justifiable. Many buildings performing at this level are expected to receive a red tag after the “expected” earthquake.
E	**Unsafe.** Partial or complete collapse. Damage that will likely lead to significant casualties in the event of an “expected” earthquake. These are the “killer” buildings that need to be addressed most urgently by new mitigation policies.

Source: SPUR analysis

Table 2.4.1. Seismic Performance Measures for Buildings
Source: SPUR Analysis 2009.

The categorizations not only include the performance of the structure, such as the beams, columns, walls, floors, roofs and foundations, but also on the equipment and systems that are necessary to keep a building usable following a disaster. These systems include water, sewage systems, gas, electricity, fire sprinklers, alarms, elevators, emergency lighting, heating, ventilation, air conditioning, weather-tightness, telephone, internet and others.[11] These performance guidelines are a useful, concise and clear way to disseminate information on building safety regulations and improvements. The guidelines appear in the Porter and Cobeen[12] document, and will be discussed in Chapter 6.

10 Maffei, J. *Building It Right the First Time* (San Francisco, SPUR, Degenkolb Engineers, 2009) 6.

11 *Ibid.*

12 Porter, K. and Kelly Cobeen. *Informing a Retrofit Ordinance: A Soft-Story Case Study* (Structures Congress, ASCE; 2012).

03
METHODOLOGY

3.0 Methodology

3.1 Seismic Performance Assessment for Structural Optimization

As code-based design focuses on life-safety standards, it tends to underrate significant economic impacts caused by building damage from seismic events. Performance based assessments are a step forward in this regard, yet costly simulations create limitations in many probabilistic assessments. Ghisbain states that "a developing trend is to consider damage directly as a measure of seismic performance. In spite of the ability to estimate the cost of future earthquakes, adjusting the investment in seismic upgrades is impeded by the computational requirements of the probabilistic damage assessment."[1] Ghisbain has, therefore, developed a damage assessment tool that uses lifetime seismic damage as the objective function in optimizing structural performance. In his study, Ghisbain varies seismic assessment procedures to analyze estimates on bottom line damage costs and building response, and modifies portions of the procedure to simplify the analysis.

"The runtime of the probabilistic damage assessment is dominated by the response analysis of the structure to a range of earthquake scenarios. [This method considers] alternatives to the standard but expensive nonlinear dynamic analysis, and evaluate[s] the error introduced by the faster analysis methods. The applicability of linear dynamic analysis is further investigated by detailing the effects of structural non-linearities on the lifetime damage assessment."[2] The research finds that the effect of non-linearities are limited in performance based design, in that the building remains nearly elastic when impulsed with low intensity, higher frequency earthquakes than larger magnitude, low frequency earthquakes. In this way, the seismic assessment procedure is "linearized" to simplify computation required, allowing more design iterations to be conducted.

The methods described by Ghisbain will be applied to a multi-family wood-frame buildings in San Francisco. An image of a typical corner building of this type is shown in Figure 3.1.1. These buildings are highly susceptible to damage in seismic events mainly due to their soft-story on the first floor level, as well as their geometry.

The goal of this seismic assessment procedure will be to evaluate various retrofits to such wood-frame structures to better understand lifetime damage costs. These lifetime damage costs can then provide a relative baseline for informed investments to be made if retrofitting is a desired option for building resiliency. Ghisbain argues that an optima exists in specifying an appropriate retrofit, and that traditional designs tend to have increased lifetime costs as do seismic mitigation investments with short-term benefits.

1 Ghisbain, Pierre. *Seismic Performance Assessment for Structural Optimization* (Cambridge, MIT; Doctoral Thesis Abstract, 2013).
2 *Ibid.*

Figure 3.1.1. Multi-Family Wood-Frame Residence, San Francisco. Image courtesy of ABAG.ca.gov

Figure 3.1.2 graphically represents this argument, and has been adapted from Ghisbain[3] A detailed description of Ghisbain's assessment methodology is presented next.

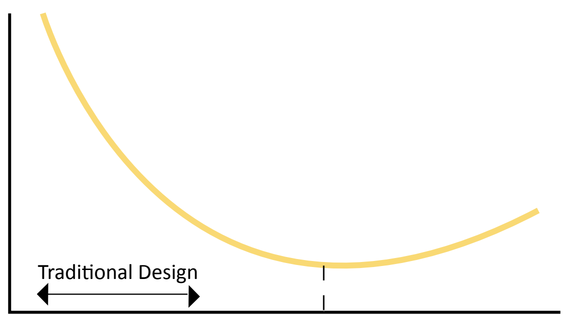

Figure 3.1.2. Optimization of Seismic Mitigation.

3.2 Damage Assessment Procedure

Foremost research on probabilistic seismic assessment began with the Pacific Earthquake Engineering Research (PEER) group at UC Berkeley. They established the following framework for lifetime earthquake damage assessment as denoted in Equation 3.2.1.

$$c_A = v \int_s \int_r \int_l lp(l|r)\, p(r|s) n_A(s) dl\, dr\, ds$$

c_A average annual damage cost ($)
s earthquake intensity; peak ground acceleration (PGA) or spectral acceleration for a particular period and damping ratio
$n_A(s)$ annual exceedance density of the earthquake intensity
r structural response
l loss, or damage (fraction of replacement value)
v replacement value ($)

Equation 3.2.1. PEER Lifetime Damage Assessment.

3 Ghisbain, Pierre. *Seismic Performance Assessment for Structural Optimization* (Cambridge, MIT; Doctoral Thesis Defense, April 26, 2013).

The primary function in producing damage values for seismic loading for this research is described in Equation 3.2.2 by Ghisbain. This is the standard procedure used to assess lifetime damage for the case study wood frame building to be described in Chapter 4.

$$c_A = \int_s \left(\sum_{i,j} v_{i,j} \left(\frac{1}{N_k} \sum_k f_{i,j} \left(\frac{1}{N_{P_i}} \sum_{p \in P_i} r_{jkp} \ (s) \right) \right) \right) n_A \, (s) ds$$

i	building floor
j	damage category (structural, nonstructural drift- and acceleration-sensitive, contents, loss of income, business inventory).
k	earthquake record (ground acceleration recorded in 3 orthogonal directions at a particular location for a particular earthquake). N_k is the number of earthquake records considered.
$v_{i,j}$	value ($) of the system subjected to damage j at floor i.
$r_{jkp}\ (s)$	elementary response function giving the value, as a function of the earthquake intensity, of the structural response parameter governing damage of category j and recorded at a particular point p in the structure subjected to earthquake record k. P_i is the set of points located on floor i.
$f_{i,j}(r)$	fragility function, giving the damage (fraction of replacement value) in system ij (category j, floor i) as a functic of the governing response parameter.

Equation 3.2.2. Lifetime Damage Assessment, Ghisbain.

Seismic performance assessment can be conducted in various manners, PEER happening to the be the archetype for many other forms of performance evaluation. As PEER does not provide guidelines for such assessment, one addition made by Ghisbain is the inclusion of various points on a floor of a building. This becomes especially useful when the simulation uses a 3D model, as is done in this research study, such that each point is able to give a different drift and acceleration estimate. Ghisbain explains further that the "response of the building over earthquake records yields a larger error in the damage assessment, but averaging over floor points has little effects. Considering the average response across a floor before computing damage provides useful hindsight into the behavior of the building and the contribution of each floor to the overall damage."[4]

The following outlines the key steps and assumptions made in conducting the lifetime damage assessment of the wood-frame building described previously. Figure 3.2.1 illustrates the lifetime damage assessment methodology suggested, and should be referred to as the various steps are described. The case study building is further detailed in Chapter 4 in terms of its construction details, geometry and structural behavior. For this chapter, however, the methodology will first be specified.

4 Ghisbain, Pierre. *Seismic Performance Assessment for Structural Optimization* (Cambridge, MIT; Doctoral Thesis, 2013).

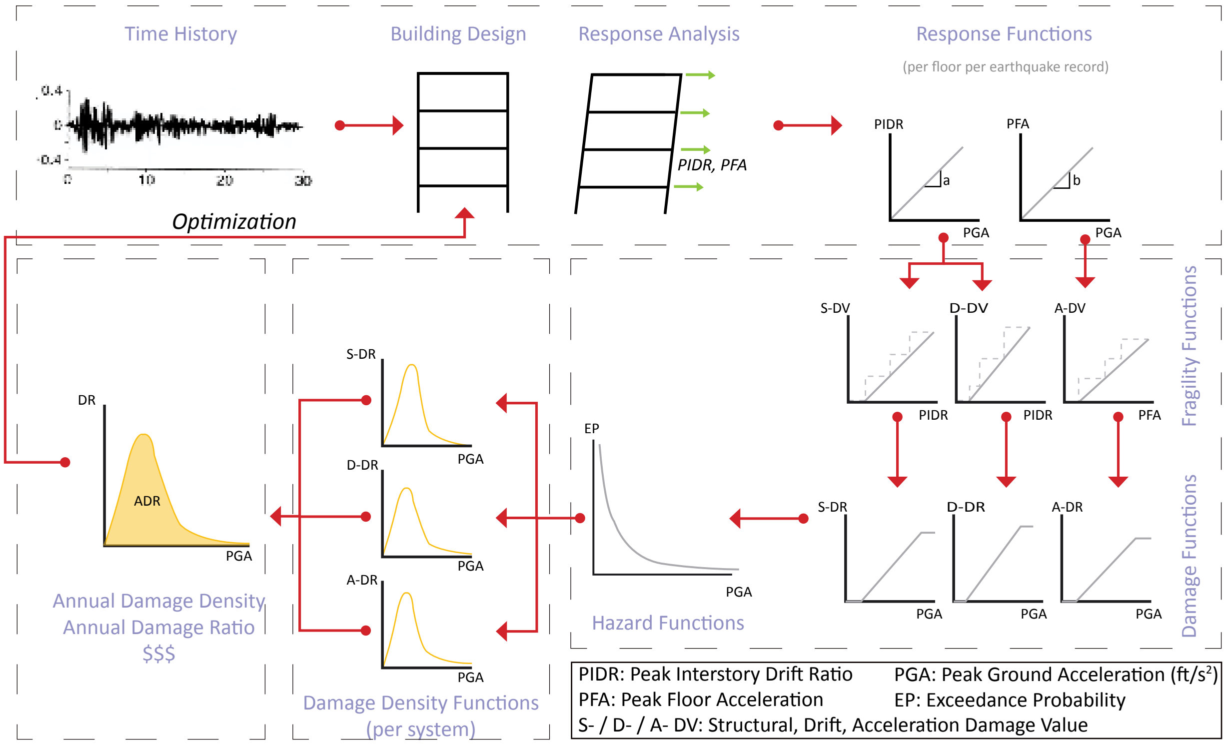

Figure 3.2.1. Lifetime Damage Assessment Procedure.

3.2.1 Seismic Hazards

The first step in the seismic assessment is an understanding of the context of building's location. In seismology terms, this usually relates to ground motion, soil type and liquefaction susceptibility, the former two being of particular concern for this study. Figure 3.2.2 shows how frequency content can be significantly different given the particular earthquake. The location chosen is in the Sunset district of San Francisco (zip code 94116) as this location has a high density of such wood-frame, soft-story buildings. The location of the building determines the probability that the site will be impulsed with a particular intensity earthquake. These are given by hazard curves found through the US Geological Survey (USGS). The seismic hazard curve is the annual occurrence density, which is the average number of earthquakes between a specified range of intensities. Figure 3.2.2 relates the Sunset District to its respective hazard curve.[5]

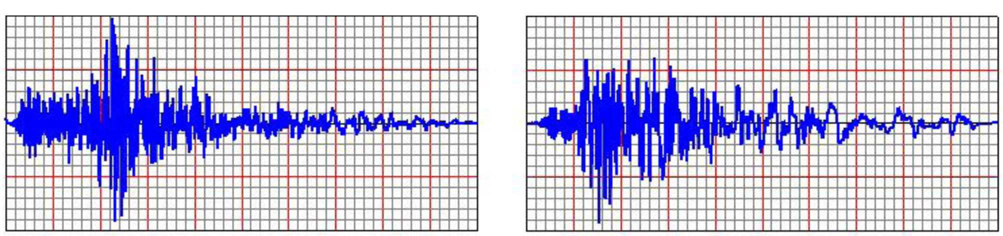

Figure 3.2.2. Accelerogram , Loma Prieta (left) Northridge (right).

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5 Hazard Curve for 94116, USGS.org

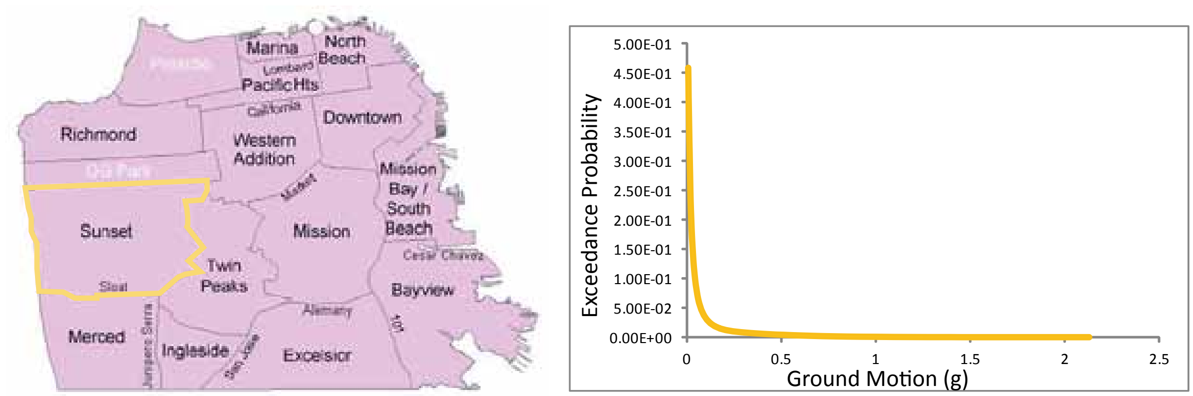


Figure 3.2.3. Sunset District and Respective Hazard Curve, USGS.

The seismic hazard curve will be reintroduced at the end of the end of the assessment procedure.

3.2.2 Response Assessment

The next step is to create a set of functions that describe the response of the structure to a range of earthquake intensities, which are known as the response functions. These are constructed through the process of floor by floor analysis of the structure, given a certain set of earthquake records.

The PEER methodology considers multiple earthquake records because different records scaled to the same intensity may result in significantly different structural responses. Most applications of the methodology in the literature consider sets of 10 to 20 earthquake records. The composition of the set affects the damage estimate, but more consistent results are obtained with larger sets. Ghisbain assessed the coefficient of variation for over 1,000 damage estimates among 131 records, and finds that a minimum set of 16 earthquakes is generally able to capture the variation in frequency content of the ground acceleration similar to that of a larger set of records.[6] This study uses the following earthquake records for the seismic assessment. Note that each record can be thought of as an independent earthquake, therefore, overlaps have been permitted.

The following earthquakes are applied to a model of the case study include the Loma Prieta 1989, M 6.9; Gilroy 2002, M 4.9; Northridge 1994, 6.7 M; Morgan Hill 1984, 6.2 M. Four records for each earthquake are applied in the response study, for a total of 16 seismic records. There is a conscious effort made to not apply the typical MCE-level earthquakes that are commonly used in Bay Area seismic assessment studies, which are a magnitude 7.9, 7.2 San Andreas Fault. This research is interested in looking at mid- to high-range earthquakes that occur more frequently in the region, and that can have significant lifetime damage without ever facing collapse. More information on these earthquakes are provided in the Appendix.

6 Ghisbain, Pierre. *Seismic Performance Assessment for Structural Optimization* (Cambridge, MIT; Doctoral Thesis, 2013).

These time histories are then input into SAP2000, from which information on floor based drift and floor based acceleration are attained. A system experiences damage when the response parameter to which it is sensitive exceeds a threshold value. Therefore, damage is governed by peak interstory drift ratio (PIDR) in drift sensitive systems and by the peak floor acceleration (PFA) in acceleration-sensitive systems. Throughout the response analysis, both peak interstory drift ratio and peak floor acceleration are recorded, producing the system-specific response functions. Three damage systems adapted from HAZUS®[7] are considered in this study. These include structural, non-structural drift sensitive, and non-structural acceleration sensitive. In both structural (S) and non-structural drift (D), peak interstory drift (PIDR) controls, while in non-structural acceleration sensitive systems (A), peak floor acceleration controls. The following table from HAZUS®[8] explains some of the building components that comprise these specific systems.

System Type	Component Description	Drift-Sensitive	Acceleration-Sensitive
Architectural	Nonbearing Walls/Partitions	■	
	Cantilever Elements and Parapets		■
	Exterior Wall Panels	■	
	Veneer and Finishes	■	
	Penthouses	■	
	Racks and Cabinets		■
	Access Floors		■
	Appendages and Ornaments		■
Mechanical and Electrical	General Mechanical (boilers, etc.)		■
	Manufacturing and Process Machinery		■
	Piping Systems		■
	Storage Tanks and Spheres		■
	HVAC Systems (chillers, ductwork, etc.)		■
	Elevators		■
	Trussed Towers		■
	General Electrical (switchgear, ducts, etc.)		■
	Lighting Fixtures		■
Contents	File Cabinets, Bookcases, etc.		■
	Office Equipment and Furnishings		■
	Computer/Communication Equipment		■
	Nonpermanent Manufacturing Equipment		■
	Manufacturing/Storage Inventory		■
	Art and Other Valuable Objects		■

Table 3.2.1. HAZUS® Classification of Drift-Sensitive and Acceleration-Sensitive Nonstructural Components and Building Contents.

7 HAZUS® is a nationally applicable standardized methodology that contains models for estimating potential losses from earthquakes, floods, and hurricanes. FEMA.gov.

8 Department of Homeland Security. *Earthquake Loss Estimation Methodology HAZUS® -MH MR5* (FEMA) 2-6.

The variable for the intensities is peak ground acceleration, or the earthquake intensity. The response analysis determines the behavior of the structure to a set of ground records from which the response function is interpolated. The earthquake variable remains independent in the intermediate functions of the proposed procedure until the very last step, where the occurrence over time of earthquakes of different earthquakes of different intensities is taken into account to estimate lifetime seismic damage.[9] This occurrence over time is depicted as the hazard curve in Figure 3.2.3.

In the driving procedure for the lifetime damage assessment (Equation 3.2.2), it is suggested that multiple points on the floor be analyzed. This is done by considering a floor-specific response function by taking the average of the point-specific response functions over all points of a floor. In this study, only one point on a floor will be used since all floors are assumed to have the same value and contents, as well as having the same occupancy class. Having more points on the floor can allow for a better picture of the building's behavior provided by the floor-specific response functions. In this study, the response of the building is determined by analyzing a point on the corner of the floorplate of each floor, as well as links that represent columns at each corner. These respectively provided information on peak floor acceleration and peak interstory drift ratios. More information on how information has been generated from the SAP2000 model to PFA and PIDR values can be found in the Appendix.

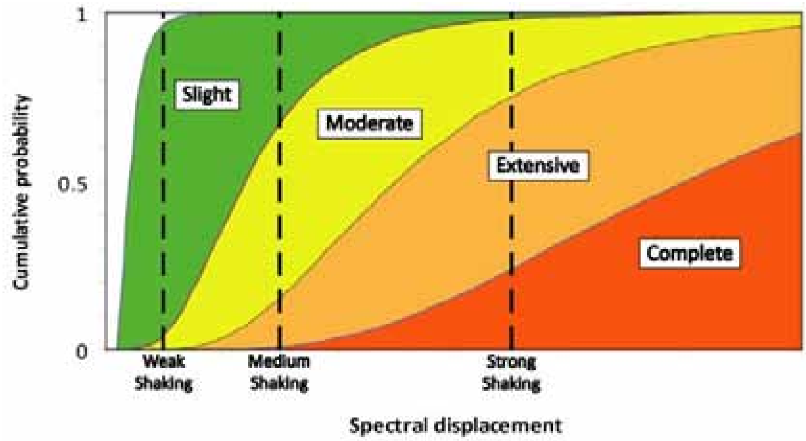
The response functions are found for each floor of every building, and a function is created with a linear relationship between peak interstory drift (PIDR) and PGA, as well as peak floor acceleration (PFA) to PGA. These are the primary functions in correlating building response to building damage, and can be visually illustrated by Figure 3.2.1.

3.2.3 Damage Assessment

The next step in the procedure is to translate the floor based response parameters, peak interstory drift ratio and peak floor acceleration, into damage values. The damage model uses information from FEMA's HAZUS® methodology to convert structural response into building damage via fragility functions. These fragility functions conflate the capacity curves and damage states, and provide the probability of such a state occurring. An example of the capacity curve and description of the damage states are shown in Figure 3.2.4.

Research has found that the capacity curves typically produce low values of damage, that can significantly affect the outcome of the life time loss. For this reason, the author conducted a simple sensitivity analysis by doubling the limits defined by the fragility functions to assess the impacts on total damage. This analysis can be found in the Appendix of the study.

9 Ghisbain, Pierre. *Seismic Performance Assessment for Structural Optimization* (Cambridge, MIT; Doctoral Thesis, 2013).



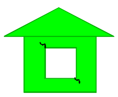
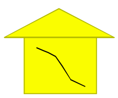
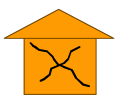
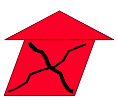
Damage State		Description
	**Slight**	Small plaster cracks at corners of door and window openings and wall-ceiling intersections; small cracks in masonry chimneys and masonry veneers. Small cracks are assumed to be visible with a maximum width of less than 1/8 inch (cracks wider than 1/8 inch are referred to as "large" cracks).
	**Moderate**	Large plaster or gypsum-board cracks at corners of door and window openings; small diagonal cracks across shear wall panels exhibited by small cracks in stucco and gypsum wall panels; large cracks in brick chimneys; toppling of tall masonry chimneys.
	**Extensive**	Large diagonal cracks across shear wall panels or large cracks at plywood joints; permanent lateral movement of floors and roof; toppling of most brick chimneys; cracks in foundations; splitting of wood sill plates and/or slippage of structure over foundations.
	**Complete**	Structure may have large permanent lateral displacement or be in imminent danger of collapse due to cripple wall failure or failure of the lateral load resisting system; some structures may slip and fall off the foundation; large foundation cracks. Three percent of the total area of buildings with Complete damage is expected to be collapsed, on average.

Figure 3.2.4. Capacity Spectrum and Damage States, Light Wood-Frame, HAZUS®.

HAZUS® provides fragility functions, to relate building response behaviors to damage values, for various model building types, ranging from wood-frame to concrete-frame to mobile homes. In addition, an occupancy class for the building of interest must be specified, and includes residential, commercial, industrial, education and more. The occupancy class provides information on value of building contents and non-structural parameters that exist. For this research, building type *W1* (light, wood-frame) and *RES3* (multi-family dwellings, such as apartments and condos) are used for the case study building. The HAZUS® manual[10] provides fragility information for each system per building type. To reiterate, these systems include structural damage (S), non-structural drift acceleration damage (D) and non-structural acceleration sensitive damage (A). The fragility functions provide limits in each category of damage (slight, moderate, extreme, collapse) in a stepwise manner. For this analysis, these functions have been idealized as a tri-linear graph to simplify the subsequent calculations (shown in Figure 3.2.1).

10 Department of Homeland Security. *Earthquake Loss Estimation Methodology HAZUS® -MH MR5* (FEMA) 2-6.

Importantly, these fragility curves relate the PIDR to the Damage Values (DV), which is a percentage of the replacement value of the building. The total damage value per system adds up to 100% of the building value, as it is assumed that only three damage systems are relevant. For this study, the Structural, Drift and Acceleration systems are weighted since they each have differing values such that each scale represents a 0-100% damage level. In addition, the assessor must specify a code level, low- , moderate- or high- code which determines the stringency to which the building was constructed. In this case study, low-code fragilities will be applied as we assume the building was made in the early 1970's before the newer, seismically conscious code was adopted in the early 1990's.

As peak interstory drift ratio informs the structural and non-structural drift sensitive damage functions, and the peak floor acceleration to the non-structural acceleration sensitive functions, the response functions can be combined linearly to the respective fragility functions. These produce damage functions which are then used to estimate loss values. The procedure in converting response functions to damage functions includes simplifying the fragility functions (as described previously) and dividing the drift and floor acceleration values by the slope of the corresponding response functions (denoted "a" and "b" in Figure 3.2.1). This, then, produces three damage functions per point per floor on the building of concern. Figure 3.2.5 shows this method of combining the functions graphically. In this example, the procedure illustrates effects of the Northridge earthquake on a point within floor 1 to combine with the structural fragility to produce a damage function. To reiterate, the damage function shows the damage ratio for the structural system (S-DR) against the ground acceleration (PGA).[11]

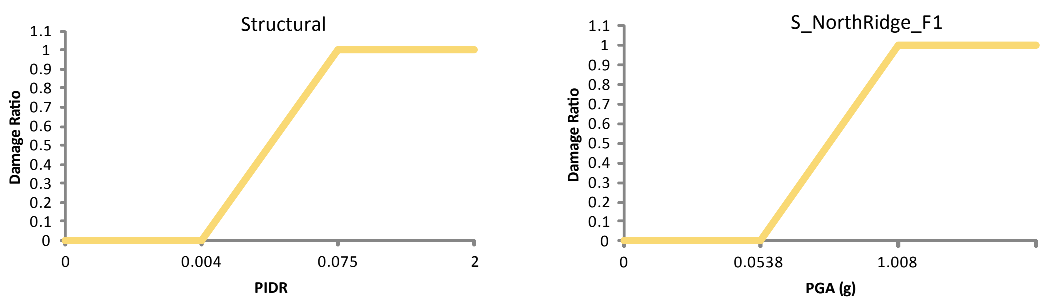


Figure 3.2.5. Fragility Function (Left) Damage Function (Right).

For the 16 earthquakes used on this 4 story building, a total of 192 damage functions are created. It is essential to highlight the importance of converting structural response into damage before averaging over the ground motion records. This also emphasizes a challenge in performance based earthquake engineering, in that the "consequences of an earthquake on a structure greatly depend on the characteristics of that earthquake, while overall seismic performance must be estimated considering a broad range of possible scenarios."[12] The case study assessment follows by averaging losses per floor

11 Fragilities can be found in Appendix

12 Ghisbain, Pierre. *Seismic Performance Assessment for Structural Optimization* (Cambridge, MIT; Doctoral Thesis, 2013).

and then by system, discussed in the next section.

3.2.4 Loss Assessment

Following the damage assessment, the next step is to find losses by taking into account the seismic hazard at the location of the building, via an exceedance probability (EP) curve, (Figure 3.2.3). In computing this step of the procedure, the opposite of the derivative of the annual exceedance frequency, referred to as the annual frequency density is multiplied by the system-specific damage functions to produce an annual damage curve as shown in Figure 3.2.6.

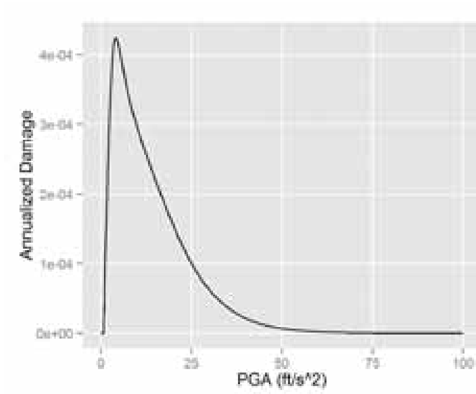


Figure 3.2.6. Annual Damage Density.

The annual damage density function is left system specific to understand the behavior of loss in a particular building, and whether those losses are primarily drift or acceleration sensitive. The final step in the assessment is to take the weighted average of the damage densities over the three systems, to find an annualized loss curve. HAZUS® provides information on system value, from which wood-frame structures have the following breakdown: Structural, 13.8% of building value; Non-structural Drift Sensitive, 42.5% of building value; Non-Structural Acceleration Sensitive, 43.7% of building value. It is, then, no surprise why an increase in the acceleration of the building causes more loss in dollar terms than does structural damage induced by drift.

The area under the damage density curve is the annualized damage ratio, which is an annual loss in terms of a percentage over the replacement cost of the building. Multiplying this ratio by the total value of the building gives a annual dollar loss amount that can then be converted into lifetime damage. It is assumed that the building has a usable lifetime of 50 years, with lifetime costs being calculated with a 5% discount rate per annum.

This annualized loss value, and its correlated lifetime damage value, can be used as the baseline to compare various scenarios of building design and retrofit, and leads to the optimization feedback presented in the diagram in Figure 3.2.1. This is an important step in the seismic assessment procedure, and as has been described, can be relatively complex. The next section comments on how Ghisbain has simplified portions of this procedure and how this can influence the design and retrofit process.

3.3 Comments on Methodology and Procedure

The PEER methodology has been tried and tested for nearly a decade, and has provided significant contribution to the understanding of damage and losses of buildings in seismic events. A common trend in Performance Based Design (PBD) includes the fact that many times, not enough time histories are used in typical probabilistic assessments due to computational limitations. In additional, often times the bottom line estimates presented in many PBD assessments provide building response as a final result, which is not useful to clients and decision makers who are generally not engineering professionals. Ghisbain has simplified this procedure by using a linear modal superposition method in his seismic assessments rather than a direct integration method that is common in advanced Performance Based Earthquake Engineering. In addition, the conversion of building response to dollar loss values proves more useful for understanding a building's behavior in general terms than the building response parameters allow. Ghisbain has constructed a set of tools to assist in the optimization of building design in regards to the types of retrofit investments that can be made. This is of great importance in this research, as extensive engineering is necessary such that informed and appropriate retrofit to vulnerable housing in the San Francisco and larger Bay Area.

The issue of current practice in which non-linear, direct integration methods are commonly used for seismic assessment is the cost and computational power that is required to conduct a single simulation. While accuracy of the results can be considered 100%, the time it takes to run a response analysis can take from days to weeks for a single design. This becomes highly impractical when design iteration is desired. Therefore, Ghisbain tested ways in which the procedure could be simplified, and noted the decrease in accuracy with each incremental simplification. He found that with linear, modal superposition, one can achieve 50-100% of accuracy as in the direct integration case, yet the major benefit is that runtime reduces to 10^{-3} to 10^{-4} as compared with a non-linear analysis. This provides significant leverage in conducting iterative design operations, and can conceivably produce a better end result or building design than in traditional methods. Figure 3.3.1 illustrates how design optimization is affected when comparing non-linear (direct integration) procedures to linear (mode superposition) assessments.

An optimal design solution will therefore be able to minimize the differences between the non-linear and linear methods described, as well as provide a best-case solution for retrofitting by balancing lifetime damage cost and investment in seismic mitigation. In this way, one can assume that a linearized method of assessment can provide up to 1000 more iterations than a non-linear, direct integration method. This allows for a better design in the end, without much loss of accuracy in the simulation and data generating process of the seismic evaluation.

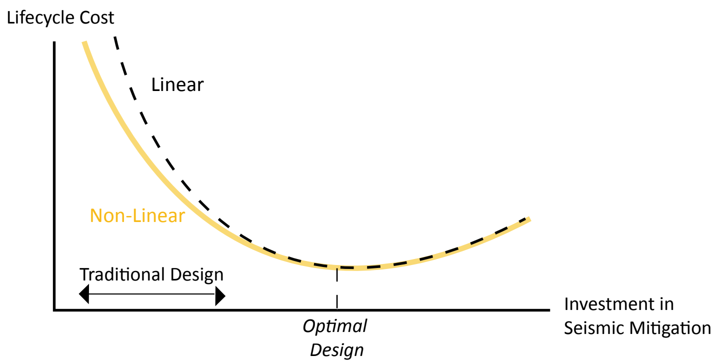


Figure 3.3.1. Nonlinear versus Linear Procedure for Optimal Design.

Finally, it should be made clear that any simulation model cannot fully capture the properties of the building in question. In this case study, the model analyzed in SAP2000 included the main frame elements and the shear walls for lateral resistance. Connection details are not specifically researched nor incorporated into the model. Given the flexible nature of woodframe structures, and with conscious effort to appropriately model such behavior, there still remain inherent and somewhat unavoidable errors that are produced by the manner in which the building has been simulated in SAP2000. However, the response values produced by these simulations correlate to those conducted by other researchers and organizations that have examined the case study building via pushover analyses. The following chapters address nuances in modelling and subsequent results from the seismic assessment.

04
CASE STUDY

4.0 Case Study

4.1 Wood-Frame Residences

Wood-frame multi-unit buildings account for some of the most susceptible structures to shaking damage and also define the City's inventory of apartments and condominiums. These buildings are considered to have soft-stories, described by the Department of Building Inspection as having ground floor openings that are 80% open or more on one side or 50% open or more on two sides.[1] A soft-story residential building typically has an open parking or commercial space located on the first floor and housing on higher floors built prior to recent codes. In a seismic event, ground shaking causes the building to sway, twist and, in many cases, collapse. The swaying and collapse of the soft story can cause great damage to individuals or cars in these open areas, as well as to the floors above. Typical soft-story multi-unit residences were built before 1990, when current codes did not specify mitigation of such vulnerable construction practices.

Figure 4.1.1. Wood-frame soft-story corner building (left). Effects of seismicity on soft-story (right). Image courtesy of Association of Bay Area Governments.

4.2 Case Study Building

In 2007, the Department of Building Inspection, SEAOC and Earthquake Engineering Research Institute (EERI) Northern California surveyed buildings to catalogue various buildings susceptible to great seismic damage. The organizations described the following four index buildings as having high earthquake vulnerabilities:

Index building 1 is a four story, 8 unit building with 5,800 square feet per floor. It is corner, stand-alone building in this case, and has a soft-story for parking underneath. Index building 2 is a three story building with 5+ units, also a corner block building featuring a soft-story. Index building 3 is a four-story mid-block building including a soft-story. Finally, Index building 4 is similar to Index building 3, as a mid-

1 Applied Technology Council. *Here Today- Here Tomorrow: The Road to Earthquake Resilience in San Francisco; Potential Earthquake Impacts: Technical Documentation 53-2A* (Redwood City, CA; Applied Technology Council, 2010).

block building, but having fewer stories.

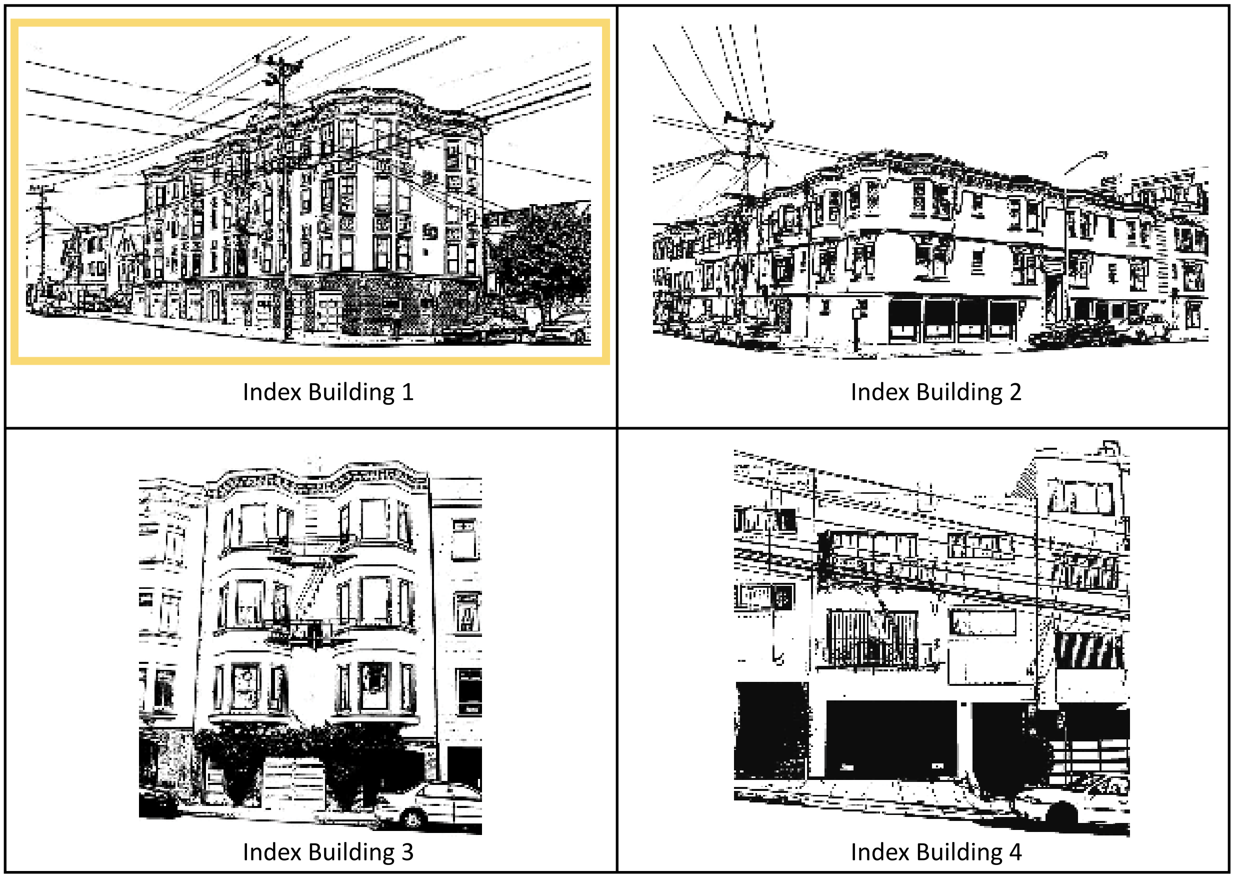


Figure 4.2.1. Index Buildings as Categorized by Applied Technology Council. Report 53-2A.

For this research, Index building one has been tested for behavior and retrofit given the performance testing method cited by Ghisbain and fully described in Chapter 3.[2] The decision to use Index Building one is purely a function of the number of vulnerable units, as this typology houses the most number of individuals with 8 units on average, and is the most prevalent type of residential housing. Table 4.2.1 shows that this type of building accounts for nearly 60% of the surveyed wood-frame residences that make up the City's housing stock. More residents within such housing would be susceptible to damage and retrofit implications, and therefore, this typology is of primary interest to this research.

Index Building one is considered a corner building with significant ground opening in the following table. As is noted, there are over 600 buildings of this type, with a high density of residents as compared with other building types in this study. These numbers will be used to provide a city-wide loss assessment in the final chapter of this report.

2 Ghisbain, Pierre. *Seismic Performance Assessment for Structural Optimization* (Cambridge, MIT; Doctoral Thesis, 2013).

Building Type	3 stories		4 stories		5 or more stories	
	Number of Buildings	Number of Residential Units	Number of Buildings	Number of Residential Units	Number of Buildings	Number of Residential Units
All buildings	1,662	13,395	2,630	28,965	121	2,159
Buildings with significant ground floor openings	1,087	9,197	1,594	17,809	55	1,082
Corner buildings with significant ground floor openings	387	3,783	626	8,184	27	586

(Significant ground-floor openings refers to buildings that are 80% open or more on one side or 50% open or more on two sides.)

Table 4.2.1. Number of Stories in Multi-Family Wood-Frame Buildings. Table 2-5, ATC 53-2A.

4.3 Building Characteristics

The four case study buildings have been analyzed by various researchers and organizations, since very good data on the buildings' makeup exists. This research relied primarily on information from the Applied Technology Council (ATC), which ran pushover analyses to each of the case study typologies to better understand the behavior of movement and damage the building can sustain. For our case study building, the ATC has delineated that yield and peak capacities on the upper floors are dependent solely on the interior finish material, which is plaster over wood lath. The ATC found that the "contribution of other materials was minimal, due to the high flexibility and low capacity. This suggested that…it is very reasonable to limit the retrofit work to the ground story, without concern that damage causing life-safety concerns will occur at the upper floors."[3] The ATC reports some of the structural nuances they have found via the pushover analysis conducted:

> "The case study building displays significant torsional response under longitudinal loading. In the original building configuration, the center of rigidity was at the rear longitudinal wall. The effect of the torsion is to put very high demands on the end transverse walls. For a ground motion at an angle to the primary axes, the combination of direct and torsional load on these walls is significant and of concern. Significant damage to end transverse walls was identified in CUREE-Caltech Project testing of an open wall building on the Berkeley shake table (Mosalam et al., 2002). This particular vulnerability of this geometry of building should be taken into consideration. This behavior was not an issue on [Index] Building 2, due to the contribution of interior walls."[4]

3 Applied Technology Council. *Here Today- Here Tomorrow: The Road to Earthquake Resilience in San Francisco; Potential Earthquake Impacts: Technical Documentation 53-2A* (Redwood City, CA; Applied Technology Council, 2010). 23.

4 *Ibid*. 24.

Information from the ATC reports and CUREE-Caltech Woodframe Project[5] provided enough information to be able to model the building for simulation of performance via SAP2000 (explained in the following section). Figure 4.3.1 depicts some of the structural features of the general case study building.

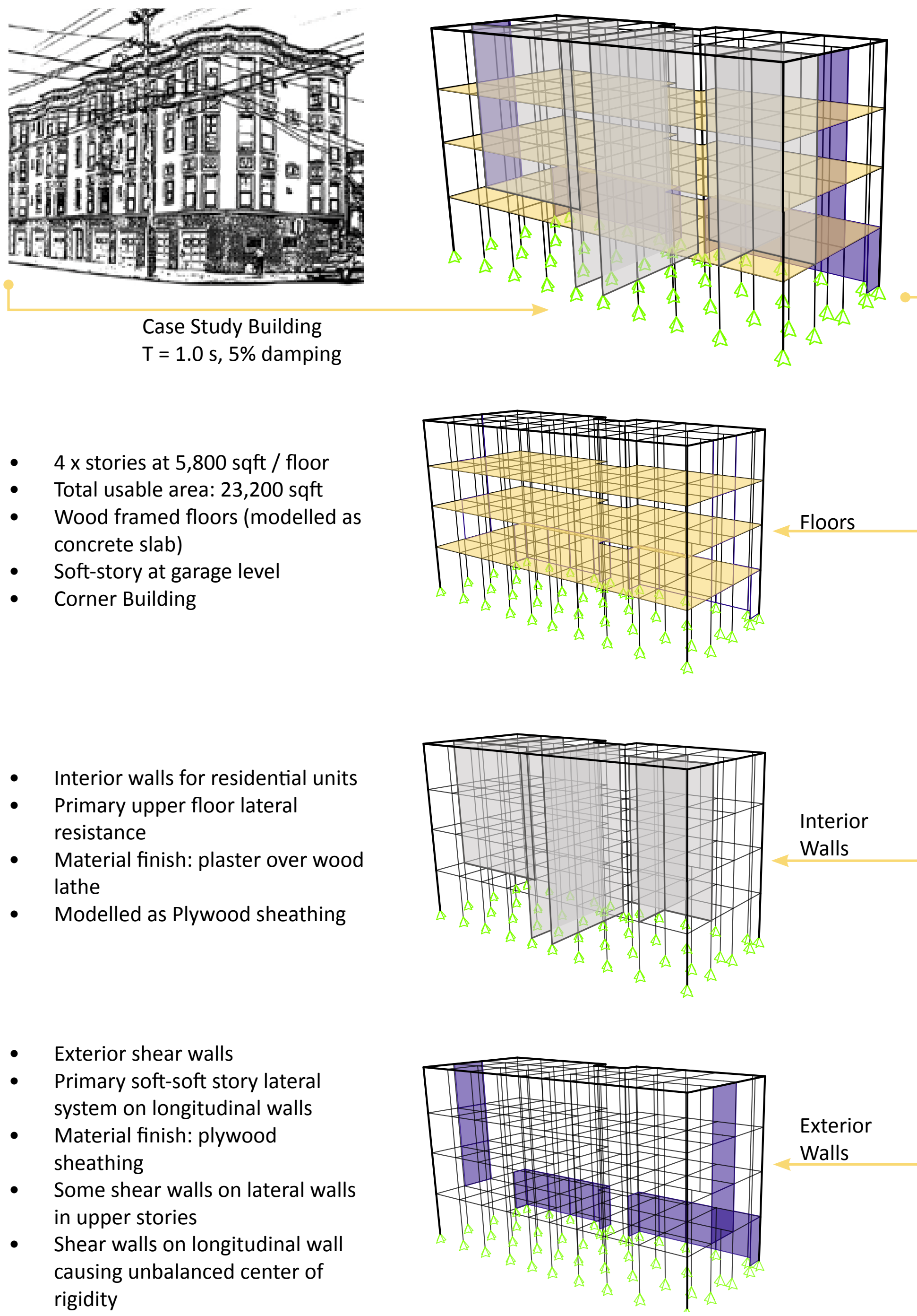


Figure 4.3.1. Structural Characteristics of Case Study Building.

5 Christovasilis, I.P., Andre Filiatrault, Michael Constantinou, Assawin Wanitkorkul. *Incremental Dynamic Analysis of Woodframe Buildings* (Earthquake Engineering and Structural Dynamics, 2008).

In addition, the ATC has found that in general, all four index buildings have a natural period of about 1.0 seconds, with a covariance of about 15%.[6] Even though this is a shorter building, the properties of the wood structure cause it to have a higher period that similar buildings of different materials, mainly due to the flexible nature of light wood construction.

4.4 Structural Model

The case study building was modeled in SAP2000, using properties provided by the California Action Plan for Seismic Safety. Some assumptions were required in regards to the material and sectional properties of the building, discussed in the Appendix. Figure 4.4.1 shows the base case model as designed in SAP2000.

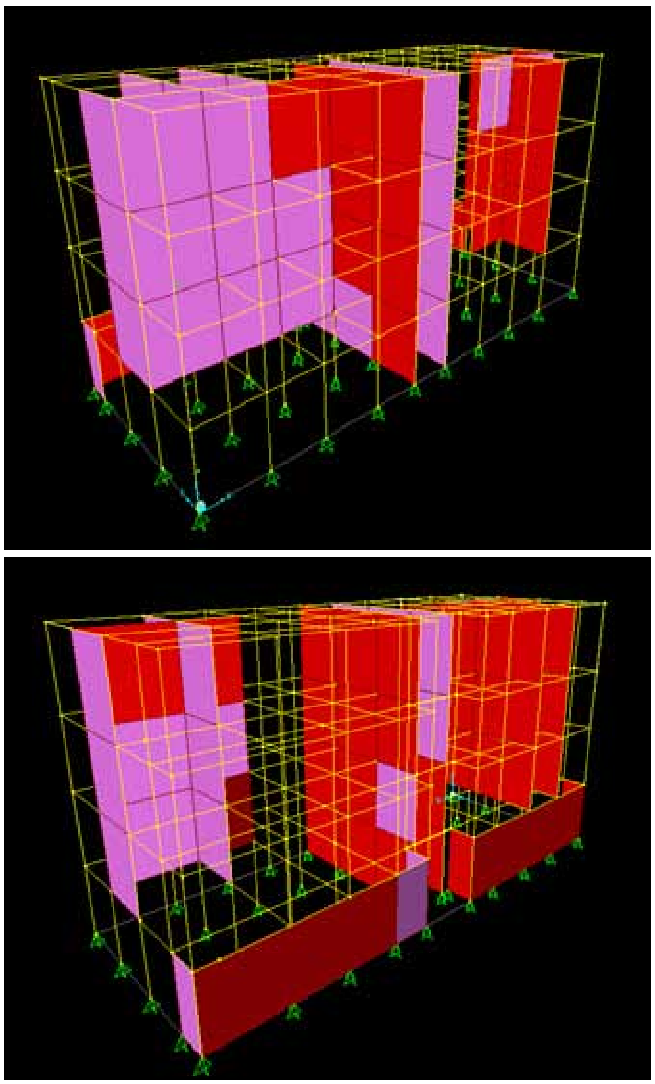


Figure 4.4.1. SAP2000 Structural Model. Top Image Shows Front Facade, Bottom Picture Shows Rear of Building.

6 Applied Technology Council. *Here Today- Here Tomorrow: The Road to Earthquake Resilience in San Francisco; Potential Earthquake Impacts: Technical Documentation 53-2A* (Redwood City, CA; Applied Technology Council, 2010). 49.

The gravity system is defined by 2x6 Douglass Fir timber members, with plywood panels for the shear walls that provide the main lateral resistance for this building. A more detailed description of the material properties can be found in the Appendix. The soft-story is shown to only have half of the longitudinal wall having structural sheathing, which attests to the center of rigidity being closer to the back wall, and causing torsional failures in the structure.[7] Although the actual building probably features wood floors, the structure is modeled as having concrete slab for flooring, to be able to capture the weight of the interior partitions and other elements atop each floor.

The geometry of the building has been extrapolated from the following plan provided by the Applied Technology Council.[8] The plan in Figure 4.4.2 shows the garage floor as not being completely

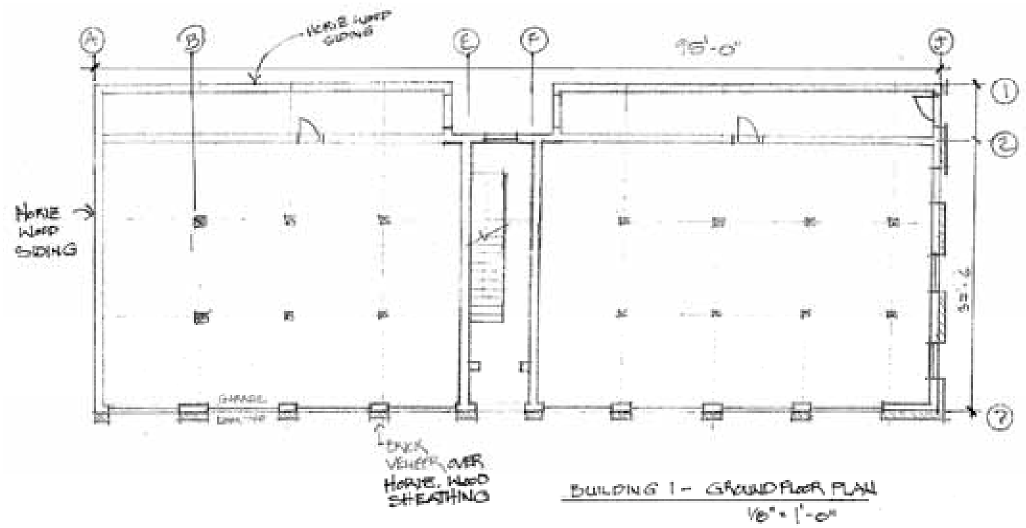


Figure 4.4.2. Ground Floor Plan. ATC 53-2A.

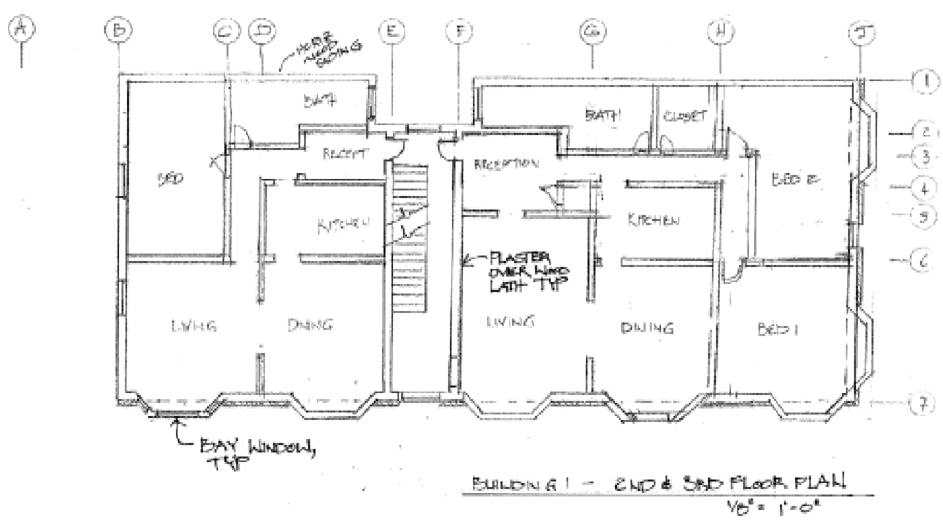


Figure 4.4.3. Second through Fourth Floor Plan. ATC 53-2A.

7 Torsional behavior is observed in the SAP2000 simulation as third mode behavior

8 Applied Technology Council. *Here Today- Here Tomorrow: The Road to Earthquake Resilience in San Francisco; Potential Earthquake Impacts: Technical Documentation 53-2A* (Redwood City, CA; Applied Technology Council, 2010). 25.

symmetrical, with an additional bay on the east side of the building. For the simulation, the building has not modelled this discrepancy in bays for sake of simplifying the structure. It is also important to note the veranda as the only continuous longitudinal wall that sits south of the exterior wall line. This veranda wall will be strengthened in one of the retrofit options to assess any benefits from increasing lateral resistance, and which will be further described in section 4.5. Also significant is the location of the rooms on the second and third floor plans in Figure 4.4.2. These show that the shear walls in the top stories (which exclusively provide lateral resistance for the above floors) are not symmetrical, which reinforces the torsional rotation issues this building faces in seismic loading.

4.5 Structural Retrofits

In order to evaluate the benefit of a linearized structural response and damage assessment described by Ghisbain[9], fourteen different retrofit options have been suggested for the abovementioned case study building. The retrofits were inspired by those described in the Applied Technology Council reports,[10] as well as a key report by Porter and Cobeen.[11] In the latter publication, the authors have tested three different retrofits on the four index buildings to assess improvements in response to four major earthquakes. These include a magnitude 7.9, 7.2 and 6.5 San Andreas Fault event, as well as a magnitude 6.9 Hayward Fault event. The following excerpt from the report discusses the three retrofit options:

> "Retrofit option 1 included the addition of structural sheathing at ground-story walls with inadequate bracing length. The performance goal was to provide a reasonable assurance that the building would be safe, though perhaps not repairable, after a major earthquake. Retrofit 2 added steel frames at garage openings. The performance objective was to provide reasonable assurance that the building would be safe and usable after repair. Retrofit 3 was like 2, but using cantilever columns designed to resist the same seismic forces as the steel frames, with a lower R factor. This option aimed to provide reasonable assurance that the building would be safe and usable after repair, and possibly during repair."[12]

For this research, Retrofit 1 and 2 have been applied, with variations of each being tested for performance. Table 4.5.1 sourced from Porter and Cobeen[13] provides cost information for the retrofit, some of which is used for the processing of the annualized damage estimates explained in Chapter 5. In the mentioned report, the case study building is cited as Index Building 2 (IB2).

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9 Ghisbain, Pierre. *Seismic Performance Assessment for Structural Optimization* (Cambridge, MIT; Doctoral Thesis, 2013).

10 Applied Technology Council. *Here Today- Here Tomorrow: The Road to Earthquake Resilience in San Francisco; Potential Earthquake Impacts: Technical Documentation 53-2A* (Redwood City, CA; Applied Technology Council, 2010). Appendix 4-5

11 Porter, K. and Kelly Cobeen. *Informing a Retrofit Ordinance: A Soft-Story Case Study* (Structures Congress, ASCE; 2012).

12 *Ibid*. 1805.

13 *Ibid.*

Retrofit	SPUR (2008) performance objective	Cost per unit, $000				Total cost, $ million
		IB1	IB2	IB3	IB4	
1. Add structural sheathing	D, safe but not repairable	$ 20	$ 6	$ 10	$ 12	$180
2. Same plus steel frames	C, safe and usable after repair	30	11	18	15	$300
3. Same by cantilever columns instead of steel frames	C or B, safe and usable during repair	28	9	16	15	$260

Table 4.5.1. Retrofit Alternatives and Costs.
Table 1, Porter and Cobeen.

The following sections describe the retrofits applied to the building. Each retrofit has been given a short hand, and is also depicted by a respective diagram. Because the interior shear walls on the upper floors have not been altered throughout the performance testing process, these have been removed from the icons for clarity. As described earlier, it is logical to focus all retrofits to the soft-story where the greatest amount of drift is to occur. In all but one of the retrofits subsequently described, alterations are limited to the first story of the building only.

4.5.1 Retrofit 1- 10% Damping (DMP 0.1)

As the building has embedded 5% damping, additional damping has been added via viscous dampers. In SAP2000, this retrofit has been modeled by applying 10% global modal damping to the structure and running the simulation. Later, links are drawn in the indicated locations in red (Figure 4.5.1) to represent the dampers (2 braces applied in this case), and are then compared to the overall damping drift values provided by global damping. Once a match in drift is made between global damping, the dampers can be sized via their axial forces to find appropriate cost for the dampers, explained further in Chapter 5. Exterior shearwalls have not been included in the diagrams for damper additions, but are present in the actual structural model.

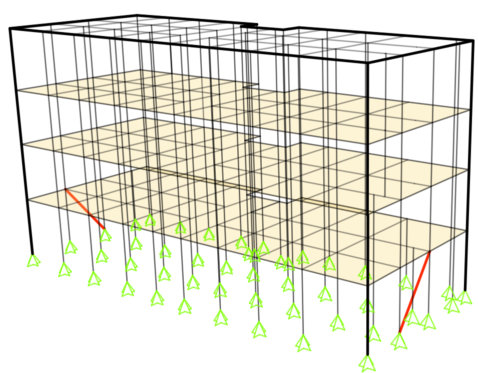


Figure 4.5.1. Retrofit 1: 10% Damping via Viscous Dampers.

4.5.2 Retrofit 2- 15% Damping (DMP 0.15)

Same as 10% damping listed above. In this case 4 braces are included.

4.5.3 Retrofit 3- 20% Damping (DMP 0.20)

Same as 10% damping listed above. In this case 4 braces are included.

4.5.4 Retrofit 4- 30% Damping (DMP 0.30)

Same as 10% damping listed above. In this case 4 braces are included.

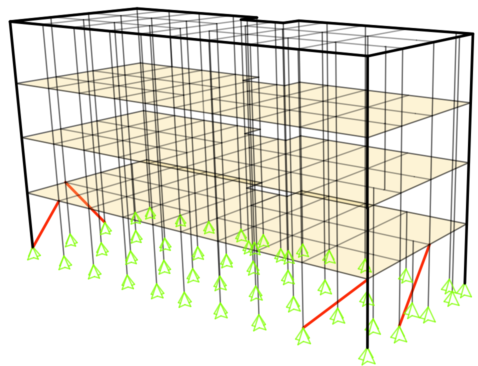


Figure 4.5.2. Retrofit 2,3,4: 15%, 20%, 30% Damping via Viscous Dampers.

4.5.5 Retrofit 5- Steel Moment Frame (SMF 1)

This retrofit strategy is very similar to the second retrofit in Porter and Cobeen's[14] study discussed previously. Two steel moment frames are added to the front façade of the soft-story, where the garage is entered. On the east side are introduced W10x45 steel moment frames, while on the west side W12x45 steel are installed. The choice of these sections and sizes of the steel moment frame are taken directly from Porter and Cobeen's suggestions.

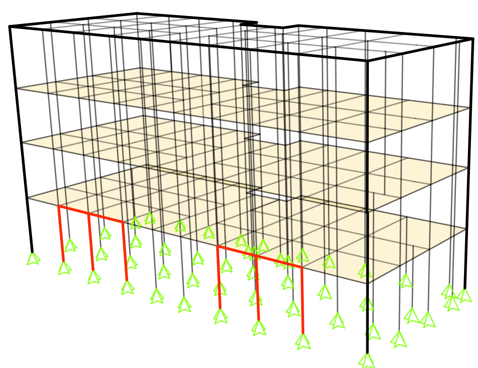


Figure 4.5.3. Retrofit 5: Addition of Steel Moment Frames to Garage Front.

14 Porter, K. and Kelly Cobeen. *Informing a Retrofit Ordinance: A Soft-Story Case Study* (Structures Congress, ASCE; 2012).

4.5.6 Retrofit 6- Steel Moment Frame (SMF 2)

In this case, the steel moment frames are installed through the entirety of the soft story, with W10x45 being applied to the east side of the structure, and W12x45 to the west of the soft story.

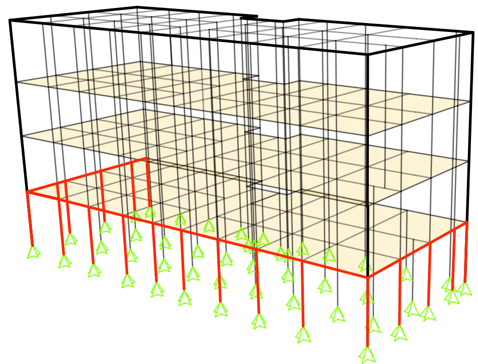


Figure 4.5.3. Retrofit 6: Addition of Steel Moment Frames Entirety of Soft-Story.

4.5.7 Retrofit 7- Shearwall 1 (SW 1)

This retrofit strategy applies plywood shear walls to the lateral walls of the soft story. This provides needed resistance against torsional movement in the weak axis of the structure.

4.5.8 Retrofit 8- Shearwall 2 (SW 2)

This retrofit strategy applies Oriented Strand Board (OSB) shear walls to the lateral walls of the soft story. This also allows for needed lateral resistance against torsional movement. In this research, OSB has been found to be stronger than plywood shear panels, as well as about 33% less in cost.[15]

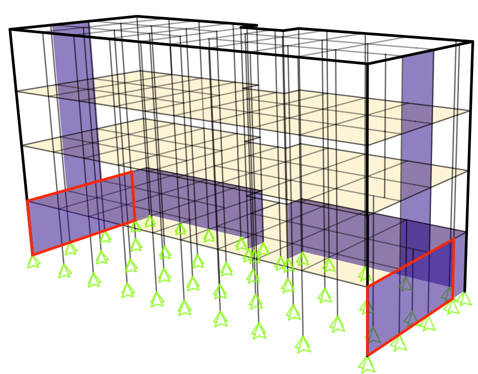


Figure 4.5.4. Retrofit 7,8: Addition of Plywood/Oriented Strand Board Sheathing on Lateral Walls.

15 HomeDepot.com for shearwall panel costs. See Appendix for more information on material properties in model.

4.5.9 *Retrofit 9- Composite 1 (CMP 1)*

Composite retrofits are described as having a combination of the above described retrofits, with additional nuances. Composite 1 introduces steel moment frames as is done in SMF1, and also adds Oriented Strand Board (OSB) panels to the lateral walls on the soft-story.

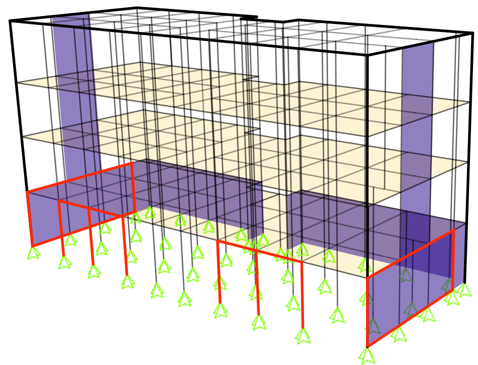


Figure 4.5.5. Retrofit 9: Addition of Moment Frame at Facade of Garage and Oriented Strand Board on Lateral Walls.

4.5.10 *Retrofit 10- Composite 2 (CMP 2)*

Composite 2 introduces steel moment frames as is done in SMF2 (on three sides of the soft-story), and also adds Oriented Strand Board (OSB) panels to the lateral walls on the soft-story.

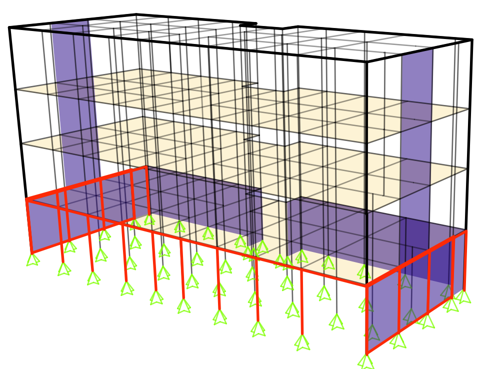


Figure 4.5.6. Retrofit 10: Addition of Moment Frame at Entirety of Soft-Story and Oriented Strand Board on Lateral Walls.

4.5.11 Retrofit 11- Composite 3 (CMP 3)

Composite 3 introduces steel moment frames as is done in SMF2 (on three sides of the soft-story), and also adds Oriented Strand Board (OSB) panels to the lateral walls on the soft-story. In this case, additional Oriented Strand Board paneling is applied to the veranda for lateral resistance in the longitudinal direction of the building.

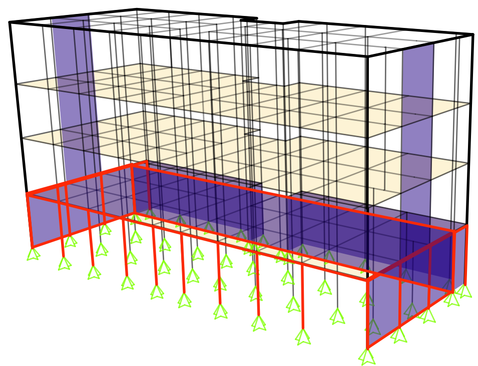


Figure 4.5.7. Retrofit 11: Addition of Moment Frame at Entirety of Soft-Story and Oriented Strand Board on Lateral Walls and Veranda.

4.5.12 Retrofit 12- Composite 4 (CMP 4)

Composite 4 is the same as Composite 3, but with an addition of 20% damping included as viscous dampers. It is intended that all retrofits occur at the same time, and therefore, there is no deconstruction of previously applied retrofit components.

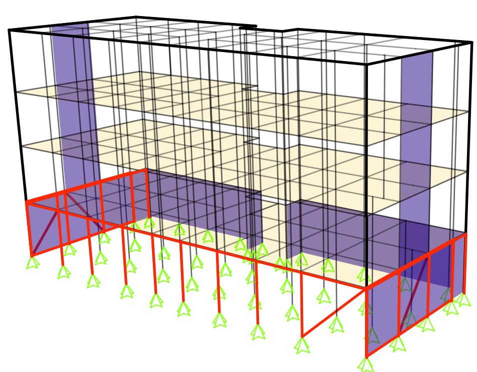


Figure 4.5.8. Retrofit 12: Addition of Moment Frame at Entirety of Soft-Story and Oriented Strand Board on Lateral Walls and 20% Damping.

4.5.13 Retrofit 13- Composite 5 (CMP 5)

Composite 5 applies moment frames to the front façade of the building only (as in SMF 1), but also strengthens the height of three sides of the exterior façade via Oriented Shear Board (OSB) panels applied as lateral resistance for the whole structure. This is the only retrofit option that suggests components beyond the first, soft-story level.

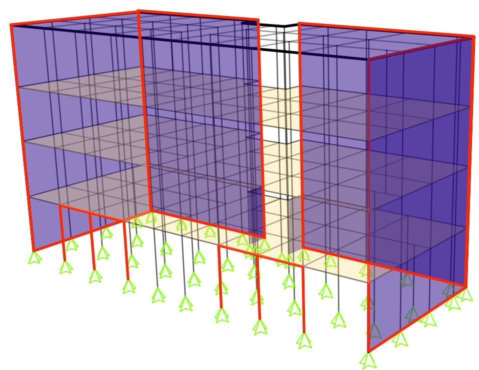


Figure 4.5.9. Retrofit 13: Addition of Moment Frame at Facade of Garage Oriented Strand Board on Exterior Walls.

4.5.14 Retrofit 14- Base Isolation (BI)

Base isolation will be introduced as lead filled rubber bearings one meter below the ground level of the building. No additional lateral resistance or steel support need be added in this case.

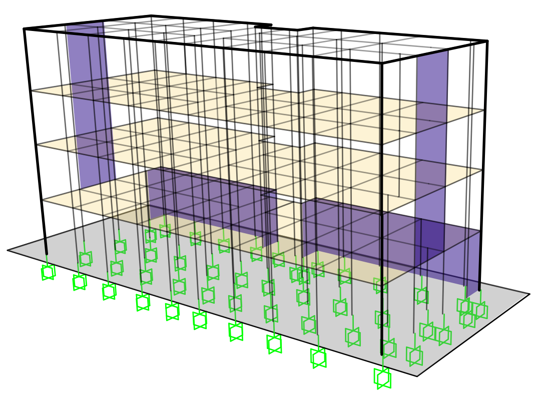


Figure 4.5.10. Retrofit 14: Base Isolation.

In all of these cases aside from the last one, a strengthening approach has been taken. However, in seismic regions, stiffening a building can reduce drift based damage, but peak floor acceleration of the building can increase, causing acceleration induced damage to rise. This becomes important because the value of non-structural acceleration sensitive elements far outweigh those of structural elemental costs. Therefore, although peak drifts are reduced when the building is strengthened, the acceleration of the structure causes more damage to higher-valued elements, increasing the total damage ratio. Connor[16] suggests softening the building to reduce the amplitude of motion such that the building does not feel the effects of seismic excitation as readily as in a strengthening approach. In this way, base isolation is able to significantly reduce both drift and acceleration by increasing the natural period of the building and creating a sacrificial level underground, removing vulnerability from the soft-story, and thereby, preventing collapse.

The following chapter provides results from base case and retrofit simulations conducted for this report. It also provides damage estimates and retrofit costs such that these seismic mitigation options can be compared.

16 Connor, J.J. *Introduction to Structural Motion Control* (Prentice Hall, 1st Edition; 2002). Chapter 4-6.

05
RESULTS

5.0 Results

5.1 Base Case Simulation Behavior

Evaluating the base case building through the seismic performance assessment delineated by Ghisbain[1] provide results that give interesting insight to the behavior of the structure after application of suggested retrofits described previously. Factors of period, frequency, and Annual Damage Ratio (ADR) of the original building will provide baseline values to appraise subsequent retrofits options. Damage Ratios are a percentage of the total replacement value of the building, which is computed to be $12,180,000.00.[2] [3] The following results will show that each retrofit is an improvement in reducing the fundamental period of the building, most commonly by strengthening the structure. However, the combination of retrofit cost plus lifetime damage costs vary significantly when comparing retrofit strategies, especially as the building is stiffened. The case study building has an initial period of 1.11 seconds, which confirms with tests done by the Applied Technology Council on the index buildings described in Chapter 4.

As can be predicted, the soft-story is not able to resist lateral movements sufficiently, and sways significantly when impulsed with seismic loading. Given the geometry of the building and a non-distinct center of rigidity between the upper floors and the ground floor, torsional rotation is especially problematic. Running the model in SAP2000 attests to the swaying of the soft story, as Figure 5.1.1 shows. In this way, the difference in stiffness between the higher residential floors and lower, open area garage or commercial space manifests in undesirable modal behavior. Instead of vibration in a linearized mode shape, which would be ideal, the soft story faces greater drift which eventually leads to collapse due to the P-Δ effect.

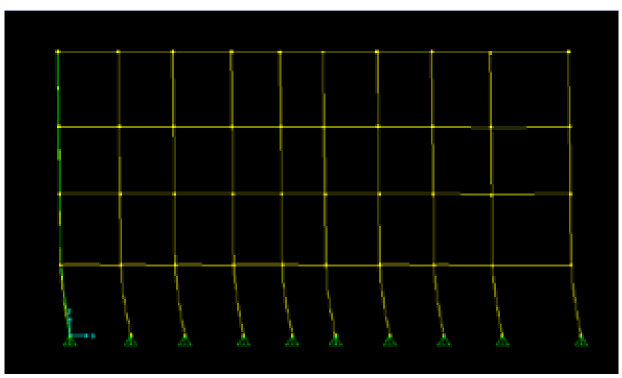


Figure 5.1.1. Base Case Behavior, SAP2000.

1 Ghisbain, Pierre. *Seismic Performance Assessment for Structural Optimization* (Cambridge, MIT; Doctoral Thesis, 2013).

2 Applied Technology Council. *Here Today- Here Tomorrow: The Road to Earthquake Resilience in San Francisco; Potential Earthquake Impacts: Technical Documentation 52-1A* (Redwood City, CA; Applied Technology Council, 2010).

3 Calculated as 5800 sqft/floor * $350/sqft * 4 stories.

5.2 Overall Results

The fourteen different retrofit options along with the base case scenario damage assessment results are provided in the following sections. In order to better understand the results, recall that the Annualized Damage Ratio (ADR) is the base line quantifier from which dollar loss amounts can be computed. Annualized Damage Ratio is calculated as the integral of the Annual Damage Density Function, which are shown for each retrofit option. In order to optimize performance, a lower ADR is desired. Annualized damage data can then be converted to lifetime damage by taking a 5% discount rate over 50 years, the expected life of the building. Where these values become useful is in adding the cost of each respective retrofit option to the lifetime damage loss in dollars. One is then able to compare the various solutions given the retrofit cost and the reduced lifetime damage cost. Information on calculating the costs of various retrofit options are provided in the Appendix. Table 5.2.1 describes the various retrofits analyzed in this study.

DMP 0.1	Damping 10%
DMP 0.15	Damping 15%
DMP 0.20	Damping 20%
DMP 0.30	Damping 30%
SMF_1	Steel Moment Frame On Front Façade Only
SMF_2	Steel Moment Frame all around Soft Story (including Lateral Walls)
SW_1	Plywood Shearwall Panels to Lateral Walls in Soft-Story
SW_2	OSB Shearwall Panels to Lateral Walls in Soft-Story
CMP_1	Composite 1: OSB Shear wall and Moment Frame on Front Façade Only
CMP_2	Composite 2: OSB Shear walls and Moment Frames on Entirety of Soft-Story
CMP_3	Composite 3: OSB Shear walls added to longitudinal veranda; Frames all around Soft-Story
CMP_4	Composite 4: Same as Composite 3 with 20% Damping Added
CMP_5	Composite 5: OSB Shear Walls on Entirety of Exterior of Building; Frames on Front Façade Only
BI	Base Isolation @ cost of 5% bulding value

Table 5.2.1. Description of Retrofits

The following images show the building characteristic to the corresponding damage density function and the calculated Annualized Damage Ratio (ADR) value. The damage density functions are plotted against Peak Ground Acceleration in feet per square second. In all cases, the interior partition walls on the upper floors are included in the simulation but are omitted from the diagram for clarity. The particularities of each retrofit scenario are highlighted in red. The base case model presented first exemplifies the manner in which the wood-frame building has been simplified for reasons of structural analysis. Each case provides the first mode period and subsequent frequency on the bottom right.

5.2.1 *Base Case*

The base case model represents the unaltered wood-frame building described as Index Building 1 in the ATC reports.

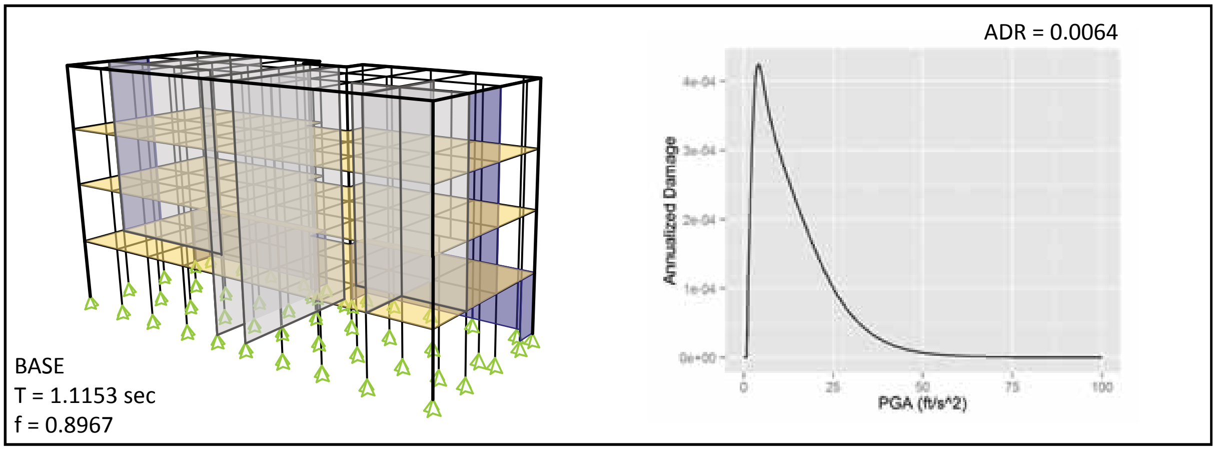


Figure 5.2.1. Base Case Results.

The base case shows the severity of damage that occurs between 6 to 8 PGA in feet per square seconds (about a magnitude 5 to 6 earthquake).[4] This is exactly the type of behavior that is expected- where high frequency, lower magnitude earthquakes cause more damage over the lifetime of the building than larger intensity, lower frequency events. A possible reasoning for this type of loss is that the building incrementally weakens and accumulates lesser extent damages over time due to the low intensity but high occurring events. When an MCE event happens, the building has already been made vulnerable over its lifetime due to the accrued damage from smaller events, hence causing it to have less resistance to large scale loss. This peak behavior of the damage density curves can be observed in mostly all the retrofit options suggested, however, with the peak damage ratios being slightly or significantly reduced from the base case as the buildings are strengthened. It is interesting to note the logarithmic decrement of the density curve. This occurs because the hazard curve reaches very low probabilities of exceedance when high intensity earthquakes are reached. When multiplying the hazard curve by the damage function, one can begin to understand the characteristic nature of the density functions.

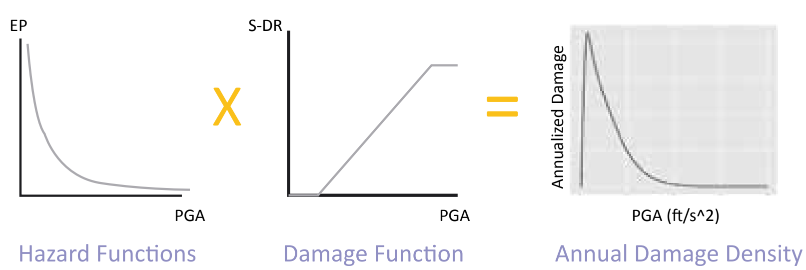


Figure 5.2.2. Computation of Damage Density.

4 Refer to Appendix for PGA to magnitude comparison.

5.2.2 Retrofit 1- 10% Damping

In this scenario, two viscous dampers have been fit into the lateral walls of the structure. For simulating this model, 10% global damping is applied to all modes.

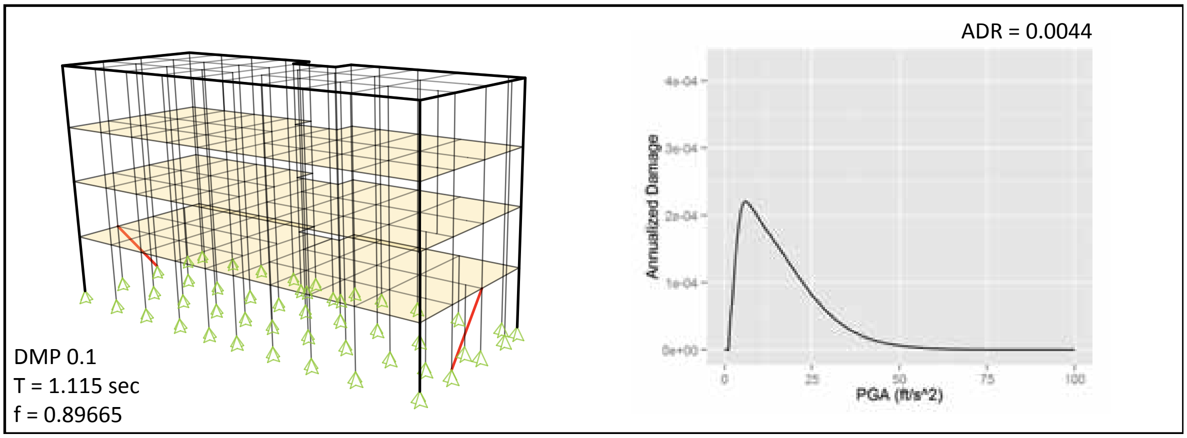


Figure 5.2.3. 10% Damping Results.

5.2.3 Retrofit 2- 15% Damping

In this scenario, four viscous dampers have been fit into the lateral walls and garage openings of the structure. For simulating this model, 15% global damping is applied to all modes.

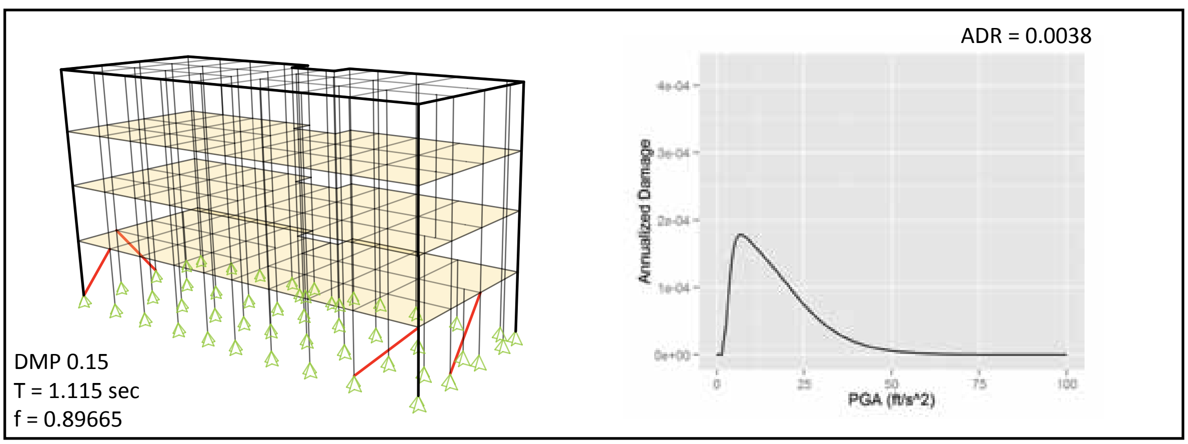


Figure 5.2.4. 15% Damping Results.

5.2.4 *Retrofit 3- 20% Damping*

In this scenario, four viscous dampers have been fit into the lateral walls of the structure and garage openings of the structure. For simulating this model, 20% damping has been applied to all modes.

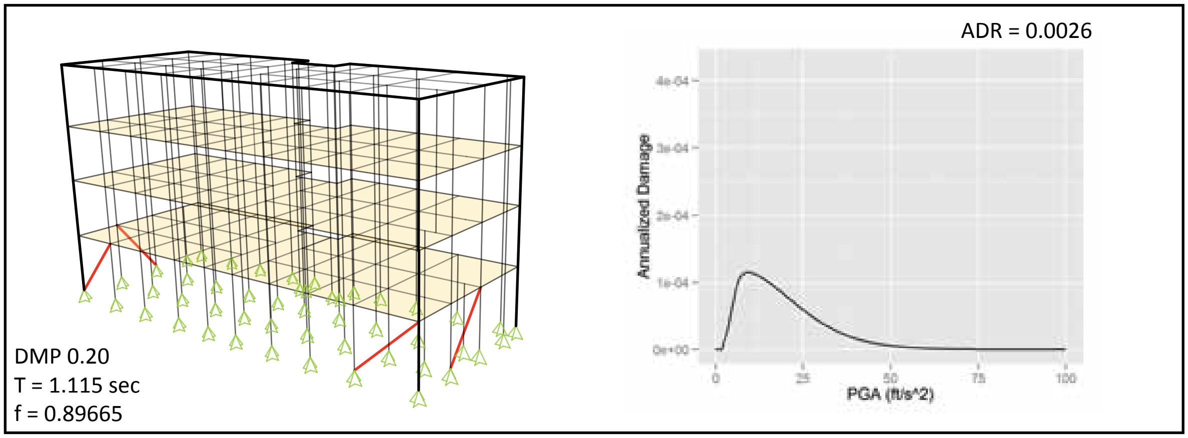


Figure 5.2.5. 20% Damping Results.

5.2.5 *Retrofit 4- 30% Damping*

In this scenario, four viscous dampers have been fit into the lateral walls and the garage openings of the structure. For simulating this model, 30% damping has been applied to all modes.

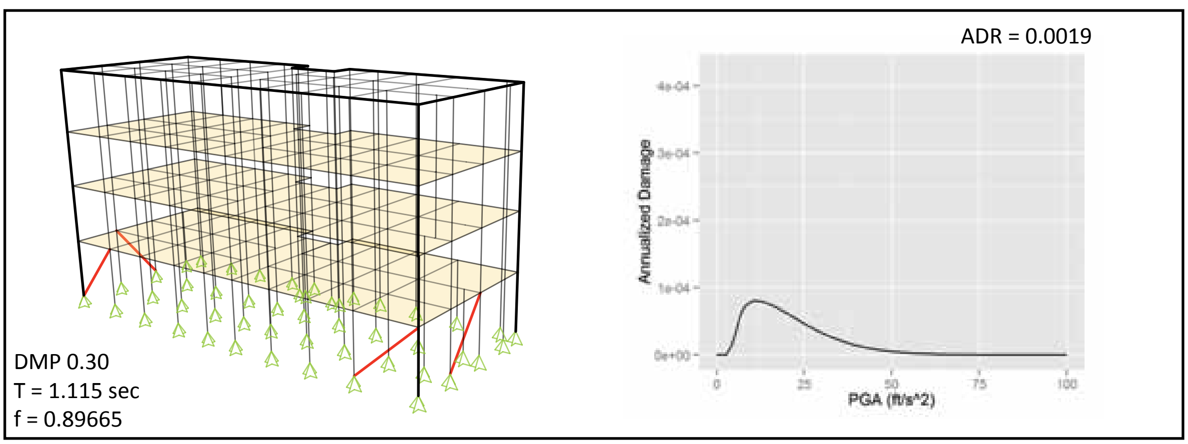


Figure 5.2.6. 30% Damping Results.

An important observation can be made in the case of global damping (i.e., provision of viscous dampers) in Retrofit options 1-4. The annualized damage density curves have lower peaks as damping in the building is increased. However, although the damage values decrease with higher modal damping, the relationship between damage and damping is not a linear one. Therefore, after a point, additional damping does not give much better improvement. In this case, 20% damping seems to provide significant benefit in reducing peak damage values, with additional damping not providing a large scale reduction in damage values. In addition, it should be noted that the building's fundamental frequency is not changing with the addition of damping, and remains the same as the base case model. This is characteristic of viscous dampers, in that the dampers are able to dissipate seismic energy without altering the building's inherent properties.

5.2.6 Retrofit 5- Steel Moment Frame 1

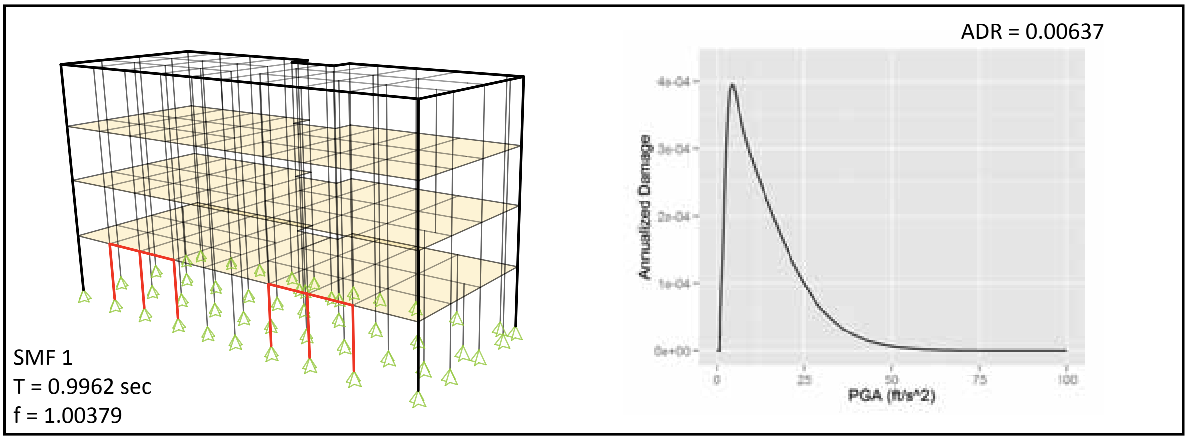


Figure 5.2.7. Steel Moment Frame 1 Results.

5.2.7 Retrofit 6- Steel Moment Frame 2

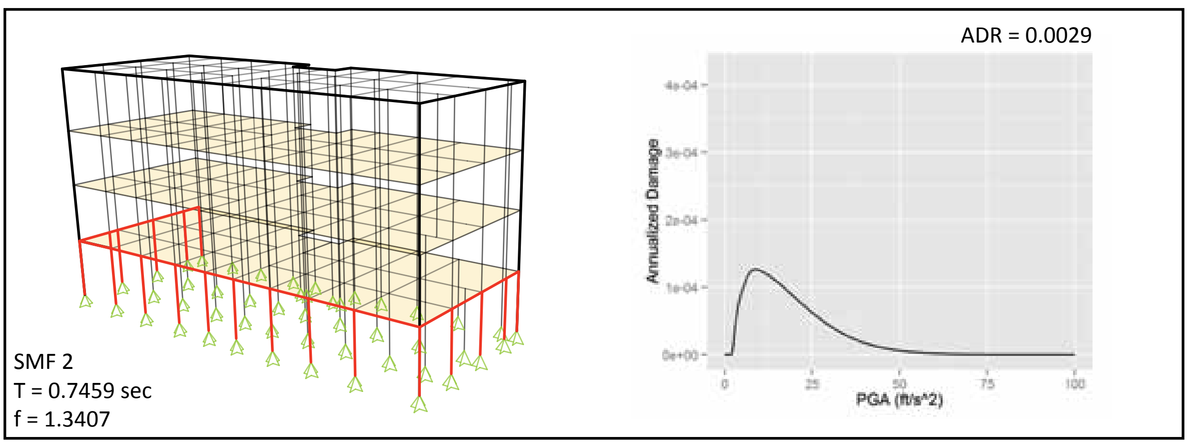


Figure 5.2.8. Steel Moment Frame 2 Results.

Retrofits 5 and 6 show that stiffening the building via steel moment frames can change the damage density quite significantly. In SMF 1, the density function has a slightly lower peak value than the base case, but damage to the building nearly identical to the base case ADR. This is probably due to the type of intervention Retrofit 5 is implementing, which may be adequate in reducing peak drifts but not so for reducing damage loss values. Comparing this to Retrofit 6, the addition of moment frames all along the soft story is able to significantly reduce the peak and overall annual damage values, and simultaneously reduces the fundamental period of the building. This sheds light on the types of retrofits typically called by the Department of Building Inspection and the ATC as minimal upgrades to prevent from building collapse, which is employed in Retrofit 5 of this study. Although collapse is prevented and interstory drifts reduced, the overall building damage is not any different from the base case. This is precisely the type of information that is necessary, and typically missing, when making informed decisions about the lifetime of a building and its applied retrofit.

5.2.8 Retrofit 7- Shearwall 1

This scenario utilizes plywood shear panels to lateral walls of the soft-story.

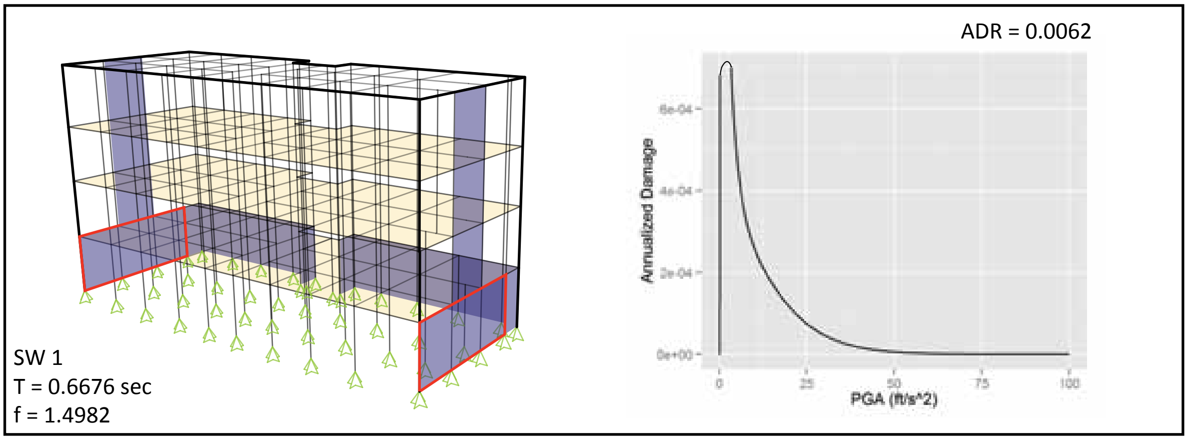


Figure 5.2.9. Shearwall 1 Results.

5.2.9 Retrofit 8- Shearwall 2

Oriented Strand Board (OSB) panels have been applied to the lateral walls of the soft-story.

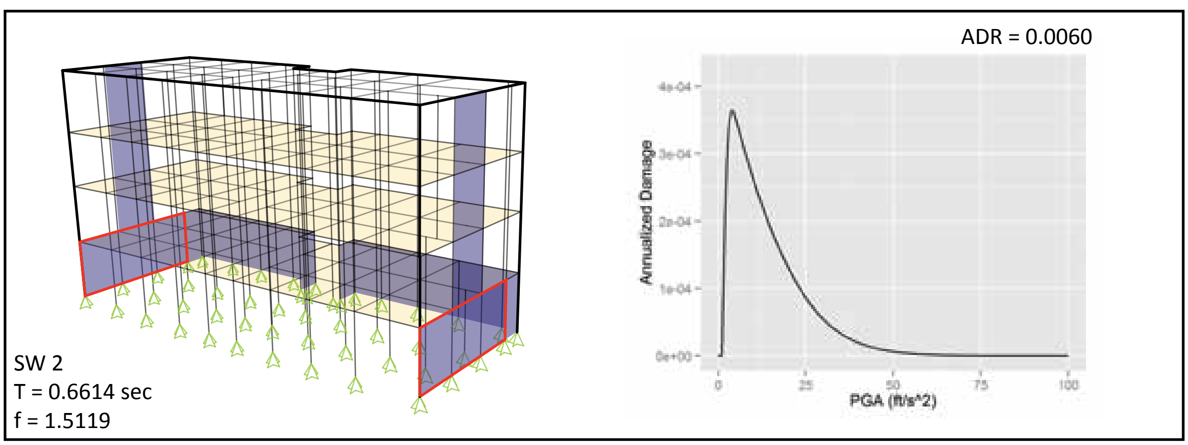


Figure 5.2.10. Shearwall 2 Results.

In the shearwall retrofit cases, the oriented strand board (OSB) panels show marginally better results in comparison to the base case. OSB has been modelled and shown to be stronger elastically than plywood, decreasing the ADR. Interestingly, SW 1 with plywood panels shows to have a higher annualized damage value than that of the base case, albeit a sharply declining curve. This is possibly due to the building responding severely to a very particular intensity earthquake, causing a spike in damage.

5.2.10 *Retrofit 9- Composite 1*

The steel moment frame to facade and OSB panels to lateral wall retrofit option is similar to Retrofit 2 delineated by the Applied Technology Council report 53-2A.[5] One notices over a 33% decrease in fundamental frequency as compared to the base case.

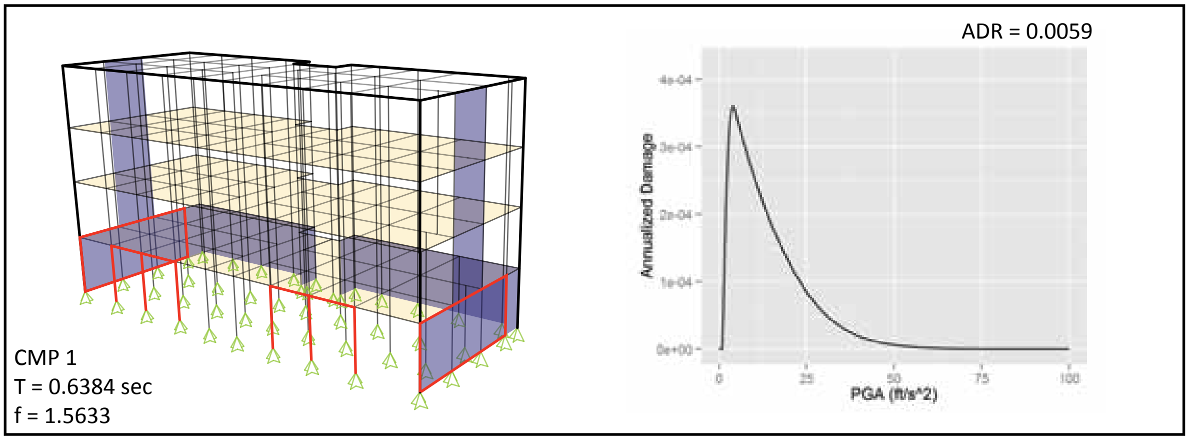


Figure 5.2.11. Composite 1 Results.

5.2.11 *Retrofit 10- Composite 2*

This retrofit is similar to the previous, aside from steel moment frames being placed all along the garage and lateral sides of the soft-story.

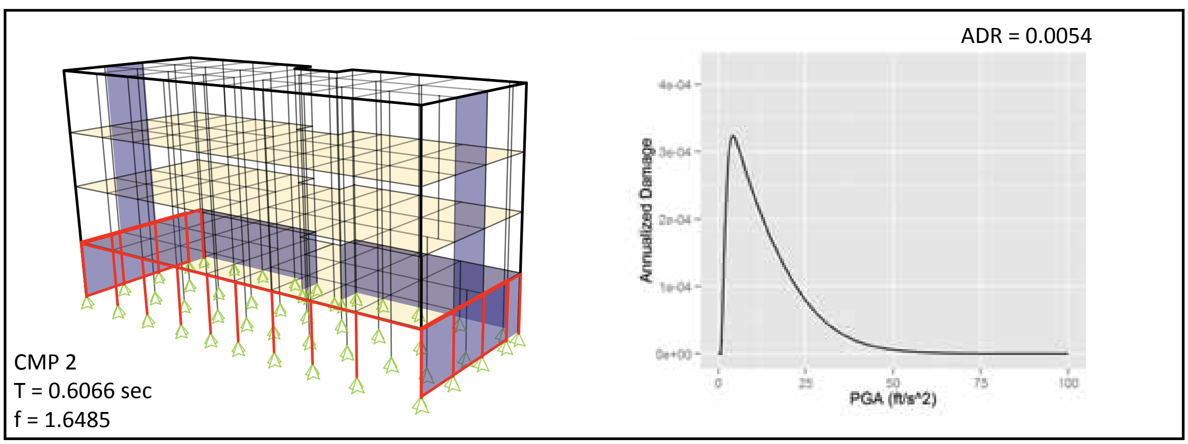


Figure 5.2.12. Composite 2 Results.

Both of these curves show nearly identical behavior aside from the peak damage value being lower in the case where additional stiffening is provided via steel members. The peak ranges around 6-8 feet per square second ground acceleration, similar to the base case condition.

5 Applied Technology Council. *Here Today- Here Tomorrow: The Road to Earthquake Resilience in San Francisco; Potential Earthquake Impacts: Technical Documentation 52-1A* (Redwood City, CA; Applied Technology Council, 2010).

5.2.12 Retrofit 11- Composite 3

Composite 3 adds OSB sheathing to the longitudinal veranda wall and soft-story lateral walls along with moment frames on the first level.

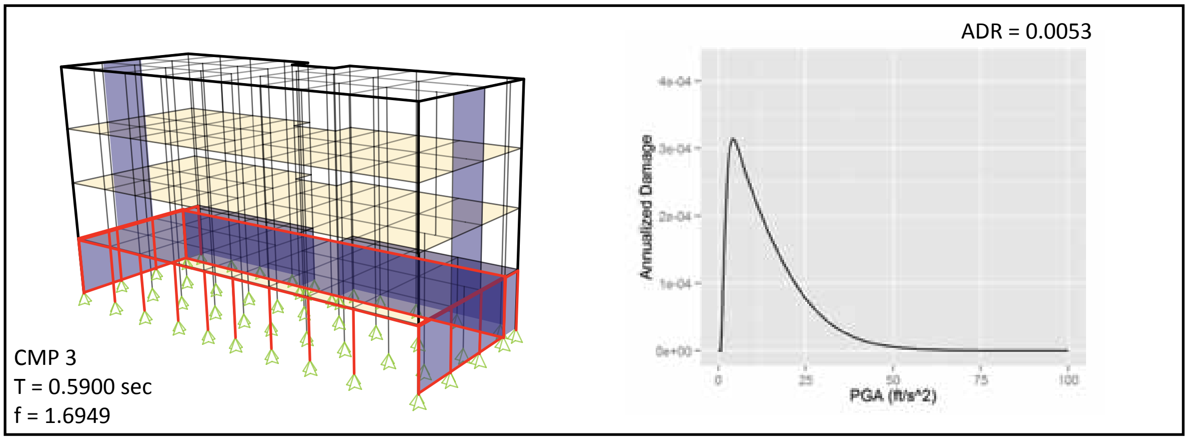


Figure 5.2.13. Composite 3 Results.

5.2.13 Retrofit 12- Composite 4

Composite 4 has similar conditions to the Composite 3 retrofit option, and adds an additional 20% global damping with four viscous dampers applied to the soft-story.

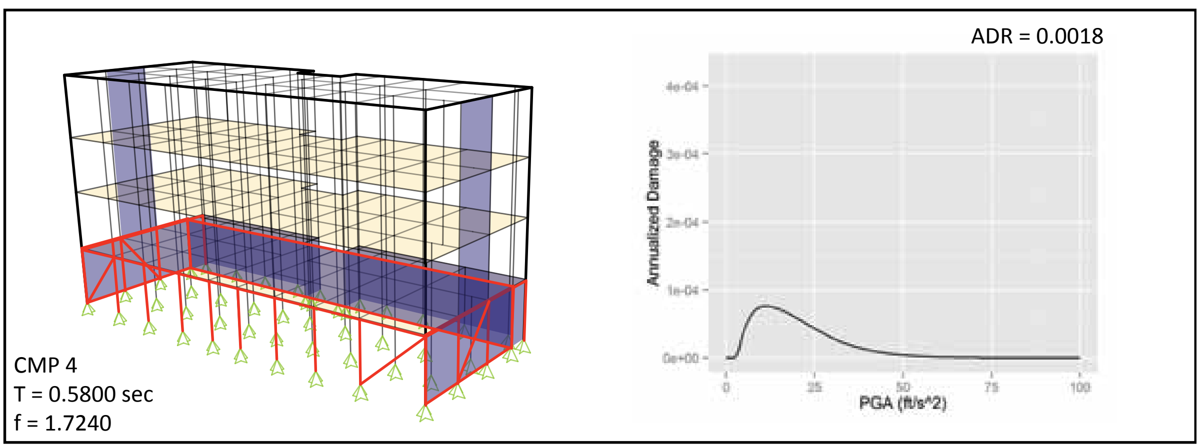


Figure 5.2.14. Composite 4 Results.

The addition of dampers seems to cause appreciable reduction on damage values, and heavily resembles the 20% damping (DMP 0.20), albeit an even lowered peak. Comparing this to the Composite 3 retrofit, it can be assumed that the damping far outweighs shear walls alone in terms of reducing peak damage losses. It also indicates that stiffening the building via shear walls or other bracing elements causes the properties of the building, such as fundamental period and frequency, to change rather unpredictably. In this way, engineers sometimes prefer to use viscous dampers as an energy dissipator in that it does not alter the inherent characteristics of the structure, and are simultaneously able to reduce engineering demand parameters such as peak drift or peak floor acceleration in a somewhat more calculable manner.

5.2.14 Retrofit 13- Composite 5

Composite 5 adds OSB shear walls to all but one exterior facade. It also includes minimal moment frames applied to the garage front of the soft-story.

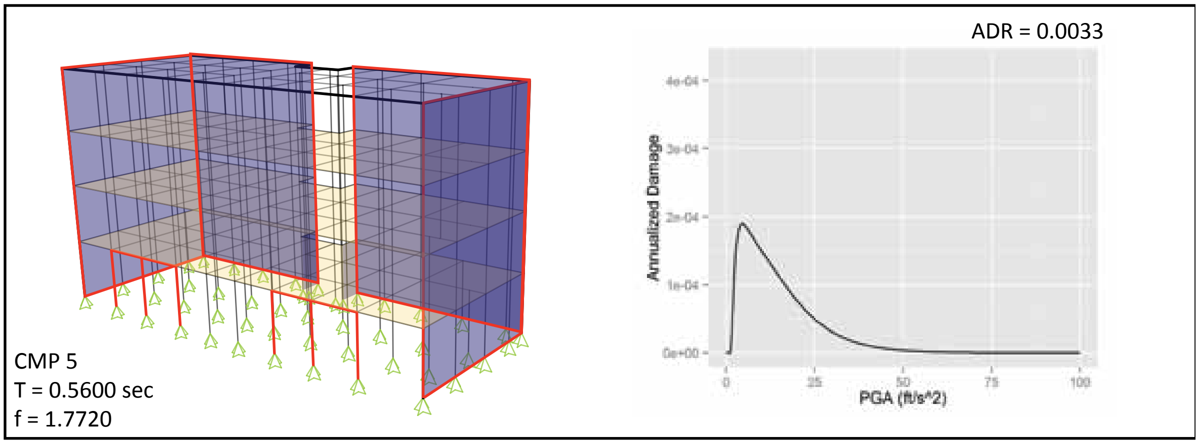


Figure 5.2.15. Composite 5 Results.

In Retrofits 9-13, all Composite schemes provide additional stiffening, with one also providing 20% damping. These cause the annual damage ratio to decrease from the base case, rather strikingly in the case of stiffening with damping. Stiffening the building versus providing damping causes very different response behavior for the woodframe structure, in that the former sheds peak values of annual damage whereas damping provides notable reduction in the damage density curve.

5.2.15 Retrofit 14- Base Isolation

This case involves addition of base isolators at every column with sub-grade connections.

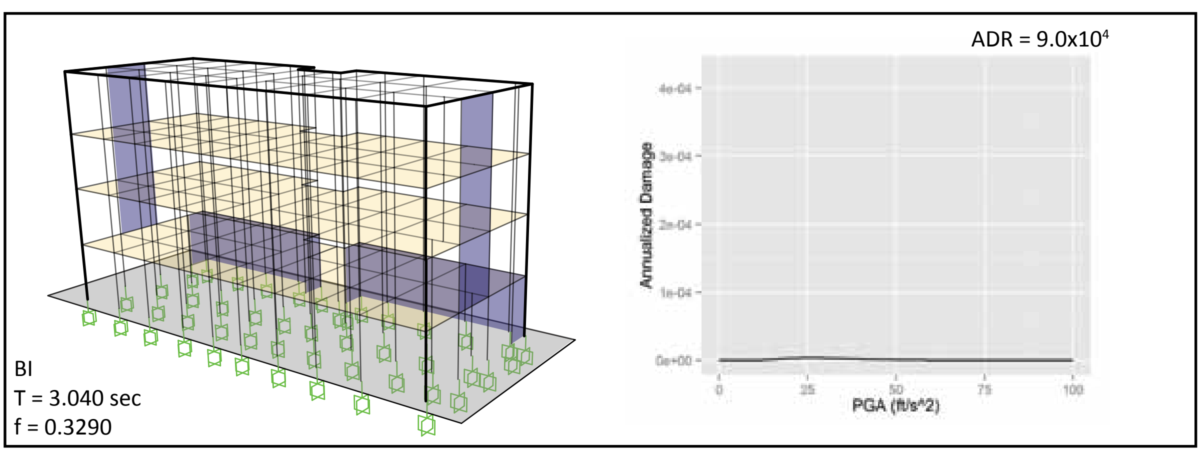


Figure 5.2.16. Base Isolation Results.

Base isolation increases the fundamental period by a factor of three, and decreases the annual damage to nearly negligible values when compared with the original case. Figure 5.2.15 shows a scaled image of the density curve for base isolation. The annual damage ratio is two orders of magnitude less than the base case and all other retrofit simulations. However, as will be described in the next section, the cost of such building performance can be high and therefore, base isolation tends to be the least implemented form of retrofit especially in existing residential buildings. It should be noted that not all buildings can feasibly incorporate base isolation, as it depends heavily on the structure.

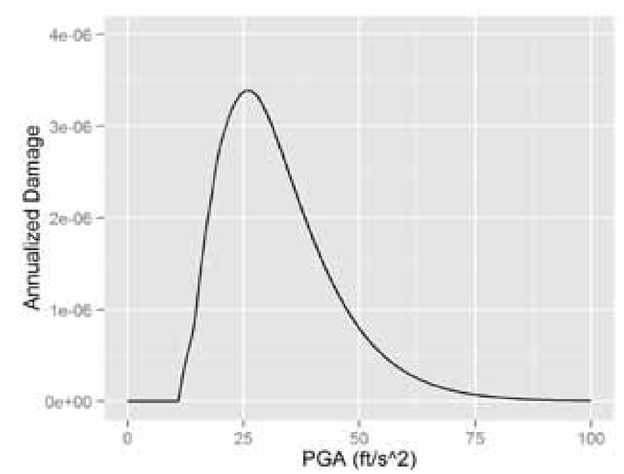


Figure 5.2.17. Base Isolation Damage Density Function.

Table 5.2.2 summarizes the results for comparison, with the base case highlighted in yellow. The results show that each retrofit provided some improvement from the lifetime damage cost of the original structure. The table also includes pertinent information on drift limit assessment for the building, which is used in code-based design. In this case, Connor[6] has delineated typical limit states based on structural drift to evaluate the building's performance. This more traditional method of performance assessment can have vastly different results than what a true performance based probabilistic assessment can provide. Drift thresholds and drift state information is a useful comparison between code-based and probabilistic seismic assessment. The results from this research show that a range of damage can occur given that minimum Life Safety requirements are sufficiently met. This gives great impetus in comparing and contrasting code-based requirements with damage-based consequences of such codes on the lifetime financial repercussions of the structure. In other words, the codes by which most of today's buildings are developed or retrofitted underestimate or disregard the financial underpinnings of the guidelines they require or prescribe. This is one of the primary reasons performance based design was initiated, to better understand the building's behavior in seismic events as well as to find a means to quantify losses including and beyond dollar amounts. These quantifications provide invaluable information on the implications of building damage, and include such decision parameters such as business downtime, casualties as well as life time costs.

6 Connor, J.J. *Introduction to Structural Motion Control* (Prentice Hall, 1st Edition; 2002).

Table 5.2.3 indicates the typical interstory drift limits and limit states as suggested by Connor and to be compared with the "Drift Limit" and "Drift Limit State" columns in Table 5.2.2.

Retrofit Type	Annual Damage Ratio (ADR, x10^{-3})	Drift Limit	Drift Limit State	Lifetime Damage Cost	Lifetime Cost Decrease from Base	Retrofit Cost	Total Cost (Damage + Retrofit)
Base Case	6.39	0.0077	Collapse	$23,347.84	-	-	$23,347.84
DMP 0.1	4.44	0.0058	Life Safety	$16,231.60	30.48%	$3,630.05	$19,861.65
DMP 0.15	3.76	0.0046	Operational	$13,732.62	41.18%	$9,495.65	$23,228.27
DMP 0.20	2.65	0.0037	Operational	$9,669.59	58.58%	$10,062.42	$19,732.01
DMP 0.30	1.92	0.0026	Operational	$6,996.02	70.04%	$11,136.00	$18,132.02
SMF_1	6.37	0.0053	Life Safety	$23,293.79	0.23%	$26,452.30	$49,746.09
SMF_2	2.92	0.0032	Operational	$10,678.84	54.26%	$52,904.60	$63,583.44
SW_1	6.23	0.0047	Operational	$22,762.36	2.51%	$5,819.40	$28,581.76
SW_2	5.96	0.0047	Operational	$21,787.11	6.68%	$5,380.40	$27,167.51
CMP_1	5.86	0.0049	Operational/ Life Safety Boundary	$21,425.86	8.23%	$27,782.60	$49,208.46
CMP_2	5.44	0.0045	Operational	$19,866.49	14.91%	$58,285.00	$78,151.49
CMP_3	5.3	0.0044	Operational	$19,373.67	17.02%	$63,665.40	$83,039.07
CMP_4	1.81	0.0013	Fully Operational	$6,602.72	71.72%	$73,727.82	$80,330.54
CMP_5	3.29	0.0008	Fully Operational	$12,006.07	48.58%	$47,973.90	$59,979.97
BI	0.09	0.0003	Fully Operational	$329.17	98.59%	$852,600.00	$852,929.17

Table 5.2.2. Tabulation of Final Results.

Limit State	Drift Limit
Fully Operational	1/500
Operational	1/400
Life Safety	1/200
Near Collapse	1/100

Table 5.2.3. Drift Limits and Limit States.

Base isolation is the clear winner from all retrofit options in terms of reduction of lifetime damage cost, a mere 1.5% of the damage cost for the original building. However, the retrofit costs are much higher than any other retrofit intervention. At a conservative cost of 7% of the building's value, it is not surprising, then, that the base isolation has the highest up front cost in comparison to all other scenarios. Contrasting the behavior of the base isolated system on the building (Figure 5.2.16) show that the isolators (right image) behave as the sacrificial floor, similar to the soft-story floor in the original case. However, since the isolators sit below grade, minimum damage or loss has occurred, and importantly, the building atop moves together as one mass. Referring back to the annual damage density curves, one can now begin to see how the peak annualized damage value in a base isolated situation is capped off, as peak interstory drift is greatly limited via such a system.

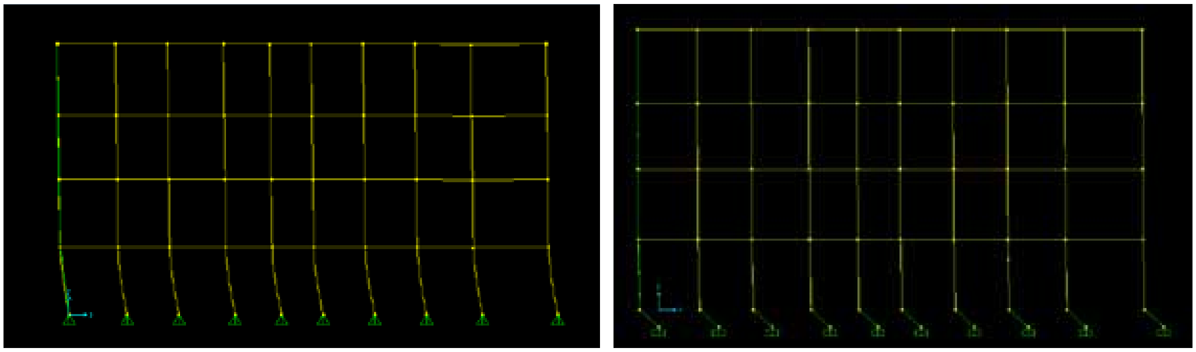


Figure 5.2.18. Comparison of Base Case to Base Isolation, SAP2000.

Examining some of the other retrofit options shows that very few of the cases result in total costs less than the original building's lifetime damage. For instance, only three out of the fourteen options had a total dollar value that is significantly lower than the base case, mainly because of high retrofit costs for most of the systems.

This presents a pressing and prevalent engineering dilemma: one of balancing a structure's health with the cost of its improvement. In other words, while the engineer may believe that base isolation is the best route to take if the building is to remain resilient for its 50 year lifetime or beyond, those making decisions such as homeowners or policy makers will only consider the balance sheet, and opt for a less robust retrofit option. It is difficult to predict the operational lifetime of some of the mentioned retrofit options, mainly because the deterioration of the components are not easily quantifiable. Therefore, while the shear wall options may seem desirable because they have low retrofit costs and achieve an acceptable drift limit state, the fact that these may need to be replaced more than once in the lifetime of the building compounds that retrofit cost, which may not be the case for more resilient interventions such as base isolation. In addition, the incremental degradation of the whole structure is probably accelerated when low-level retrofits are utilized, further increasing the lifetime damage cost of the building. For these simulations, it is challenging to capture this age-based weakening of the structure, and therefore, difficult to use retrofit degradation as a parameter in evaluating the various mitigation options.

5.3 Damage Value Comparison

Though the annualized damage value can provide relative assessment between the different retrofit options, it is also useful to understand what comprises these damage values. As engineers, parsing the damage value into its inherent components, structural and non-structural, can provide insight into the building's behavior. This information is useful when making decisions about cost-benefit-and behavior of seismic mitigation actions. Table 5.3.1 lists the breakdown of the total annualized damage value into its structural, non-structural drift-sensitive, and non-structural acceleration sensitive components.

Retrofit Type	Struc- ADR, 10^{-4}	% Total, Struc-ADR	Drift- ADR, 10^{-4}	% Total, Drift-ADR	Acc- ADR, 10^{-4}	% Total, Acc-ADR	TOTAL ADR, 10^{-4}
Base Case	6.9	10.80%	31.3	48.98%	25.7	40.22%	63.9
DMP 0.1	4.32	9.73%	19.9	44.82%	20.2	45.50%	44.4
DMP 0.15	3.07	8.16%	14.3	38.03%	20.2	53.72%	37.6
DMP 0.20	2.33	8.79%	10.9	41.13%	13.2	49.81%	26.5
DMP 0.30	1.51	7.91%	7.10	37.17%	10.5	54.97%	19.1
SMF_1	5.74	9.01%	26.2	41.13%	31.8	49.92%	63.7
SMF_2	3.29	11.27%	15.4	52.74%	10.5	35.96%	29.2
SW_1	3.52	5.65%	16.4	26.32%	42.4	68.06%	62.3
SW_2	3.50	5.87%	16.3	27.35%	39.8	66.78%	59.6
CMP_1	3.47	5.92%	16.1	27.47%	39.1	66.72%	58.6
CMP_2	2.72	5.00%	12.7	23.35%	38.9	71.51%	54.4
CMP_3	2.64	4.98%	12.3	23.21%	38.0	71.70%	53.0
CMP_4	0.50	2.76%	2.40	13.26%	15.2	83.98%	18.1
CMP_5	0.05	0.15%	0.21	0.64%	32.6	99.09%	32.9
BI	0.03	3.33%	0.13	14.44%	0.75	83.33%	0.90

Table 5.3.1. Annualized Damage Value.

This tabulation is a significant indicator of the building's behavior as various retrofit options are introduced. A primary observation is that non-structural induced damage (Drift-ADR and Acc-ADR) far outweigh structural damage in overall damage ratios. This is commonly exhibited in similar seismic damage assessments that categorize damage in this manner. Also important to note is that as the building is stiffened via shear walls and steel moment frames, the percent of damage attributed by acceleration-sensitive non-structural damage is increased. Lowering the fundamental period of the building by stiffening the structure causes it to "feel" the seismic excitation more. In general, although the damage to the structure is visibly reduced, the damage to interior partitions, walls, ceiling tiles and other non-structural elements is severe and accounts for much of the building's total cost. Therefore, any decision to strengthen the building must take into consideration the implications of potentially increasing non-structural damage to the building. As HAZUS® damage values are typically low, the Appendix has included a sensitivity assessment of the above results as comparison.

5.4 Validation

The Applied Technology Council along with the Department of Building Inspection and SEAOC spearheaded the primary surveys and studies on wood-frame structures in San Francisco. They have specified retrofits similar to those mentioned in Porter and Cobeen[7], and therefore, have inspired some of the seismic mitigation techniques in this report. Table 5.4.1 specifies retrofits and the subsequent elastic period of the building[8], which can be used to evaluate the feasibility of this study.

Index Building	D_y	A_y	D_u	A_u	T_E	μ
Building 2. 4-story corner building both sides > 50% open, walls between garages	0.51	0.05	0.71	0.06	1.00	1.38
Index Building 2 Retrofit 1. Ditto with new steel frames both facades	0.68	0.11	1.03	0.13	0.80	1.51
Index Building 2 Retrofit 2. Ditto but wood shearwalls all interior garage walls	0.84	0.20	2.09	0.31	0.65	2.49
2r3. Ditto but cantilever columns at garage openings instead of moment frames	0.99	0.23	2.62	0.37	0.67	2.64

Table 5.4.1. Capacity Curve Parameters for Four Index Buildings. Table 8-1, ATC 53-2A.

The fields outlined in orange are of particular interest to this research. Retrofit one in ATC 53-2A is similar to SMF 1 in this research, in which case the period of the building after application of steel moment frames to the facade is about 0.996 seconds. This is higher than the value noted by the Applied Technology Council, and may be related to specific connection detailing that could not be captured in the SAP2000 model. However, Retrofit 2 is analogous to Composite 1 in this study, and has a very comparable elastic period to one found in this report. Composite 1 found a natural period of 0.64 seconds, and provides a good estimation of the possible improvements in the case study building. In addition, both base case natural periods are about the same in each study, 1.00 seconds in the ATC pushover tests, and about 1.11 seconds in this study.

An important distinction should be made between the motivations for each study, given that retrofits are assessed for the same building. As is noticed, Retrofit 3 in the ATC report produces the best results, with a natural period reduction of 33%. In this research, the retrofit options suggested are much more incremental to provide better insight into behavior and damage values of the structure. The ATC had consulted engineers, architects, and surveyors when deciding to use these particular retrofits, because they seemed to be the most pragmatic in strengthening the index buildings against seismic action. In this report, the building retrofits may not necessarily be as feasible, but given an optimized and linearized seismic assessment procedure, many more iterations are possible to fully understand the scope and variation of seismic mitigation. Therefore, while some of the retrofits suggested in this study may not be the most reasonable in reducing building damages, the research has taken full advantage of the probabilistic seismic assessment methodology detailed in Chapter 3 to provide many more trials

7 Porter, K. and Kelly Cobeen. *Informing a Retrofit Ordinance: A Soft-Story Case Study* (Structures Congress, ASCE; 2012).

8 Building 2 in this ATC 53-2A is the same typology as the case study building in this research.

of retrofit design. This allows for a holistic understanding of the building's structural behavior, and subsequently, its lifetime damage costs.

5.5 Retrofit Costs

In providing basic cost estimates for lifetime damage of the building, costs for the various retrofits needed to be researched. Very detailed cost information for the stiffening retrofits could be found in ATC 53-2A and are presented in the Appendix. For cases in which damping has been applied, some assumptions about the cost of the dampers were made. Ghisbain[9] has provided methods by which global damping in a system could be translated into damper placement via a particular inner and outer optimization process. In a simplified version of this method, global damping is applied following the design and placement of the dampers within the simulation model. From these dampers, axial forces could be extracted from which costs for the viscous dampers are determined. Information gathered by Connor[10] has estimated a linearized relationship between the inherent force in the damper and its respective cost. The following figure describes this relationship, which has been used in identifying costs for retrofits in this research. It should be noted that these values are estimates, because actual costs are produced by the manufacturer and heavily depend on the particular building and design of the damper.

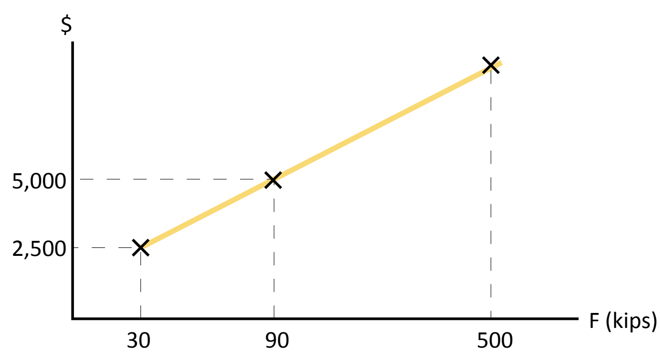


Figure 5.5.1. Cost of Viscous Dampers.

It is useful to understand the physics behind the viscous dampers so as to distinguish the variables that allow the damper to absorb energy in the system. The value of ***c***, the coefficient of viscous damping, is dependent on the amount of damping provided as related by the following equation:

$$\xi = \frac{c}{2\sqrt{km}}$$

9 Ghisbain, Pierre. *Seismic Performance Assessment for Structural Optimization* (Cambridge, MIT; Doctoral Thesis, 2013).

10 Connor, J.J., Professor of Civil Engineering, MIT.

This means that as ξ(xi), or the amount of damping, increases, so too does the coefficient of viscous damping. This relationship of force to the coefficient of damping is by:

$$F = cv$$

Where F varies linearly with ***v***, the velocity of the damper. As ***c*** is increased, ***v*** is decreased and vice-versa. Therefore, as damping is increased in the structure, velocity decreases and the axial force essentially remains the same. Therefore, a correct balance must be made in the parameters that allow the viscous damper to work such that the cost of the dampers provide useful benefit to the seismic mitigation intended for the structure.

Finally, in the case of base isolation costs, Connor[11] has estimated base isolation costs of 5% of building value for the San Francisco City Hall. In another report, Robinson Seismic Limited in New Zealand has reported a 3% building value cost for a new structure.[12] An upper bound value for an existing building in a dense urban milieu could be up to 7% building value. This report uses an estimate of 7% of building value for a cost of $853,600. The more conservative value hopes to include the cost of excavation since the building is existing and will require more attention for such a retrofit. Albeit that the base isolation retrofit is the most costly, the induced lifetime damage with isolators is significantly lower than other alternatives, which maintains that base isolation is the most effective retrofit option for the health of the building.

This reiterates the complex issue of managing building longevity and health with the cost of achieving that resilience. The following chapter embellishes this notion, and provides city-wide cost data given the nature of the case study building presented here.

11 Connor, J.J. *Introduction to Structural Motion Control* (Prentice Hall, 1st Edition; 2002). Chapter 5.

12 Devine, M., *Costs and Benefits of Seismic / Base Isolation* (Wellington, New Zealand; Robinson, Ltd., 2012).

06
DISCUSSION

6.0 Discussion

6.1 Performance Based Assessment of Existing Structures

Evaluating a building beyond its life-safety expectations has become increasingly important mainly because post-disaster rebuilding and repair can be tremendously expensive and impact the vitality of a city and its peoples. Performance based assessments provide an alternative optimization strategy, not necessarily a prescriptive approach as is done in code-based design, but one that quantifies future losses given probabilistic seismic events. Geological studies and finite understanding of building behavior has improved significantly in the past years as performance based engineering had become popularized. Ghisbain[1] has demonstrated that performance based assessments can be significantly improved if multiple iterations are feasible, which can be achieved via a linearized method of response measure to then find damage costs. This research is mainly interested in applying this methodology to existing buildings in vulnerable contexts, and has therefore chosen San Francisco as a dense urban environment where seismic hazards are prevalent. This study finds that application of an optimized seismic assessment can be applied to existing woodframe structures, and that shortened runtimes[2] for simulations did not compromise the breadth of the study.

6.2 Retrofit Feasibility

Given that this investigation is dually interested in applying an optimized structural performance methodology as it is to wood-frame building resilience, much of this research intended to analyze the implications of both the methods and the results of those methods. As this study focused on retrofitting wood-frame structures, many improvement options have been suggested but few may have realistic applicability in San Francisco. This is because the costs of some forms of retrofit do not wholly consider inflation costs of construction, and temporary housing of residents of woodframe buildings and business interruption. However, these retrofit scenarios can provide considerable insight into the incremental improvement provided by seismic mitigation alternatives. Other reports have chosen retrofits based on professional advice and expertise, and have limited to mainly two or three variations. This study chose to expand those retrofit options, to some that may not be wholly feasible, but which are necessary in order to appraise the building's behavior under seismic loading and its lifetime damage costs.

6.3 Cost Implications for City of San Francisco

Due to the dense nature of the housing in San Francisco, it is useful to quantify the losses in whole-city terms given the information the report provides. The following map shows locations of woodframe structures with 3 stories or more and 5+ housing units, along with NEHRP soil classes.

1 Ghisbain, Pierre. *Seismic Performance Assessment for Structural Optimization* (Cambridge, MIT; Doctoral Thesis, 2013).

2 Each retrofit simulation took approximately 1 minute 14 seconds in SAP2000, considerably less than a non-linear simulation.

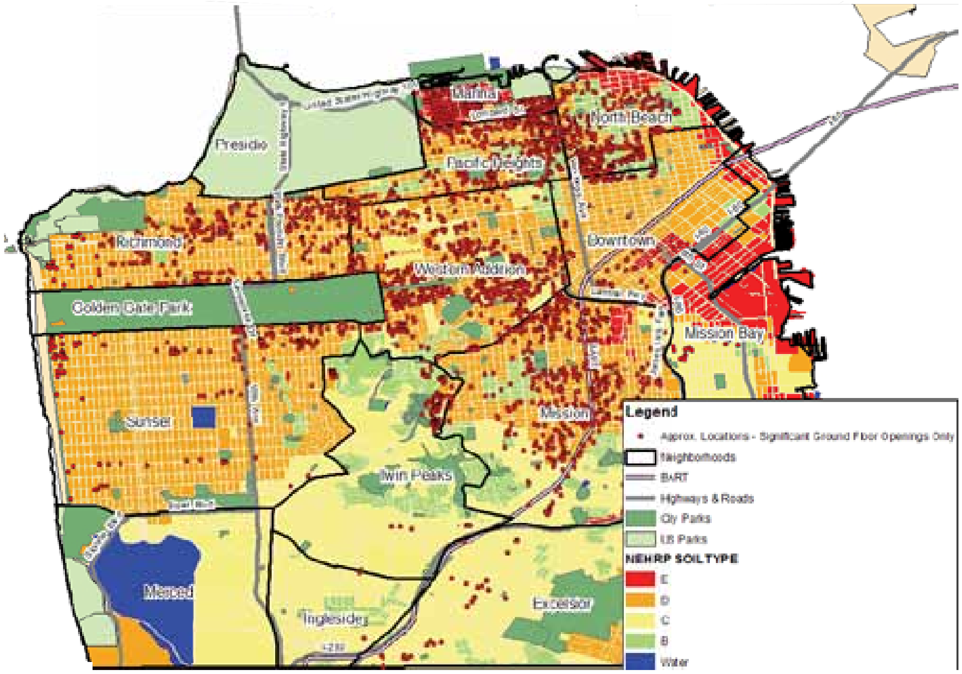


Figure 6.3.1. Woodframe Buildings with 3+ Stories and 5+ Residential Units with Significant Ground Floor Openings; NEHRP Soil Class Data. ATC 53-2A.

The United States Geological Survey has defined the NEHRP soil classes as having the following implications in the Bay Area.

Soil Type A	Vs>1500 m/sec	Includes unweathered intrusive igneous rock. Occurs infrequently in the Bay Area. Soil types A and B do not contribute greatly to shaking amplification.
Soil Type B	1500 m/s > Vs > 750 m/sec	Includes volcanics, most Mesozoic bedrock, and some Franciscan bedrock.
Soil type C	750 m/sec > Vs 350 m/sec	Includes some Quaternary sands, sandstones and mudstones.
Soil Type D	350 m/s > Vs > 200 m.sec	Includes some Quaternary muds, sands, gravels, silts and mud. Significant amplification shaking by these soils is generally expected.
Soil Type E	200 m/sec > Vs	Includes water-saturated mud and artificial fill. The strongest amplification of shaking due is expected for this soil type.

Vs is shear wave velocity;

Table 6.3.1. Description of NEHRP Soil Classes in Bay Area. USGS. http://earthquake.usgs.gov/regional/nca/soiltype/

This information shows that seismic excitation cannot be decoupled from soil class and amplification, making many woodframe residences vulnerable to significant damage and/or collapse. Comparing several of the retrofit options to the base case scenario can provide City-level decision makers with overall loss expectations such that appropriate seismic mitigation can take place. Figure 6.3.3 shows a comparison on a city level to lifetime costs associated with the Steel Moment Frame 1, Composite 4 and Base Isolation retrofits.[3] These options were chosen as they represent increasingly complicated retrofits, with SMF 1 being a minimum intervention as delineated by the ATC. Base Isolation provides the lowest lifetime damage costs, yet has considerably high retrofit cost. The number of buildings that have the same qualities as the case study building in this study is 626 buildings with 8,184 residential units in the City of San Francisco[4].

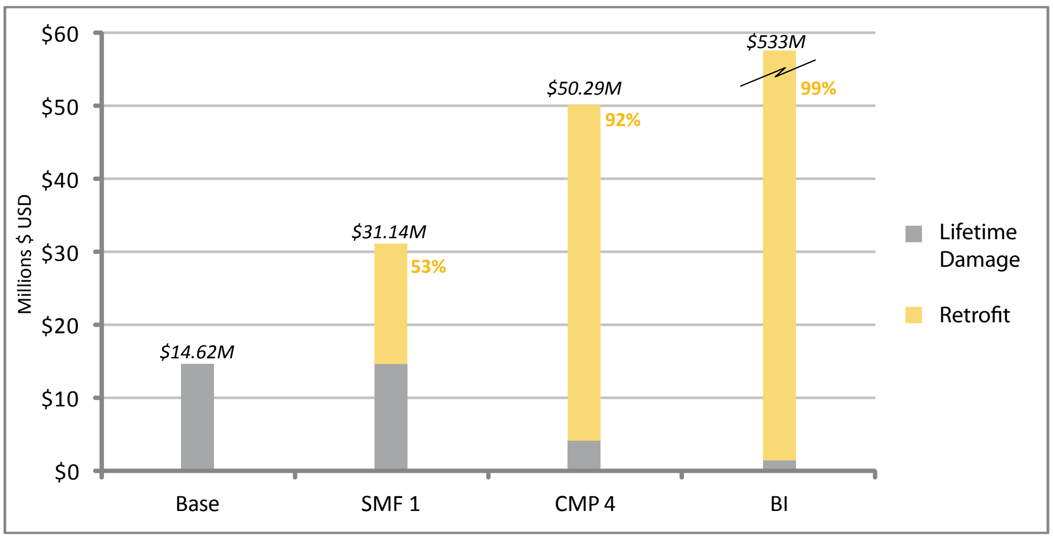


Figure 6.3.3. Lifetime Loss and Retrofit Cost Comparison, Citywide.

The graph shows that the largest upfront cost is that of the base isolation, which also has the lowest lifetime total cost. SMF 1 case has nearly equal lifetime damage as retrofit cost, while CMP 4 has relatively high seismic mitigation costs as compared to the lifetime damage of the building. As in many cases, decision-makers must contend with larger front-end costs for citywide seismic resilience, verses fewer costs accumulating throughout the life of the structure.

A damage density function comparing the base case to the three retrofits described previously is shown in Figure 6.3.4. Interestingly, the behavior of the Steel Moment Frame 1 application is nearly coincident with the base case, with a slightly lower peak value. As suggested in the previous chapter, this may be due to the fact that the steel moment framing can help in limiting interstory drift, but are not sufficient enough a retrofit to significantly reduce the non-structural damage that may cause the overall

3 See Chapter 5 for explanation of retrofits.

4 Applied Technology Council. *Here Today- Here Tomorrow: The Road to Earthquake Resilience in San Francisco; Potential Earthquake Impacts: Technical Documentation 53-2A* (Redwood City, CA; Applied Technology Council, 2010) Table 2-5.

annual damage to be high. The Composite 4 case has a implemented nearly all stiffening mechanisms prescribed to the soft-story, and has introduced 20% damping to significantly reduce the yearly losses due to damage. Damping provides substantive decrease in non-structural drift damage, which balances out with the increase in non-structural acceleration based damage allowing for a visible reduction in the damage density function. Finally, the base isolation system disallows the structure above grade to respond to the seismic action as it absorbs the impulses below grade, causing it to have minimal to no induced damage from mid- to high-range earthquakes.

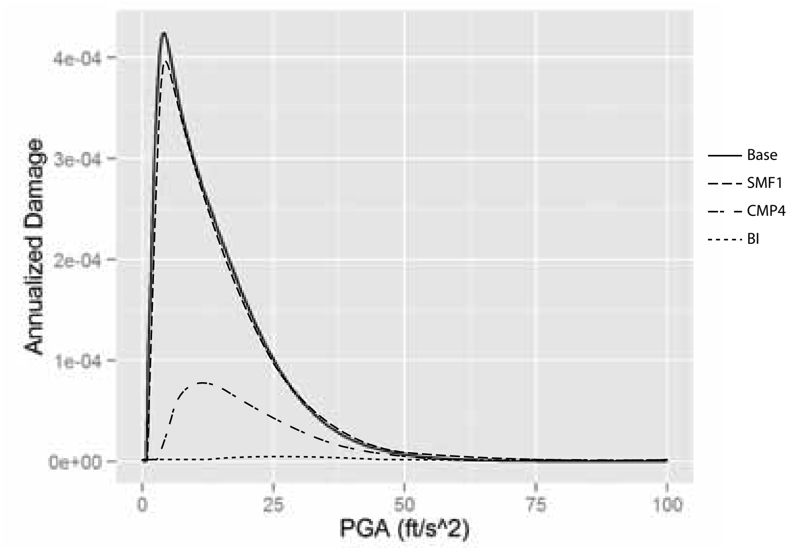


Figure 6.3.4. Comparison of Annualized Damage Density.

As many city officials, organizations, and inhabitants of the City of San Francisco realize, action must be taken to ensure that homes are lives are protected in seismic events. As such, research and advocacy on retrofitting has been pushed by organizations such as the San Francisco Planning and Research Association (SPUR), especially after the ATC conducted intensive surveys of San Francisco's building stock. SPUR finds that engineering guidelines on building performance, usually code-based, are too vague for communication with the general public. The performance guidelines described in Chapter 2 as delineated by SPUR are simpler to understand, and have been included in the Porter and Cobeen report cited in this research. In their report on a soft-story retrofit ordinance, Porter and Cobeen[5] relate the various SPUR performance levels to some of the retrofit options they have suggested and tested in their study. Two of the three retrofit options are analogous to those presented in this study. Retrofit one that Porter and Cobeen have introduced adds structural sheathing to the same case study building tested in this report, which is similar to the Shearwall 1 retrofit case. In the second retrofit described by Porter and Cobeen, an addition of steel frames is additionally applied to the case study building, and has inspired the Composite 1 retrofit scenario. The following table from Porter and Cobeen indicates how these retrofits perform when classified by SPUR measures.

5 Porter, K. and Kelly Cobeen. *Informing a Retrofit Ordinance: A Soft-Story Case Study* (Structures Congress, ASCE; 2012).

Retrofit	SPUR (2008) performance objective	Cost per unit, $000				Total cost, $ million
		IB1	IB2	IB3	IB4	
1. Add structural sheathing	D, safe but not repairable	$ 20	$ 6	$ 10	$ 12	$180
2. Same plus steel frames	C, safe and usable after repair	30	11	18	15	$300
3. Same by cantilever columns instead of steel frames	C or B, safe and usable during repair	28	9	16	15	$260

Table 6.3.2. Retrofit Alternatives and Costs
Source: Porter and Cobeen, 2012, Table 1.

IB2 is the same case study building in the Porter and Cobeen study as the one in this investigation. The authors have found that SPUR damage states relate to tagging schemes following earthquakes in the following way: levels A & B correspond to green tagging, C to yellow tagging, D to red tags and E to collapse. From this, the application of retrofit scheme SW1 and CMP1 can be interpolated as the building being not immediately occupiable until after repair. This suggests that the residents of these buildings must find interim shelter in our around the City of San Francisco, which then means that job accessibility and overhead costs are increased for the City to accommodate for a transient population following a disaster. Although this research mainly focuses on the lifetime damageability of a discrete building module, the affects on a city-wide scale show that each building retrofit can have very severe consequences for the City's recovery, and have been addressed accordingly by reports such as Porter and Cobeen's, along with SPUR's resilient city initiatives.

6.4 Base Isolation

Given that base isolation provides the best case scenario in terms of lowering lifetime damage cost for woodframe housing, it is surprising that isolation techniques are only used on high-stake buildings (such as City Halls, Hospitals, Fire Departments, etc.). Robinson Seismic has stated that base isolation is typically applied to new structures, but can also be suitable for existing structure, the caveat being that there exist too many variables to give a meaningful indication of cost. In addition, a widely held misconception exists that seismic isolation is expensive. However, when viewed against savings the isolation system can provide, it has very large benefits. Devine of Robinson Seismic details some of the benefits of base isolation:

- "Base isolation allows for a reduction in structural elements of the building with less ductile detailing needed;
- Crawl spaces or basements can have multiple benefits e.g. in siting services, additional income from a carpark, flexibility for future development;
- Protection of contents- with controlled movement caused by seismic isolators contents are not subject to violent and sudden shakes thereby reducing the impacts on the contents;
- Protection of the integrity of the internal structures e.g. stairs, internal walls, partitions;
- Building is safer for occupants and contents are protected;

- Continuity of operations is much more likely."

In terms of maintenance of seismic isolators, Devine[6] argues that:

- "Contrary to belief, seismic isolation devices require no more maintenance during the life of the building;
- Following any significant event they should be inspected to ensure bolts and load plates are still in place;
- Devices do not need replacing after an earthquake unless the event was in excess of their design specification in which case we recommend the removal of some devices for testing;
- Because the building is protected from major damage, repair costs following an earthquake will be lower to non-existent"[7]

It should be highlighted that for this case study, the cost of the base isolation is much higher than the lifetime damage mainly because the building of the value is relatively low in comparison to the typical buildings that have implemented the isolation scheme. Labs, hospitals and historic or civic structures tend to have a greater value and therefore the cost of base isolation has high returns in much lower damage losses. In the case of wood-frame structures, a retrofit subsidy policy must be offered for isolation, such that landlords and homeowners can conceivably afford the retrofit, and without which the forward costs become too high for consideration.

The social incentives that come with base isolation provide higher chances for uninterrupted functionality of buildings and places of work, which for many means continued employment is secured and more individuals can collectively assist in the rebuilding and recovery of their community.[8] There exist many benefits in pushing for seismic base isolation in buildings, however, the front-end costs are significant for any homeowner/landlord or city to conceivably provide for the safety of their building. Structurally, base isolation does not allow the building to greatly feel the impacts of a high ground motion event, and therefore are growing in numbers for new construction in seismically prone regions. In this study, an average of 7% building value cost is for the application of base isolation retrofit, although realistically, this may range anywhere from 3% to 7% of building value.

6.5 Future Work

The investigation conducted on multi-family wood-frame residences in California is certainly a first-attempt in quantifying damageability from incremental improvement analysis of this building typology in dollar loss amounts. Several simplifications have been made, and future work would hope to address the nuances of the structural behavior of this building more thoroughly. In addition, this work could be greatly benefitted by a non-linear evaluation of the same case study building, to compare the

6 Devine, M., *Costs and Benefits of Seismic / Base Isolation* (Wellington, New Zealand; Robinson, Ltd., 2012).
7 *Ibid*
8 *Ibid*

accuracy of the results presented and the runtime outputs of the simulation. Furthermore, the author hopes to conduct expanded studies on vulnerable building typology assessments, such as non-ductile concrete frame and single-family wood-frame damage assessments for a more holistic discernment of loss values in San Francisco.

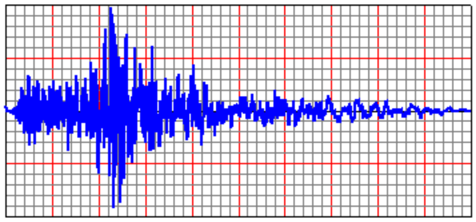
An optimized methodology for seismic improvement is certainly relevant for new construction design iterations, but should not be limited to future buildings. There is great potential in assessing existing structures using probabilistic methods that can determine many scenarios for reshaping the built environment through retrofit. This type of iteration process can open up dialogue for designers, engineers and for those engaged in public policy. Enumerating building response in terms of lifetime damage costs gives engineers the ability to present alternatives understandable to stakeholders, and allows more individuals to be a part of the resilience discussion, which is an important aspect of any retrofit or design optimization. The hope of this research is to reevaluate existing buildings via a streamlined assessment methodology such that the engineer can more appropriately convey the building's performance, and therefore, can provide comprehensive data to catalyze an informed retrofit decision.

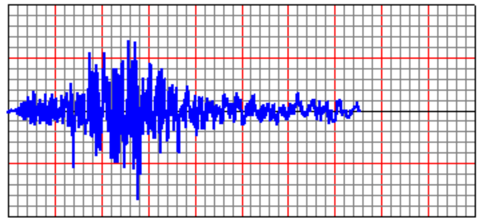
APPENDIX

A.1.0 HAZARDS

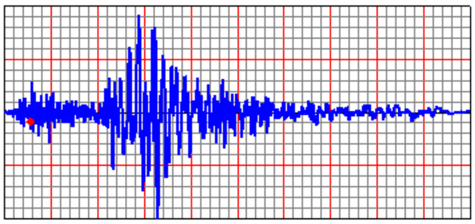
A.1.1. TIME HISTORIES USED IN SAP2000 ANALYSIS

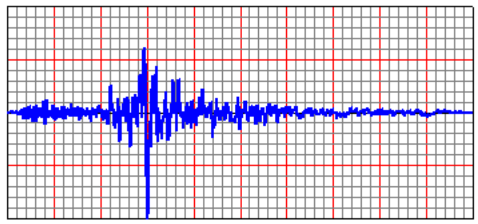
Loma Prieta, 1989, 6.93M. Station Hayward CSUH; U1 (left) U2 (right):



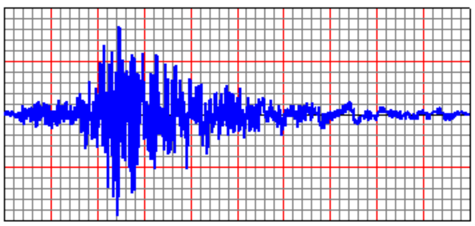


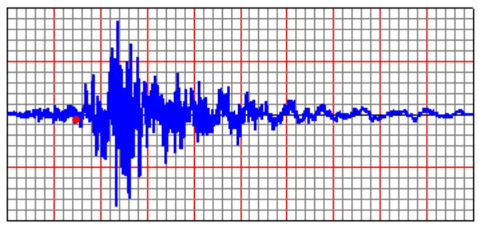
Loma Prieta, 1989, 6.93M. Station SF Presidio; U1 (left) U2 (right):



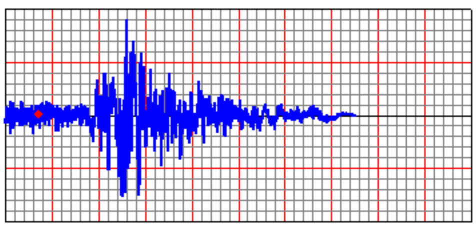


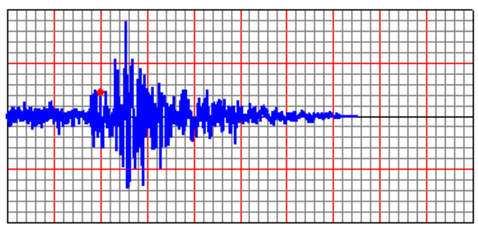
Loma Prieta, 1989, 6.93M. Station Lower Crystal Springs Dam; U1 (left) U2 (right):



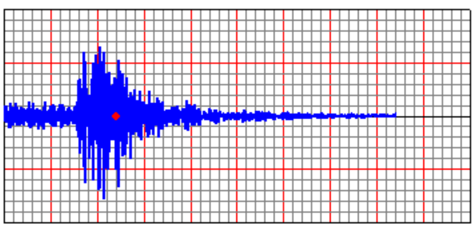


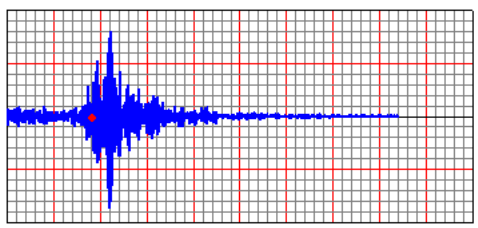
Loma Prieta, 1989, 6.93M. Station 1295 Shafter; U1 (left) U2 (right):



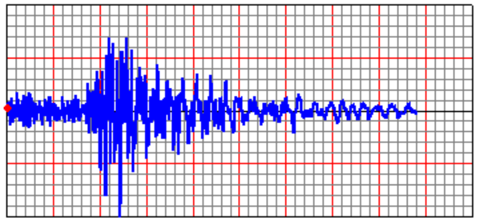


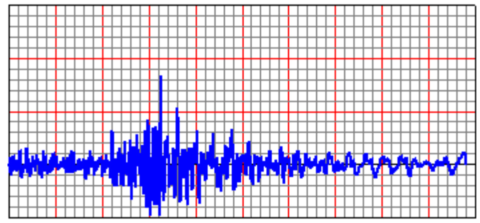
Gilroy, 2002, 4.9M. Station Point Molate Depot Component; U1 (left) U2 (right):



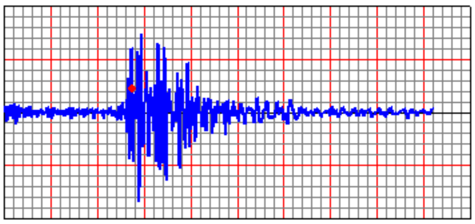


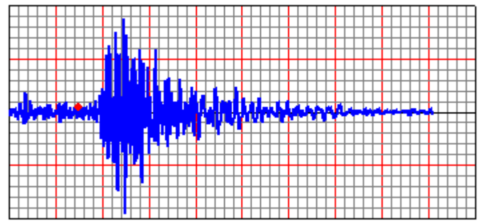
Gilroy, 2002, 4.9M. Station Foster City; U1 (left) U2 (right):



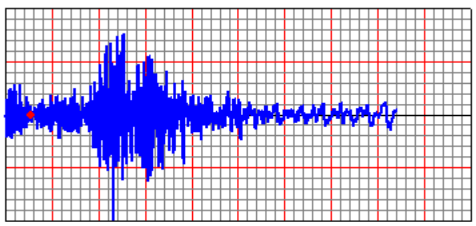


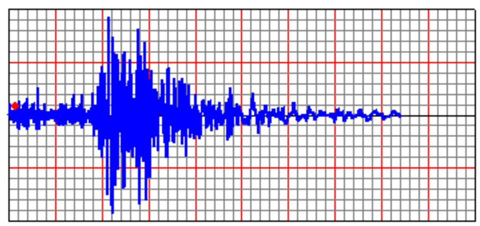
Gilroy, 2002, 4.9M. Station SF Marina School; U1 (left) U2 (right):



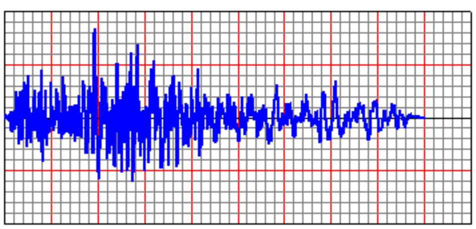


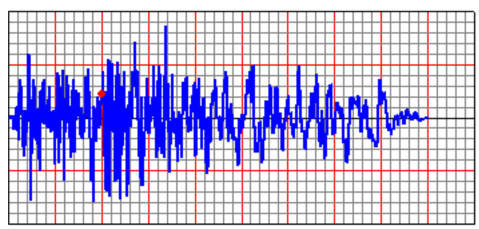
Gilroy, 2002, 4.9M. Station Oakland Airport; U1 (left) U2 (right):



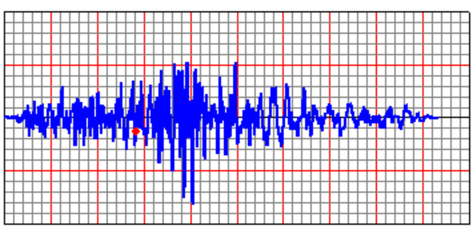


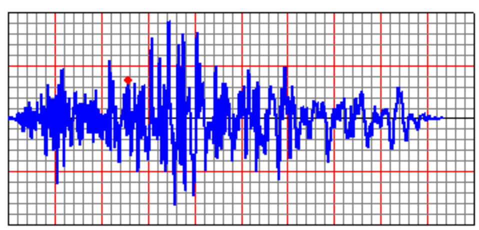
Morgan Hill, 1984, 6.19M. Station Fremont Mission San Jose; U1 (left) U2 (right):



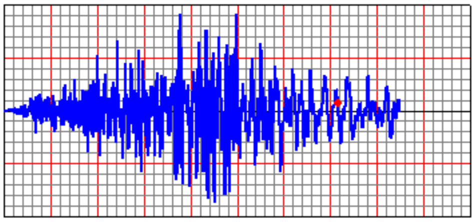


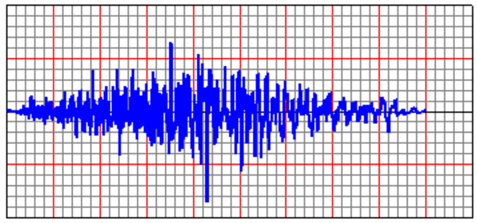
Morgan Hill, 1984, 6.19M. Station 1E Hayward; U1 (left) U2 (right):



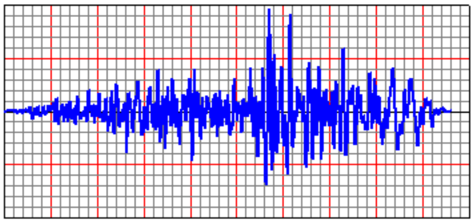


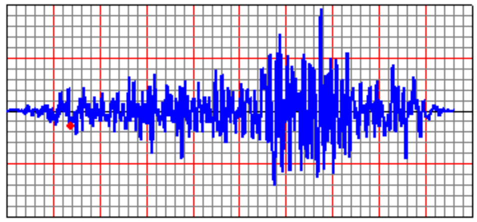
Morgan Hill, 1984, 6.19M. Station Redwood City; U1 (left) U2 (right):



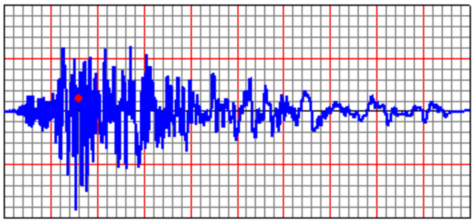


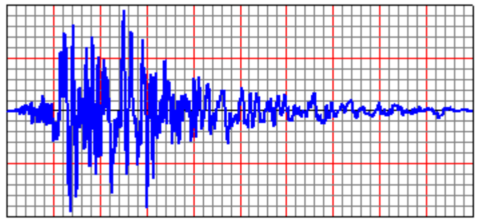
Morgan Hill, 1984, 6.19M. Station SF Airport; U1 (left) U2 (right):



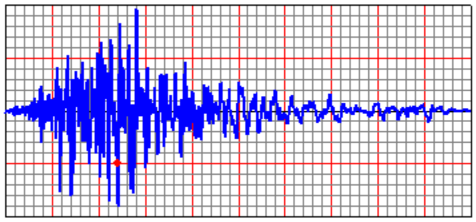


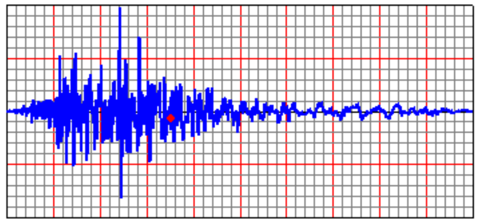
Northridge, 1994, 6.7M. Station Northridge Saticoy; U1 (left) U2 (right):



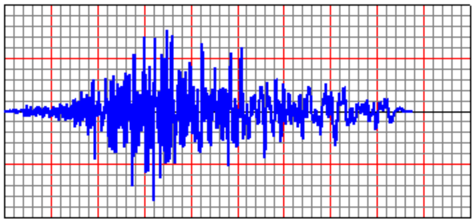


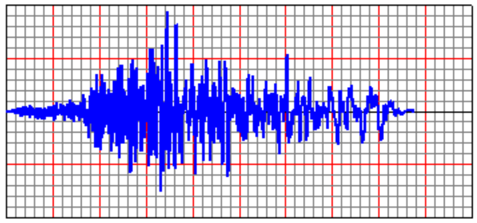
Northridge, 1994, 6.7M. Station Moorpark; U1 (left) U2 (right):



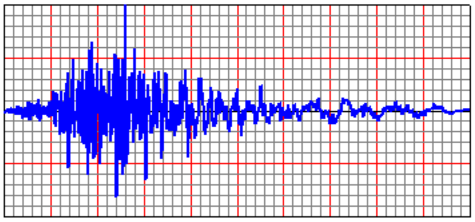


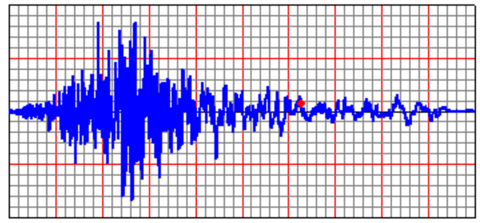
Northridge, 1994, 6.7M. Station Hacienda Heights; U1 (left) U2 (right):





Northridge, 1994, 6.7M. Station Baldwin Hills; U1 (left) U2 (right):





Note that U1 and U2 represent orthogonal directions for each connection node of the structural model.

A.1.2. EXCEEDANCE PROBABILITIES FOR SUNSET DISTRICT, USGS

Conterminous 48 States
2002 Data
Hazard Curve for PGA
Zip Code - 94116
Zip Code Latitude = 37.743599
Zip Code Longitude = -122.487999
Data are based on a 0.05 deg grid spacing
Frequency of Exceedance values less than
1E-4 should be used with caution.

Ground Motion (g)	Frequency of Exceedance (per year)
0.005	4.5864E-01
0.007	4.0942E-01
0.010	3.4995E-01
0.014	2.8388E-01
0.019	2.1685E-01
0.027	1.5666E-01
0.038	1.0825E-01
0.053	7.2093E-02
0.074	4.6931E-02
0.103	2.9982E-02
0.145	1.8779E-02
0.203	1.2335E-02
0.284	8.6457E-03
0.397	5.9373E-03
0.556	3.4528E-03
0.778	1.5284E-03
1.090	4.7681E-04
1.520	1.0076E-04
2.130	1.2133E-05

A.1.3. MMI INTENSITY TO PGA COMPARISON

Instrumental Intensity	Magnitude	Acceleration (g)	ft/s^2	Perceived Shaking
I	Under 2.0	<0.0017	~0.0547	Not Felt
II-III	2.0-2.9	0.0017-0.014	~0.2524	Weak
IV	3.0-3.9	0.014-0.039	~0.8520	Light
V	4.0-4.9	0.039-0.092	~2.1060	Moderate
VI	5.0-5.9	0.092-0.18	~4.3727	Strong
VII	6.0-6.9	.18-0.34	~8.3596	Very Strong
VIII	7.0-9.9	0.34-0.65	~31.5124	Severe
IX	7.0-9.9	0.65-1.24	~30.3839	Violent
X+	10.0 or higher	>1.24	~39.8687	Extreme

A.1.4. LIQUEFACTION SUSCEPTIBILITY CLASSIFICATIONS, APPLIED TECHNOLOGY COUNCIL

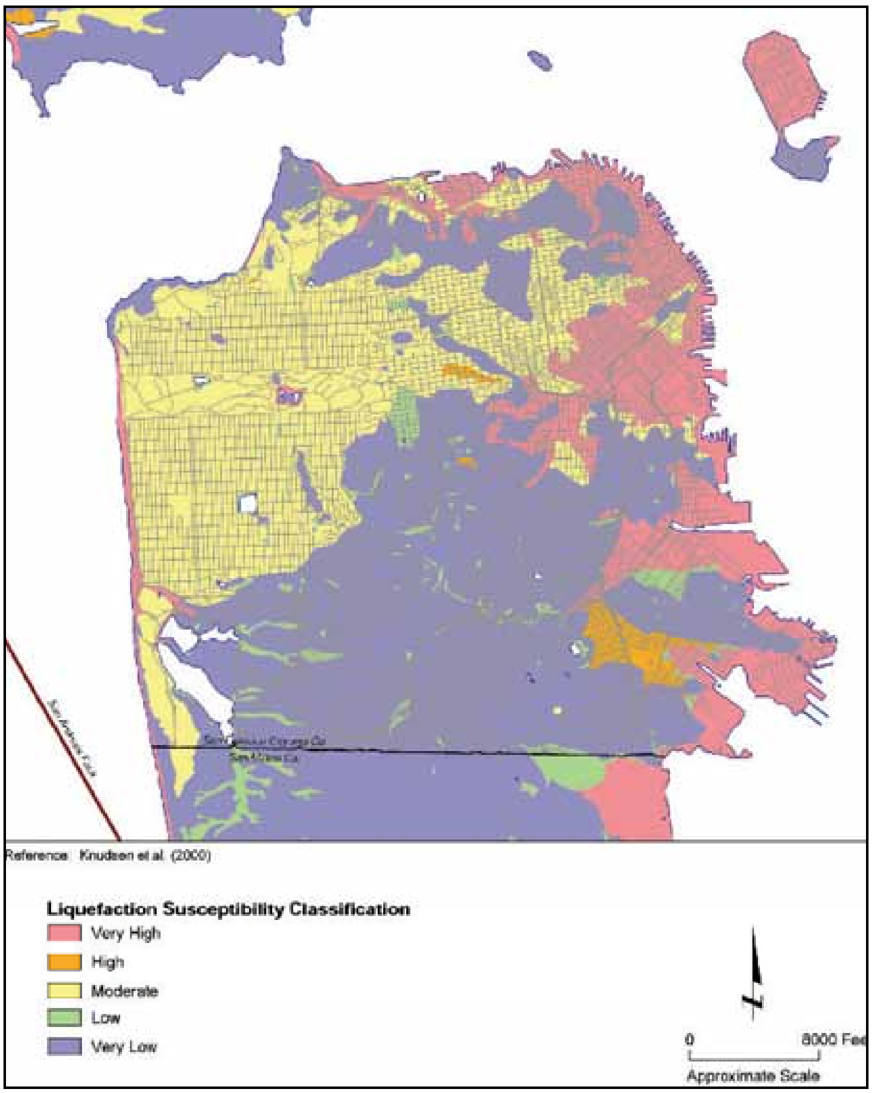


Figure 3-2 Liquefaction susceptibility classifications.

A.1.5. NEHRP SOIL AMPLIFICATION FACTORS, APPLIED TECHNOLOGY COUNCIL, 53-2A

Table 3-2 NEHRP Soil Amplification Factors

Site Class B Spectral Acceleration	Site Class E Amplification Factor
Short Period, S_{as} (g)	
≤0.25	2.5
0.50	1.7
0.75	1.2
1.0	0.9
≥1.25	0.8*
1-Second Period, S_{a1} (g)	
≤0.1	3.5
0.2	3.2
0.3	2.8
0.4	2.4
≥0.5	2.0*

*The NEHRP Provisions do not provide site class E amplification factors when S_{as} >1 or S_{a1} >0.4. Values for these conditions were obtained from the HAZUS®99-SR1 Technical Manual.

A.2.0 DAMAGE

A.2.1. DEFINITION OF MODEL BUILDING W1, HAZUS® TECHNICAL MANUAL, 15-17

Wood, Light Frame (W1):

Slight Structural Damage: Small plaster or gypsum-board cracks at corners of door and window openings and wall-ceiling intersections; small cracks in masonry chimneys and masonry veneer.

Moderate Structural Damage: Large plaster or gypsum-board cracks at corners of door and window openings; small diagonal cracks across shear wall panels exhibited by small cracks in stucco and gypsum wall panels; large cracks in brick chimneys; toppling of tall masonry chimneys.

Extensive Structural Damage: Large diagonal cracks across shear wall panels or large cracks at plywood joints; permanent lateral movement of floors and roof; toppling of most brick chimneys; cracks in foundations; splitting of wood sill plates and/or slippage of structure over foundations; partial collapse of "room-over-garage" or other "soft-story" configurations; small foundations cracks.

Complete Structural Damage: Structure may have large permanent lateral displacement, may collapse, or be in imminent danger of collapse due to cripple wall failure or the failure of the lateral load resisting system; some structures may slip and fall off the foundations; large foundation cracks. Approximately 3% of the total area of W1 buildings with Complete damage is expected to be collapsed.

A.2.2. DAMAGE RESPONSE LIMITS, HAZUS® TECHNICAL MANUAL

Table 5.8 Typical Drift Ratios Used to Define Median Values of Structural Damage

Seismic Design Level	Building Type (Low-Rise)	Drift Ratio at the Threshold of Structural Damage			
		Slight	Moderate	Extensive	Complete
High-Code	W1/W2	0.004	0.012	0.040	0.100
	C1L, S2L	0.005	0.010	0.030	0.080
	RM1L/RM2L, PC1/PC2L	0.004	0.008	0.024	0.070

Table 5.10 Drift Ratios Used to Define Median Values of Damage for Nonstructural Drift-Sensitive Components

Drift Ratio at the Threshold of Nonstructural Damage			
Slight	Moderate	Extensive	Complete
0.004	0.008	0.025	0.050

Table 5.13c Nonstructural Acceleration-Sensitive Fragility Curve Parameters - Low-Code Seismic Design Level

Building Type	Median Spectral Acceleration (g) and Logstandard Deviation (Beta)							
	Slight		Moderate		Extensive		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta
W1	0.20	0.71	0.40	0.68	0.80	0.66	1.60	0.66
W2	0.20	0.67	0.40	0.67	0.80	0.70	1.60	0.70

A.2.3. RES3 OCCUPANCY CLASS DEFINITION, HAZUS® TECHNICAL MANUAL

Developing the multi-family (RES3A through RES3F) and manufactured housing (RES2) inventory requires additional information and effort compared to the single family occupancy classification. In the 1999 census extract, the STF1B (census block data) extract identifies only those housing units within the 10 or more unit classification, unfortunately, the 2000 census extract no longer provided that Information. Therefore in order to define of the multi-family units, it is necessary to utilize the STF3A extract. The multi-family definition in the STF3A extract identifies Duplex, 3-4 Unit, 5-9 unit, 10-19 unit, 20-49 unit, and 50+ dwellings. Additionally the STF3A census data provides a definition of the Manufactured Housing (MH) units within a block group and therefore the RES2 was processed at the same time. The census data has an "other" classification for that will be ignored since this classification represent a very small portion of the universe of housing units and there is no "other" damage functions that can be assigned to these facilities. Examples of the "Other" Census classification include vans and houseboats. Unlike the single family residential that used the Housing Characteristics 1993 to define heated floor area, assessor data from around the United States, including that from the six Proof-of-Concept (POC) communities, was reviewed to develop preliminary estimates of average floor area for multi-family housing. This data was then peer reviewed by engineering experts to develop an average floor area per number of units for the unit ranges provided by the census data.

A.2.4. DAMAGE VALUES, HAZUS® TECHNICAL MANUAL, 3-14

The following repair and replacement costs were used for the RES3 occupancy class when creating fragility functions per point per floor per earthquake applied to the case study building.

Table 15.2: Structural Repair Cost Ratios (in % of building replacement cost)

No.	Label	Occupancy Class	Structural Damage State			
			Slight	Moderate	Extensive	Complete
		Residential				
1	RES1	Single Family Dwelling	0.5	2.3	11.7	23.4
2	RES2	Mobile Home	0.4	2.4	7.3	24.4
3-8	RES3a-f	Multi Family Dwelling	0.3	1.4	6.9	13.8

Table 15.4: Drift Sensitive Non-structural Repair Costs (in % of building replacement cost)

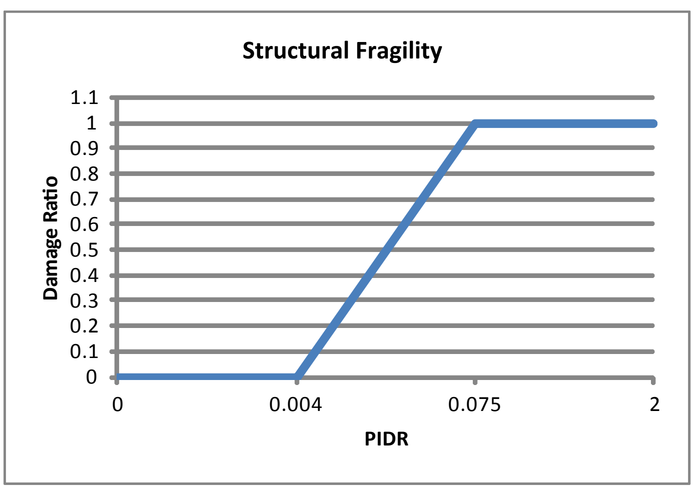
No.	Label	Occupancy Class	Drift Sensitive Non-structural Damage State			
			Slight	Moderate	Extensive	Complete
		Residential				
1	RES1	Single Family Dwelling	1.0	5.0	25.0	50.0
2	RES2	Mobile Home	0.8	3.8	18.9	37.8
3-8	RES3a-f	Multi Family Dwelling	0.9	4.3	21.3	42.5

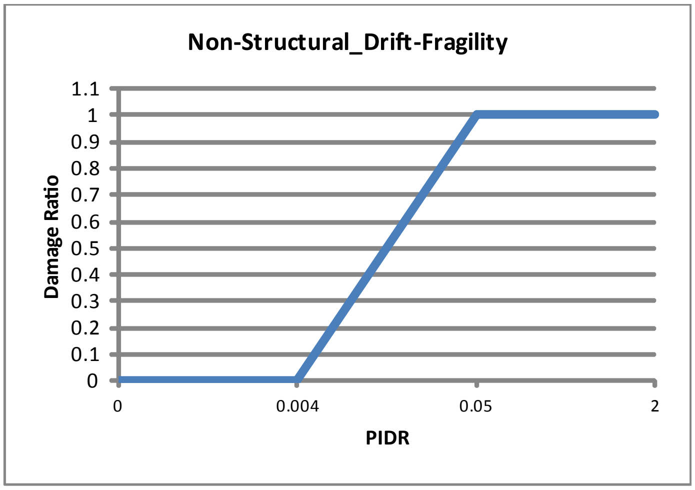
Table 15.3: Acceleration Sensitive Non-structural Repair Cost Ratios (in % of building replacement cost)

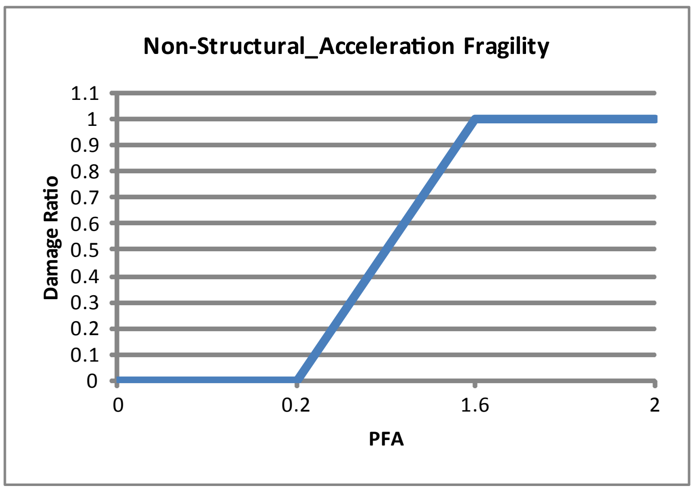
No.	Label	Occupancy Class	Acceleration Sensitive Non-structural Damage State			
			Slight	Moderate	Extensive	Complete
		Residential				
1	RES1	Single Family Dwelling	0.5	2.7	8.0	26.6
2	RES2	Mobile Home	0.8	3.8	11.3	37.8
3-8	RES3a-f	Multi Family Dwelling	0.8	4.3	13.1	43.7

A.2.5. EXAMPLE FRAGILITY FUNCTIONS

The following are examples of fragility functions for each of the three systems (structural, non-structural drift, and non-structural acceleration) for a case of the Northridge earthquake and for one point on floor 1 of the base case building.

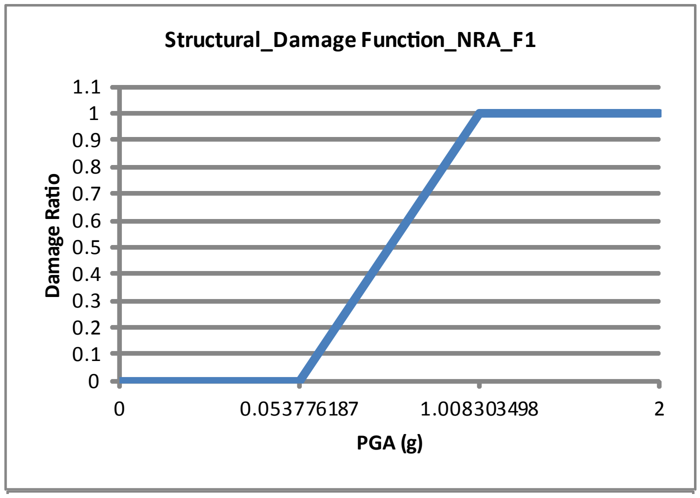


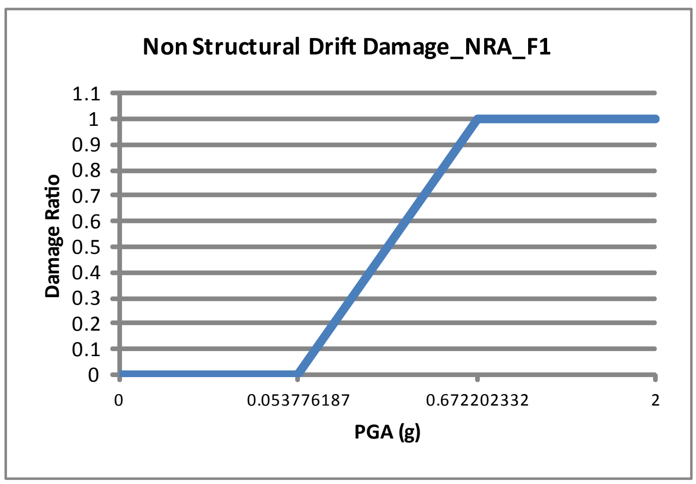

Non-Structural_Drift-Fragility
Damage Ratio
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
0.004
0.05
2
PIDR

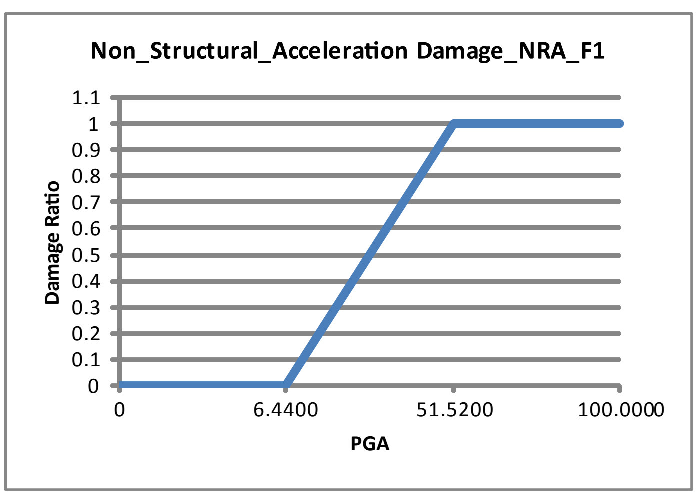

Non-Structural_Acceleration Fragility
Damage Ratio
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
0.2
1.6
2
PFA

A.2.6. EXAMPLE DAMAGE FUNCTIONS

The damage functions shown below correspond to the fragilities in A.2.5. For these and all cases, PGA has been converted from g units to feet per square second.



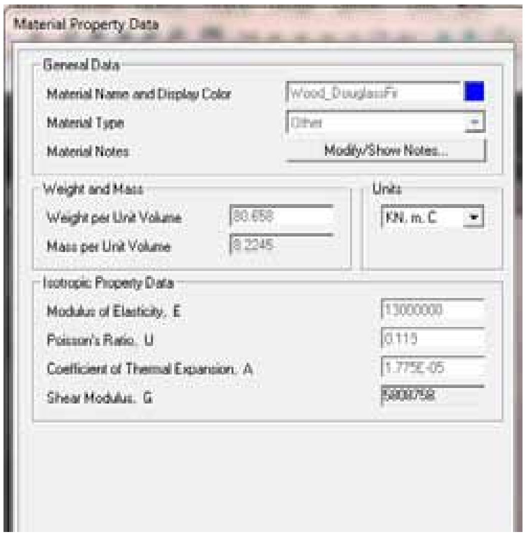


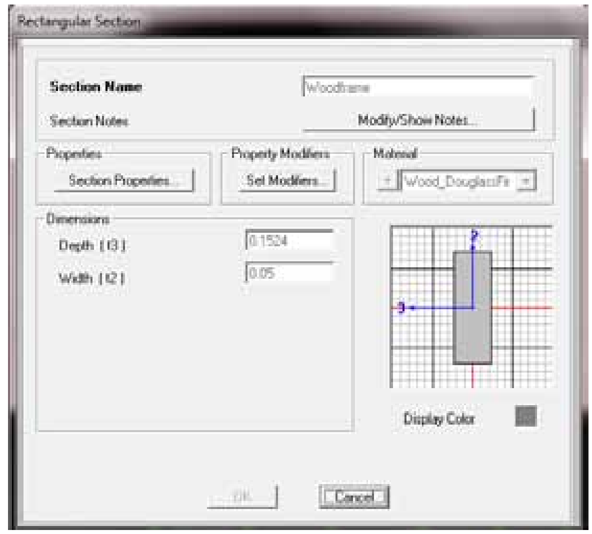


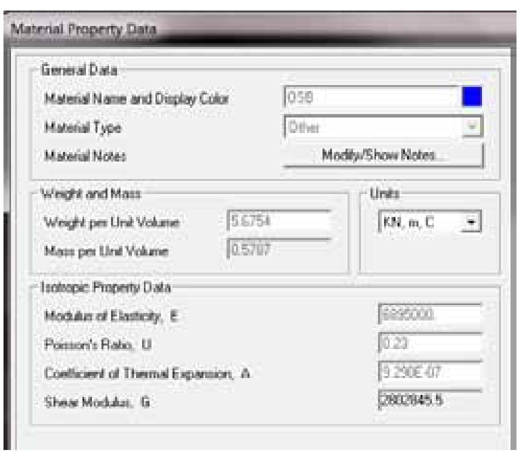
A.3.0 MODEL & SIMULATION

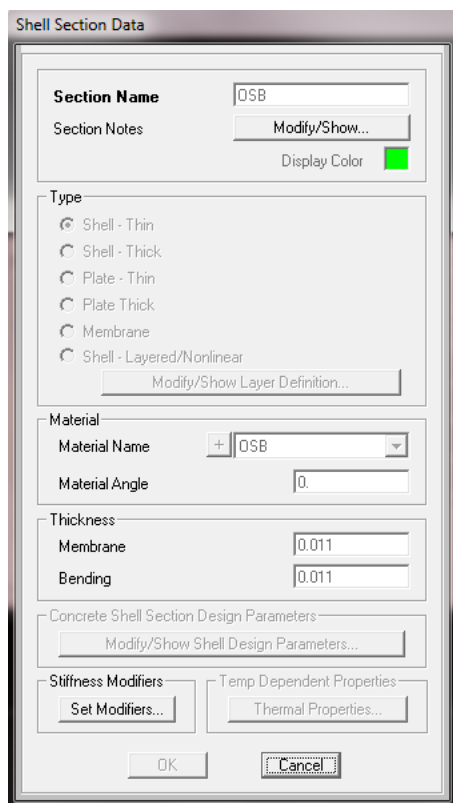
A.3.1. MATERIAL & SECTION PROPERTIES, SAP2000

The important material and section properties for the wood-frame building and OSB panel retrofits modelled for this study are depicted below.



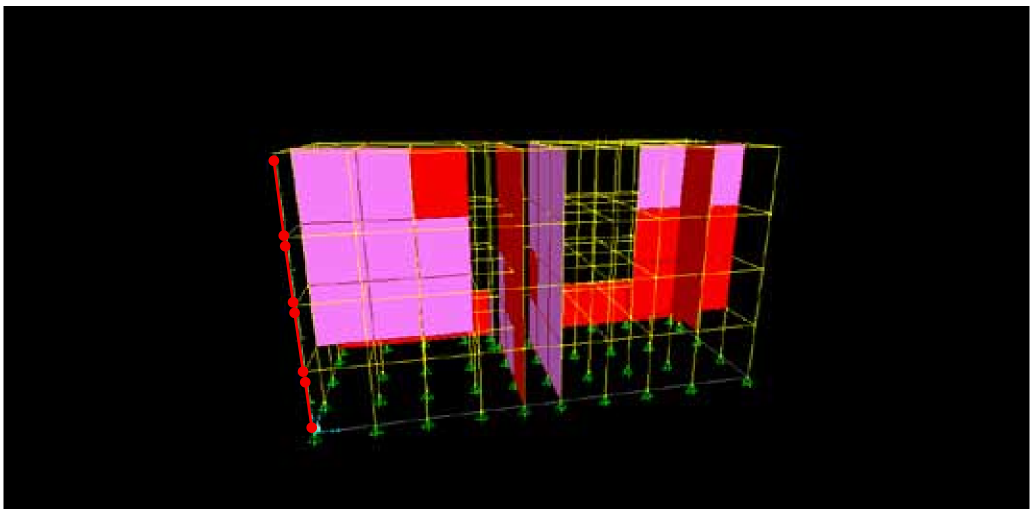


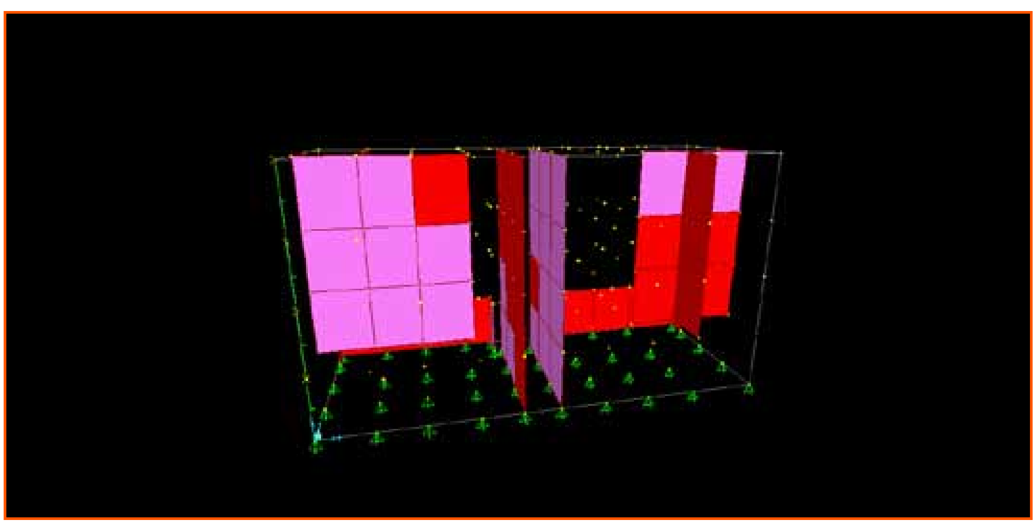




A.3.2. PIDR AND PFA CALCULATIONS

In order to calculate the floor accelerations and peak interstory drift ratio, particular strategies were used in SAP2000 to be able to extract the needed information. In calculating peak interstory drift ratio, four links are drawn to represent columns on the front corner of the building, highlighted in red below. From these links, relative displacements can be found in tables from SAP2000, which are then squared and summed to find maximum drifts over the time histories for use in the damage functions. For peak floor acceleration, a corner point per floor shown in the second image represent the movement of the entire floor in this case, and are used to find the accelerations over the time histories. The average of the U1 and U2 direction accelerations are taken per point per floor and per earthquake to then create the damage functions necessary to conduct the seismic assessment.





A.4.0 RETROFIT

A.4.1. RETROFIT COST ESTIMATES, APPLIED TECHNOLOGY COUNCIL 53-2A

The retrofits for this study were interpolated from the breakdown found in the ATC report 53-2A. For some of the retrofits, the information was parsed from the below graph and evaluated for the specific retrofit case. All values are in 2008 dollars and an inflation rate of 1.06 has been applied for 2013 USD. In addition, Steel Moment Frame and Shearwall 1 retrofit costs are tabulated.

Table 5-1 Building 1: Summary of Cost Estimates for All Schemes

Division	Description	Scheme 1: Moment Frames & Limited Shear Walls		Scheme 2: Moment Frames & Greater Shear Walls		Scheme 3: Cantilevered Columns & Greater Shear Walls	
		Subtotal	Total	Subtotal	Total	Subtotal	Total
1	**General Conditions**		**$9,905**		**$12,349**		**$11,549**
	Personnel	4,000		5,000		5,000	
	Small Tools	500		500		500	
	Temporary Utilities	125		125		125	
	Clean - Up	1,400		1,525		1,525	
	Debris Removal	1,150		1,725		1,725	
	Testing & Inspections	2,730		3,474		2,674	
2	**Sitework**		**$9,070**		**$12,670**		**$12,670**
	Demolition	5,750		9,350		9,350	
	Structural Excavation	3,320		3,320		3,320	
3	**Concrete**		**$11,600**		**$11,600**		**$11,600**
	Reinforcing Steel	4,420		4,420		4,420	
	Concrete Footing	6,920		6,920		6,920	
	Finish Slab	260		260		260	
4	**Masonry**						
	None						
5	**Metals**		**$11,700**		**$11,700**		**$8,220**
	Structural Steel	11,700		11,700		8,220	
6	**Carpentry**		**$8,442**		**$25,602**		**$25,952**
	Rough Carpentry	7,762		24,922		25,272	
	Finish Carpentry	680		680		680	
7	**Moisture Protection**						
	None						
8	**Doors, Windows, & Glass**						
	None						
9	**Finishes**		**$3,188**		**$7,374**		**$7,374**
	Lath & Plaster	1,500		2,250		2,250	
	Drywall	918		4,104		4,104	
	Painting	770		1,020		1,020	
10	**Specialties**						
	None						
11	**Equipment**						
	None						
12	**Furnishings**						
	None						
13	**Special Construction**						
	None						
14	**Conveying Systems**						
	None						
15	**Mechanical**						
	None						
16	**Electrical**						
	None						
	Subtotal	53,905	**$53,905**	81,295	**$81,295**	77,365	**$77,365**
	Building Permit @ 2.4%		**$1,294**		**$1,951**		**$1,857**
	Contingency @ 15%		**$8,086**		**$12,194**		**$11,605**
	Subtotal		**$63,284**		**$95,440**		**$90,826**
	Overhead & Profit @ 25%		**$15,821**		**$23,860**		**$22,707**
	Total		**$79,106**		**$119,300**		**$113,533**

Items not included in estimates:

- relocating any conflicting utilities
- painting new shear walls: walls are not painted, but instead are covered with 5/8" Type X sheetrock and fire taped
- clearing garages of cars and tenants' possessions before construction
- no code upgrade construction for remainder of building
- costs are current for November, 2008; there is no allowance for inflation

Table 5-2 Building 1: Cost Estimate for Scheme 1

Division	Description	Quantity	Unit	Labor Unit Cost	Labor	Material Unit Cost	Material	Subcontractor Unit Cost	Subcontractor	Total
1	**General Conditions** Subtotal for Division: **$9,905**									
	Personnel:									
	Foreman w/truck & phone - 2 hours/day - 4 weeks	40	hours	$100	$4,000					$4,000
	Small Tools:	1	lump sum					$500	$500	$500
	Temporary Utilities:									
	Temporary Sanitation	1	months					$125	$125	$125
	Clean Up:									
	Daily Clean Up - 1 man, 1 hour/day	20	hours	$25	$500					$500
	Final Clean up	16	hours	$25	$400					$400
	Flagman	20	hours	$25	$500					$500
	Debris removal:									
	Debris box	2	each			$575	$1,150			$1,150
	Special Inspections:									
	Welding	1	lump sum					$1,200	$1,200	$1,200
	Concrete cylinders	1	lump sum					$600	$600	$600
	Epoxied dowels	6	hours					$93	$558	$558
	Shear panel nailing	4	hours					$93	$372	$372
2	**Sitework** Subtotal for Division: **$9,070**									
	Demolition:									
	Cut and remove 4' strip of ceiling plaster, 500 sf.	32	hours	$45	$1,440					$1,440
	Remove interior plaster from shear walls and intersecting walls, 720 sq. ft.	32	hours	$45	$1,440					$1,440
	Sawcut, load and haul 6" concrete floor slab, 168 lf. 5 cy.	1	lump sum						$2,870	$2,870
	Structural Excavation:									
	Excavate new footing at garage entries	26	cubic yard	$70	$1,820					$1,820
	Load dump truck (loader)	6	hours					$150	$900	$900
	Haul away dirt (dump truck)	8	hours					$75	$600	$600
3	**Concrete** Subtotal for Division: **$11,600**									
	Reinforcing steel:									
	6 ea #6's t&b w/#5 stirrups @ 12"oc. 84 lf. 2600#. Delivered cages.	1	lump sum					$3,700	$3,700	$3,700
	Place cages in footings	16	hours	$45	$720					$720
	Concrete footing:									
	Place concrete in 2'-6" x 2'-6" footing plus floor slab. 4x6x35/32. 84 lf.	32	cubic yard	$26	$840	$140	$4,480			$5,320
	Concrete pump	1						$1,600	$1,600	$1,600
	Finish slab on grade:									
	Smooth trowel finish concrete garage slab at top of footing. 3'	4	hours	$65	$260					$260
4	**Masonry** Subtotal for Division: **$0**									

Table 5-2 Building 1: Cost Estimate for Scheme 1 (continued)

Division	Description	Quantity	Unit	Labor		Material		Subcontractor		Total
				Unit Cost	Labor	Unit Cost	Material	Unit Cost	Subcontractor	
5	**Metals** Subtotal for Division: **$11,700**									
	Structural Steel: Steel moment frames, 8' x 20', 10WF45 w/RBS. 2 ea. Erected on anchor bolts in new concrete footings & field welded.	1	lump sum					$11,700	$11,700	$11,700
6	**Carpentry** Subtotal for Division: **$8,442**									
	Rough Carpentry: • Additional studs @ shear wall plywood joints and misc framing @ windows. Material.	156	board ft.			$1	$148			$148
	• Additional studs @ shear wall plywood joints and misc framing @ windows. Labor.	4	hours	$60	$240					$240
	• Drill and epoxy anchor bolts.	35	each	$35	$1,225	$45	$1,575			$2,800
	• Drill and epoxy hold downs.	4	each	$35	$140	$110	$440			$580
	• Additional 4x4 posts and tie downs. 48 bf material @ $1.20/bf.	4	each	$30	$120	$15	$60			$180
	• Clips to framing above shear walls.	68	linear ft.	$4	$240	$5	$340			$580
	• Collector beam and connections to floor above @ door opening head. 2 ea 2x12, 168 bf.	168	board ft.			$2	$319			$319
	• Collector beam and connections to floor above @ door opening head. 2 ea 2x12, 42 lf.	16	hours	$60	$960					$960
	• Shear wall sheathing. 15/32 plywood.	612	sq. ft.	$1	$765	$1	$490			$1,255
	• Set anchor bolts for steel frames	4	hours	$75	$300					$300
	• Set anchor bolts for steel frames	4	each			$100	$400			$400
	Finish Carpentry • Exterior siding in 3 ea 4x4 windows	8	hours	$60	$480		$200			$680
7	**Moisture Protection** Subtotal for Division: **$0**									
8	**Doors, Windows, & Glass** Subtotal for Division: **$0**									

Table 5-2 Building 1: Cost Estimate for Scheme 1 (continued)

Division	Description	Quantity	Unit	Labor		Material		Subcontractor		Total
				Unit Cost	Labor	Unit Cost	Material	Unit Cost	Subcontractor	
9	**Finishes** Subtotal for Division: **$3,188**									
	Plaster:									
	Patch openings in ceilings.	500	sq. ft.					$3	$1,500	$1,500
	Drywall:									
	Cover shear walls, 5/8 type x, fire taped.	612	sq. ft.					$2	$918	$918
	Painting:									
	Plaster soffit, seal and 2 coats.	500	sq. ft.					$1	$500	$500
	Steel beams, 10" WF.	72	linear ft.					$2	$144	$144
	2 ea 2x12 collector beams	84	linear ft.					$2	$126	$126
10	**Specialties** Subtotal for Division: **$0**									
11	**Equipment** Subtotal for Division: **$0**									
12	**Furnishings** Subtotal for Division: **$0**									
13	**Special Construction** Subtotal for Division: **$0**									
14	**Conveying Systems** Subtotal for Division: **$0**									
15	**Mechanical** Subtotal for Division: **$0**									
16	**Electrical** Subtotal for Division: **$0**									
				Subtotal:	$16,390		$9,602		$27,913	$53,905
				Subtotal Check:	$53,905		Permits @ 2.4%			$1,294
							Contingency @ 15%			$8,086
				Division Subtotal Check:	$53,905		Subtotal			$63,284
							Overhead & Profit @ 25%			$15,821
							TOTAL			**$79,106**

Items not included in estimates:

- relocating any conflicting utilities
- painting new shear walls: walls are not painted, but instead are covered with 5/8" Type X sheetrock and fire taped
- clearing garages of cars and tenants' possessions before construction
- no code upgrade construction for remainder of building
- costs are current for November, 2008; there is no allowance for inflation

Shear Moment Frame 1

Concrete		
	Reinforcing Steel	$3700
	Concrete Footing	$5320
Metals		
	Structural Steel	$11700
Carpentry		
	Studs	$148
	Clips to framing above shear walls	$580
	Collector Beam & Connections	$319
Finishes		
		$3188
SUB		$24955
Inflation (1.06%)		**$26452.3**

Shearwall 1

Carpentry		
	Shear wall Sheathing plywood	$1255
	Studs	$148
	Clips to framing above shear walls	$580
	Collector Beam & Connections	$319
Finishes		
		$3188
SUB		$5490
Inflation (1.06%)		**$5819.4**

A.5.0 SENSITIVITY

A.5.1. LIFETIME DAMAGE

Because HAZUS® damage limit values tend to be low, a sensitivity test has been conducted by doubling the lifetime damage estimates as an analog to making the fragility functions more conservative. The difference in the Total Cost between the original and conservative states is on average about 45% higher. More retrofits are now closer in range to the base case total cost, if this is to be used as a the baseline factor, to provide cost efficient options for the building. From this case, it would seem that 30% damping is the optimal solution in that it is able to reduce lifetime damage cost by 70% with a relatively low retrofit cost. Of course, viscous dampers vary in cost with the particular building structure, and the values shown here are estimates with embedded approximation errors.

Retrofit Type	Annual Damage Ratio (ADR, x10-3)	Drift Limit	Drift Limit State	Lifetime Damage Cost	Lifetime Cost Decrease from Base	Retrofit Cost	Total Cost (Damage + Retrofit)
Base Case	6.39	0.0077	Collapse	$46,695.67	-	-	$46,695.67
DMP 0.1	4.44	0.0058	Life Safety	$32,463.19	30.48%	$3,630.05	$36,093.24
DMP 0.15	3.76	0.0046	Operational	$27,465.24	41.18%	$9,495.65	$36,960.89
DMP 0.20	2.65	0.0037	Operational	$19,339.18	58.58%	$10,062.42	$29,401.60
DMP 0.30	1.92	0.0026	Operational	$13,992.03	70.04%	$11,136.00	$25,128.03
SMF_1	6.37	0.0053	Life Safety	$46,587.58	0.23%	$26,452.30	$73,039.88
SMF_2	2.92	0.0032	Operational	$21,357.68	54.26%	$52,904.60	$74,262.28
SW_1	6.23	0.0047	Operational	$45,524.72	2.51%	$5,819.40	$51,344.12
SW_2	5.96	0.0047	Operational	$43,574.22	6.68%	$5,380.40	$48,954.62
CMP_1	5.86	0.0049	Operational/ Life Safety Boundary	$42,851.72	8.23%	$27,782.60	$70,634.32
CMP_2	5.44	0.0045	Operational	$39,732.99	14.91%	$58,285.00	$98,017.99
CMP_3	5.3	0.0044	Operational	$38,747.35	17.02%	$63,665.40	$102,412.75
CMP_4	1.81	0.0013	Fully Operational	$13,205.44	71.72%	$73,727.82	$86,933.26
CMP_5	3.29	0.0008	Fully Operational	$24,012.14	48.58%	$47,973.90	$71,986.04
BI	0.09	0.0003	Fully Operational	$658.34	98.59%	$852,600.00	$853,258.34

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