Implementation of Linear Analysis in the Early Stages of Performance-Based Design for Steel Structures

By

Antonio Ayala

Bachelor of Science in Civil Engineering
University of Notre Dame, 2011

Submitted to the Department of Civil and Environmental Engineering in Partial Fulfillment of the Requirements for the Degree of

MASTER OF ENGINEERING
in Civil and Environmental Engineering

at the

MASSACHUSETTS INSTITUTE OF TECHNOLOGY

June 2012

Author: ______________________________
Department of Civil and Environmental Engineering
May 11, 2012

Certified by: ______________________________
Jerome J. Connor
Professor of Civil and Environmental Engineering
Thesis Supervisor

Accepted by: ______________________________
Heidi M. Nepf
Chair, Departmental Committee for Graduate Students

Implementation of Linear Analysis in the Early Stages of Performance-Based Design for Steel Structures

By

Antonio Ayala

Submitted to the Department of Civil and Environmental Engineering on May 11, 2012 in Partial Fulfillment of the Requirements for the Degree of Master of Engineering in Civil and Environmental Engineering

Abstract

In the aftermath of the destructive 1994 Northridge Earthquake in Southern California, the earthquake engineering industry experienced a shift towards expanding seismic requirements beyond surviving global collapse to include performance criteria. As a part of this effort, the Pacific Earthquake Engineering Research Center has developed a performance-based earthquake engineering (PBEE) procedure that outputs relevant non-technical data to aid major building stakeholders in making important decisions.

While PBEE has made great strides in the last decade, its current standing as a *verification* tool has prevented it from being fully adopted by the seismic design industry. In order for PBEE to be fully integrated into the seismic design process, a method that circumvents the problems associated with the preferred method of nonlinear analysis must be developed.

The following study compares interstory drift results from linear and nonlinear analysis to gain insight into their relationship and determine conditions for which linear analysis is an appropriate substitute, yielding a much faster and computationally cheaper procedure. It is hoped that this study will contribute to the adoption of linear analysis in the early seismic design stages, allowing for an optimal structural system selection procedure that integrates performance metrics from the beginning.

Thesis Supervisor: Jerome J. Connor

Title: Professor of Civil and Environmental Engineering

Acknowledgements

Over the years, I have encountered many people who have had a strong impact on my life. While my abilities are God-given, these people have influenced how I utilize my talents and have shaped me to be the person I am today, without whom I would not have been able to succeed.

To my parents, Eddie and Ronna, for being the best role models a son could ask for. You have shown me nothing but love and support throughout the years, and ingrained in me the value of a good education. I know that I wouldn't have the drive to be where I am without you.

To Chris, Alex, and Carlos, for always providing me an outlet to be goofy. Being your older brother is a blast, which is why visits home never come soon enough.

To Caitlin, for always being there for me, whether it was during monthly visits or through phone and Skype calls. It was always relieving to know that, no matter how stressed I was this year, you were always a phone call away. I can't wait for all of our upcoming Boston adventures.

To Noel, Kristen, and Kimmy, for all of the good times we had growing up. Hard to believe we're adults already.

To my friends in St. Louis, for being part of the reason why I love the Lou so much. Going to each others' houses (except Chesterfield) every weekend never got old. Can't wait 'til the five of us are all in the same place at the same time again.

To my friends at Notre Dame, I could not imagine a better group of people to spend my four years of college with. I really miss being in South Bend, and I'm looking forward to our annual reunions. And a special shout out to the 2011 NDSEED Team, for being a part of my first real structural engineering project. I'll never forget our month in Nicaragua.

To all my "high society" friends in the MEng class of 2012, for being a fun and caring group of people amidst a year full of work. I've learned so much from all of you, and I can't wait for our next reunion.

To all of my teachers, past and present, for providing me a wealth of knowledge and forming my love for structural engineering. To Dr. KC and the rest of the Notre Dame faculty, for teaching me so much about structural engineering and skyscrapers. To Dr. Connor, for always having your door open and for being a great advisor, mentor, and Notre Dame football fan. To Pierre Ghisbain, for offering your time and insightful help well beyond your required TA hours.

Table of Contents

Table of Figures, Tables, and Equations

1. Introduction

1.1 Traditional Seismic Design Methods

In earthquake engineering, technological developments and higher performance requirements are shifting the seismic design landscape from traditional methods to newer ones. Traditionally, engineers have adhered to a strength-based approach, which prescribes that a building be safe for rare ground shaking demands and remain safe for significant aftershocks.[1] The first widely used set of design codes against seismic loads in the United States was the Structural Engineers Associate of California's (SEAOC) *Recommended Lateral Force Requirements and Commentary* in 1960, commonly known as the "Blue Book." By following Blue Book provisions, structures should be able to:[2]

- Resist minor earthquakes without damage;
- Resist moderate earthquakes without structural damage, but with some nonstructural damage; and
- Resist major earthquakes, of the intensity of severity of the strongest experience in California, without collapse, but with some structural as well as nonstructural damage.

The magnitudes of earthquakes noted above are simply referred to as ground motions in design, and their definitions have also evolved as the seismic codes evolved. While a design basis earthquake (DBE), expressed as a 10% probability of exceedance in 50 years was used for nearly three decades, a more stringent parameter called the maximum considered earthquake ground motion (MCE) was later adopted (2%/50 years). The code design philosophy was then to provide a

---

[1] Moehle, "Performance-Based Seismic Design of Tall Buildings in the U.S.," 2.
[2] SEAOC.

uniform margin against collapse for the MCE, implemented by traditional means for motions of 2/3 of the MCE.[3]

While seismic events can be expressed as a probability of exceedance over a period of time (e.g., 2%/50 years), they can inversely be expressed as a return period, which is an estimate of the period of time between a specific magnitude of earthquake. In the strength-based scenario, the capacities of individual structural elements are designed to be greater than the demand associated with the earthquake, with some damage permitted.

In 1994, the 6.7-magnitude Northridge Earthquake in Southern California caused significant structural and nonstructural damage to many buildings in the area. According to reports, hospitals and health care facilities experienced these damages as well, which "impaired their ability to protect occupants during the earthquake and to treat earthquake victims afterwards. Eleven hospitals were completely or partially closed, and their patients evacuated, due to earthquake damage."[4] This devastating event forced structural engineers to reconsider the current strength-based method of structural design and include serviceability after an earthquake into design, so that predictable performance can be provided.

SEAOC's response to these concerns was a document called Vision 2000 that signaled the advent of performance-based earthquake engineering (PBEE). Within this document, performance levels for buildings in the aftermath of an earthquake are defined, as well as ground motion design levels and performance objectives for the structures.[5] This improved method allows for the seismic design of a building according to its importance and desired performance level. For instance, a hospital is in a high importance category, since it must allow for immediate occupancy and minimal component damage following a strong earthquake. Consequently, the

---

[3] Holmes, Kircher and Petak, 11.
[4] Lin, v.
[5] Holmes, Kircher and Petak, 13.

stricter performance requirements of a hospital dictate a stronger design with a higher associated cost than a typical building.

SEAOC's chart of seismic performance objectives, which balance performance levels of buildings with ground motion levels to represent a specific design performance objective, can be seen in Figure 1. In this chart, structures are organized into three categories based on function and importance: ordinary buildings, essential buildings, and hazardous facilities. These categories are represented as lines on a chart that balance varying levels of earthquakes with building performance levels, whose description can be seen in Table 1. Vision 2000 was a giant step in improving the way buildings are designed against seismic loads. While designing against collapse is certainly the priority in any structural design, post-seismic serviceability was a strong and overlooked need that was exposed in the Northridge Earthquake.

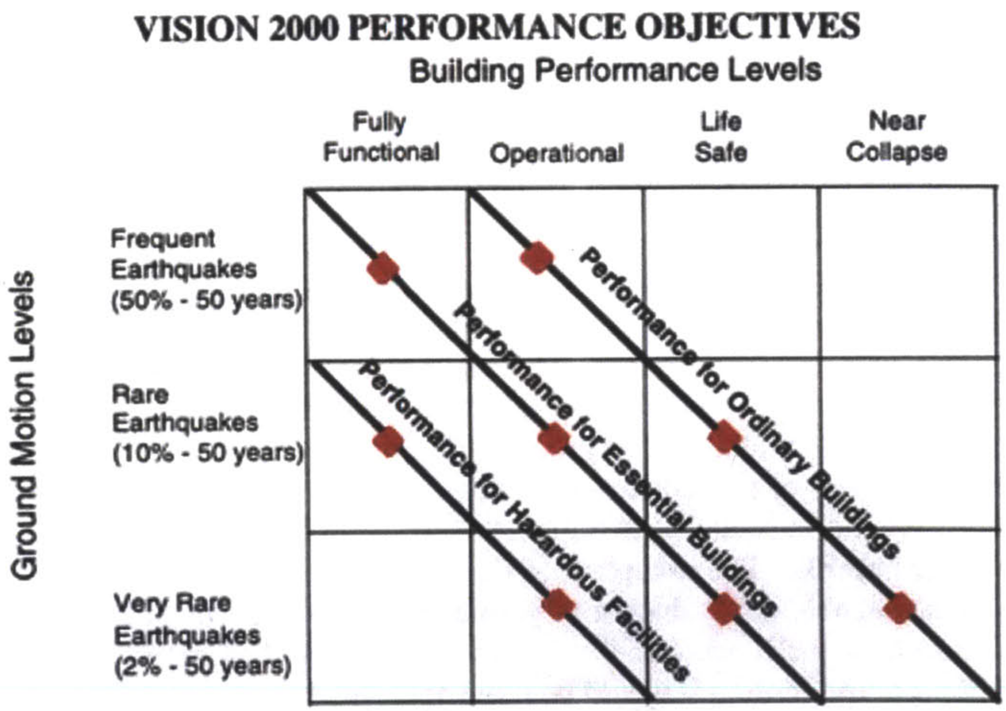


Figure 1: SEAOC's Vision 2000 Chart[6]

[6] Hamburger.

Table 1: Vision 2000 Performance Objectives[7]

Performance Level	Description
Fully Functional	No significant damage has occurred to structural and nonstructural components. Building is suitable for normal intended occupancy and use.
Operational	No significant damage has occurred to structure, which retains nearly all of its pre-earthquake strength and stiffness. Nonstructural components are secure and most would function, if utilities available. Building may be used for intended purpose, albeit in an impaired mode.
Life Safe	Significant damage to structural elements, with substantial reduction in stiffness, however, margin remains against collapse. Nonstructural elements are secured but may not function. Occupancy may be prevented until repairs can be instituted
Near Collapse	Substantial structural and nonstructural damage. Structural strength and stiffness substantially degraded. Little margin against collapse. Some falling debris hazards may have occurred.

While Vision 2000's conception in the mid-1990s was a groundbreaking advancement in seismic design, a shortcoming in its definitions of performance levels prevented it from being a completely useful and adopted tool. In Table 1, it can be seen that performance levels are defined in non-quantitive descriptions, which are not particularly useful in design. Consequently, performance targets are vague and it is hard to say whether a design meets these vague criteria or not. Additionally, building owners, who sometimes provide specific performance criteria and assume the costs of both the initial structural and repairs after an earthquake, are not as involved in the decisions within the design process as they should be.

---

[7] Hamburger.

1.2 Next-Generation Seismic Design

While Vision 2000 provided a significant step in earthquake engineering by setting design requirements beyond global structural collapse, the lack of quantitative and descriptive measures of damage remained a hurdle to making PBEE a widely adopted and useful process. A better design process would include damage estimates under seismic events that major stakeholders of the building can evaluate before making decisions or approvals. This loss is not limited to the costs of repairs and replacements to damaged components, but can also extend to downtime during these repairs and other nonmonetary losses.

Recently, earthquake engineering experts have been developing an improvement to the PBEE process that focuses on setting performance criteria and quantifying the aforementioned losses in order that all important decision makers be involved in the design process. These developments and their implementation in seismic design have become an increasingly popular research field over the past decade.

Professionals in the earthquake engineering field are improving the PBEE process by developing design and performance assessment methods that express building design options in terms that the major stakeholders, including building owners and insurance companies, will find most applicable to them. Performance criteria such as damage to building components and downtime of building activities after an earthquake can be specified during design, allowing for a more informed design process that includes all the important decision makers and their needs. As a result, buildings will not only be designed against collapse, but also adhere to stricter performance criteria.

1.3 Pacific Earthquake Engineering Research Center

The primary promoting entity for PBEE, the Pacific Earthquake Engineering Research Center (PEER), was established as a consortium of nine West Coast Universities in 1996 and has since grown to include investigators from over twenty universities, several consulting companies, and researchers at various government agencies that work on developing PBEE technology. PEER's mission is to "develop, validate, and disseminate performance-based seismic design technologies for buildings and infrastructure to meet the diverse economic and safety needs of owners and society. [Their] research defines appropriate performance targets, and develops engineering tools and criteria that can be used by practicing professionals to achieve those targets, such as safety, cost, and post-earthquake functionality."[8]

In developing the PBEE approach, PEER is pursuing a method that is much more scientifically based than the current code-based design method and whose results are more meaningful to those who must make decisions for structures and networks of structures. They recognize the need to formulate a consistent and reliable approach to balance the relationship between engineering demands and member performance. As research and design methods develop further, it is PEER's hope that PBEE will be formalized into a widely accepted method in practice, replacing first-generation methods of seismic design. An overview for the PBEE methodology is outlined in the next chapter.

---

[8] http://peer.berkeley.edu/about/what_is_peer.html

2. Performance-Based Earthquake Engineering

2.1 Framework for Performance-Based Earthquake Engineering Methodology

In evaluating the previous earthquake engineering approaches, PEER noted that finding a better method to determine the relationship between a seismic event and damage incurred by the structure was a critical and necessary improvement. Furthermore, they hoped to quantify this "damage" in more descriptive and useful terms, each with significance to different parties. They hoped, for instance, that an engineer could determine the probable downtime that the facility would incur given a seismic event of a certain intensity. These outputs, rather than traditional technical parameters such as floor accelerations and interstory drift ratios, are of much more significance and interest to facility stakeholders. With this knowledge, a balance between initial monetary investment of the design and projected loss over the life of the building, with other performance metrics in mind, can be made by the appropriate people.

As mentioned in the preceding chapter, seismic design is done in a probabilistic manner. That is, the inherent uncertainty and variability in seismic response prevents an engineer from designing a structure that will undoubtedly withstand an earthquake. It is impossible to know with absolutely certainty whether an earthquake of a specific intensity will ever hit a certain geographical region. Compounding this uncertainty is the fact that the behaviors of earthquakes of the same intensity are not identical, resulting in differing building responses between these events. Therefore, all variables associated with PEER's framework for PBEE, which was introduced in its final form in 2005, are expressed as probabilities of exceedance.

In PEER's framework for PBEE, earthquake intensity measures, such as peak ground accelerations, are more directly translated into meaningful terms of damage. From these non-technical descriptions of damage, appropriate parties are able to make informed design decisions. A diagram for this framework can be seen in Figure 2.[9]

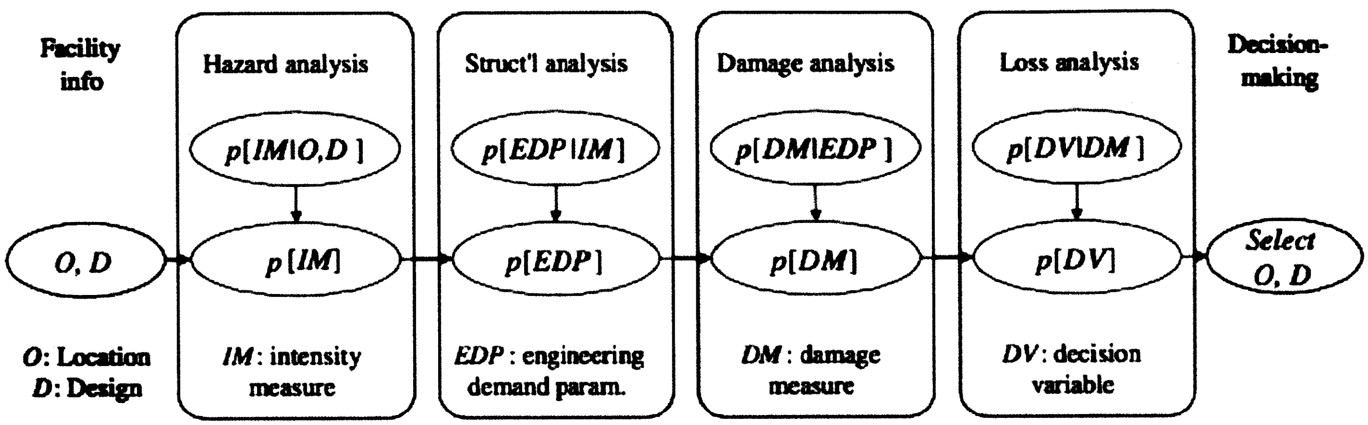


Figure 2: PEER Framework for Performance-Based Earthquake Engineering

2.2 Current Performance-Based Methodology

Figure 2 shows that PEER's methodology for seismic performance evaluation yields pertinent decision-making information after taking baseline facility information through four separate analyses: a hazard analysis, structural analysis, damage analysis, and loss analysis.

The first step of the framework, hazard analysis, involves evaluating a selected *Intensity Measure (IM)*, which is a characteristic of a specific earthquake. While PEER has been working in close coordination with leaders in the earth science, geotechnical engineering, and engineering seismology fields to develop alternative *IM* variables, conventional variables currently being used in PBEE include peak ground acceleration (PGA) and spectral acceleration at the period for the n^{th} mode

---

[9] Moehle and Deierlein, "A Framework Methodology for Performance-Based Earthquake Engineering."

of the structure ($S_a(T_n)$).[10] From this step, a specific value for the *IM* variable is selected to reflect the intensity of the earthquake and governs the resulting building behavior and damage. Within this probabilistic procedure, *IM* is typically described as a mean annual probability of exceedance, *p[IM]*.

In the structural analysis stage, a series of nonlinear dynamic analyses are executed to determine *engineering demand parameters (EDP)* resulting from the selected *IMs*.[11] The nonlinear dynamic analyses output *EDP* values that express the structural response of a building given an earthquake event; these results are given in terms of deformations, accelerations, induced forces, or other quantities. A response function can show the variation of building response to earthquakes of the same intensity. Determining an appropriate *EDP* variable depends on the desired final damage measurement, though the most commonly used variables are maximum interstory drift ratios and peak floor accelerations. An *EDP* value can be quantified by finding the probabilistic relationship between *IM* and *EDP*, *p(EDP|IM)*, which can then be integrated with the *IM* probability of exceedance to calculate the mean annual probabilities of exceeding the desired *EDP*.

The next stage in the PBEE process is to relate *EDPs* to quantifiable *damage measures (DM)* in a damage analysis stage. In this step, components in the structure are organized into "performance groups" according to similar characteristics. Performance groups can be arranged by location (within a story or by story), structural significance (drift-sensitive structural or acceleration-sensitive nonstructural elements), more discrete groups including a combination of the two, or other divisions. In order to get a meaningful description of damage, performance groups are arranged so that components within a group possess the same general damage conditions as others within the group; this allows for damage to be quantified and checked for both the global structural system and individual

---

[10] Moehle and Deierlein, "A Framework Methodology for Performance-Based Earthquake Engineering."
[11] Moehle, "An Application of PEER PBEE Methodology."

components of the building. For instance, nonstructural performance groups (partitions or mechanical equipment) can be arranged by floor, with peak floor accelerations governing their damage. With these performance groups, engineers will more easily quantify the repair, safety, and monetary implications for various *EDP* values. This quantification is done by calculating the mean annual probability of exceedance for the *DM*, *p(DM)*, by integrating the conditional probability *p(DM|EDP)* with *p(EDP)*.

The final step in the PEER's framework is to calculate *decision variables (DV)*, which are the key connections between raw data of a seismic event to the decision makers of the building. *DV*s are descriptive terms of damage that are easily understood by stakeholders in order to make informed design decisions. *DV*s can be quantified in direct dollar losses, repair time, or casualties. As with the other three variables, the mean annual probability of exceedance for *DV*, *p(DV)*, is calculated by integrating the *DM* probability with the conditional probability relating *DV* to *DM*. Equation 1 shows the entire probabilistic result for the PBEE method expressed in a triple integral, where $\lambda(DV)$ is the desired realization of the *DV*, and G is the complementary cumulative distribution function.[12]

$$\lambda(\mathrm{DV}) = \iiint G\langle \mathrm{DV}|\mathrm{DM}\rangle \, \mathrm{d}G\langle \mathrm{DM}|\mathrm{EDP}\rangle \, \mathrm{d}G\langle \mathrm{EDP}|\mathrm{IM}\rangle \, \mathrm{d}\lambda(\mathrm{IM})$$

Equation 1: Formulation for Evaluating Decision Variables in the PBEE Method

There are several ways to communicate decision variables, such as repair costs, to the building owners. One such option is to express *DV*s as a distribution associated with levels of *IM*s, as seen in Figure 3. In this situation, the owner can make an informed decision on the level of earthquake that will cause a certain amount of damage. Balancing the associated damage costs with initial investment, along with

[12] Krawinkler, Zareian and Medina, 116.

other performance requirements, a level of earthquake against which a building will be designed can be selected.

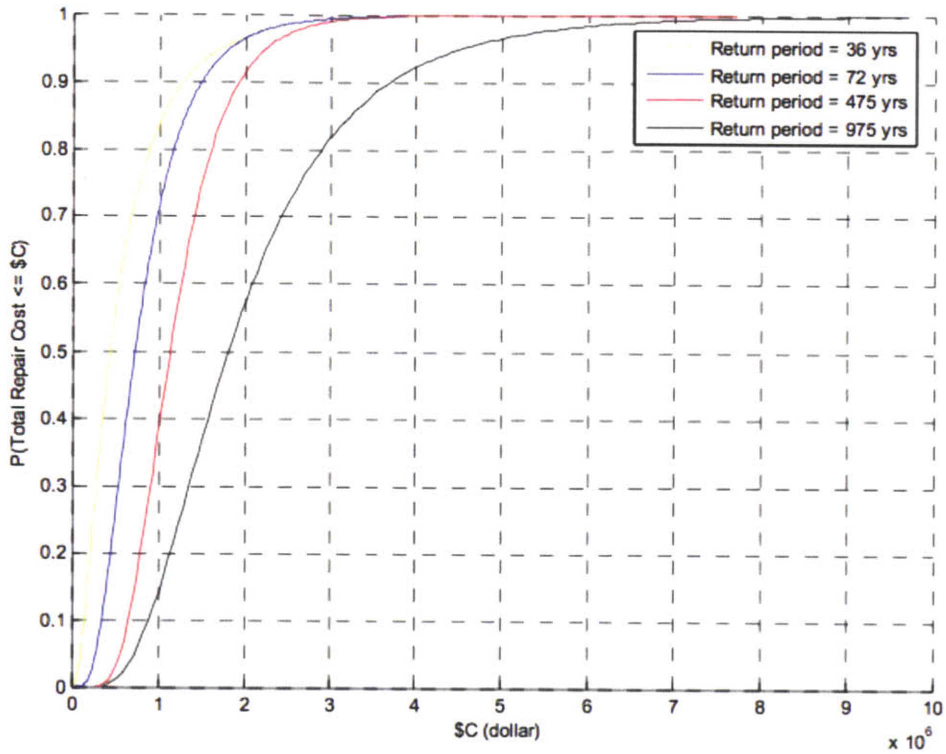


Figure 3: Distribution of Building Repair Cost for Different IM Levels

3. Linear vs. Nonlinear Considerations in Seismic Design

3.1 Description of Nonlinear Structural Analysis

Due to the significantly larger computational cost and detail associated with nonlinear structural analysis, many buildings are designed for seismic resistance using elastic analysis. However, in highly seismic areas, it is not economically feasible to design a building to remain fully elastic under extreme ground motions.[13] For this reason, enabled by advancing computational capabilities, the earthquake engineering field is increasingly using nonlinear analysis procedures, which allow for structural response calculations into the plastic range.

As opposed to the limitations of linear analysis, nonlinear analysis "provides the means for calculating structural response beyond the elastic range, including strength and stiffness deterioration associated with inelastic materials behavior and large displacements."[14] This method takes into account a structural member's abrupt transition from elastic to plastic behavior under extreme loading, requiring the structure to redistribute load capacity among other members.

In addition to the loss of strength due to the onset of plastic behavior in structural members, geometric nonlinear effects also contribute to lateral structural instability. These effects are caused by gravity loading on the deformed configuration of the structure, leading to an increase of internal forces in members and connections.[15] When classifying the effects, those associated with deformations within a member (P-∂ effect) are differentiated from those on a global scale (P-Δ effect) that are exacerbated by interstory drifts. In the analysis of a structure under a large event, the latter is more of a concern due to the magnification of internal

---

[13] Moehle, "Performance-Based Seismic Design of Tall Buildings in the U.S.," 2.
[14] Deierlein, Reinhorn and Willford, 1.
[15] Deierlein, Reinhorn and Willford, 9.

forces and moments throughout the building due to the nonlinearity; consequently, the structure loses lateral stiffness and thus lateral strength.

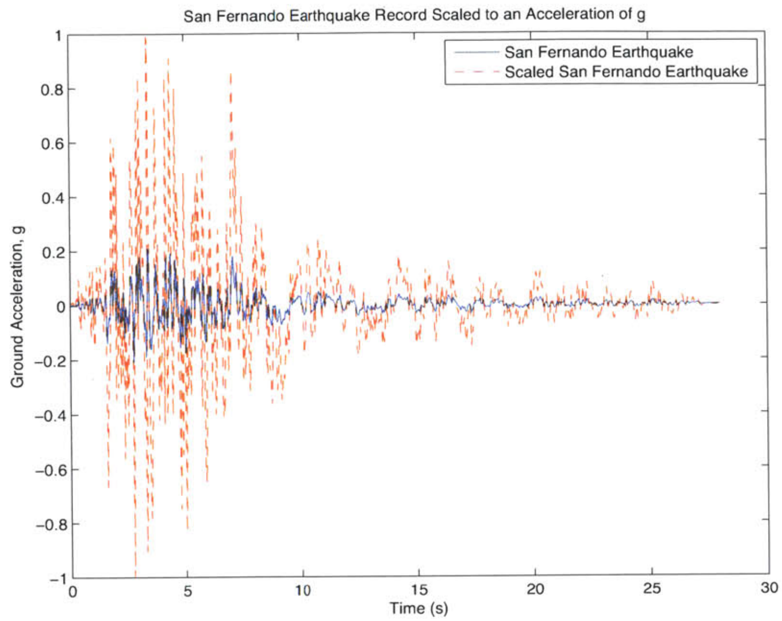
3.2 Implementation of Nonlinear Analysis

While many undefined aspects of nonlinear analysis procedures have yet to be determined, guidelines (FEMA 356, 2000) and code requirements (ASCE 7, 2002; IBC, 2003; UBC, 1997) for these procedures exist.[16] With code-based procedures, seismic ground shaking levels for PBEE are typically based on approved contour maps or response spectrum shapes provided. However, a more thorough and accurate method would be to undergo the site-specific seismic hazard analysis as described in Chapter 2. In this analysis, representative ground motion records for site-specific conditions are selected for the building.

In current practice, it is typical to select and manipulate earthquake records from databases and apply them to the structure after manipulating them to represent target intensity measures. This manipulation can be done by either scaling or spectrum matching. Scaling involves applying a constant factor to the earthquake record in order to make the peak response match the design spectrum at a target period(s). An advantage of this method is that the earthquake records maintain their time-dependent characteristics, such as the peaks and valleys in the record, as scaling only affects the magnitude; an example of scaling for two earthquake records to an acceleration of 1 g can be seen in Figure 4. In spectrum matching, "individual ground motion records are manipulated (usually in the time domain by addition of wave packets) to adjust the linear response spectrum of the motion so it matches the target design response spectrum."[17]

---

[16] Moehle, "Nonlinear Analysis for Performance-Based Earthquake Engineering," 385.
[17] Moehle, "Performance-Based Seismic Design of Tall Buildings in the U.S.," 4.



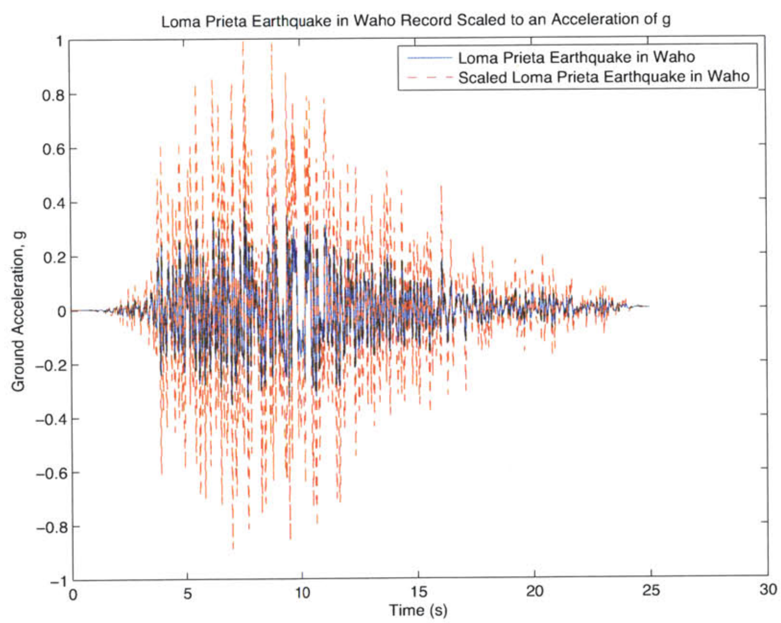


Figure 4: Scaling of the San Fernando and Loma Prieta Earthquakes to a Peak Intensity of g

After the appropriate selection and manipulation of ground motion records, building response under these loads are studied. Taking into account nonlinear action, analysis can be done to estimate the internal forces associated with the nonlinear response from the earthquakes, including the P-Δ effects and formation of plastic hinges. Additionally, records can be scaled to different values to observe the nonlinear response of the structure under varying earthquake intensities as well.

3.3 Reasons for Using Nonlinear Analysis

Due to its more precise description of a structure's behavior under extensive loading, nonlinear analysis has become increasingly popular in PBEE.[18] Linear analysis procedures, while computationally cheaper, generally provide poor indications both of the level of axial load and the degree of nonlinear action required.[19] Implemented properly, nonlinear dynamic analysis specific to the structural system and seismic environment is the best way to identify nonlinear dynamic response characteristic, including yield mechanisms, associated internal forces, deformation demands, and detailing requirements.[20]

Another important factor favoring the use of nonlinear analysis is the fact that "internal actions cannot be scaled directly from linear results; similarly, nonlinear behavior at one hazard level cannot be scaled from nonlinear results at another hazard level."[21] Therefore, using linear analysis for a seismic event does not create a general building performance envelope that can accurately be scaled for other earthquakes; individual seismic load cases must be run. As noted in a previous chapter, every earthquake record has unique characteristics that may have different effects on a building, such as the formation of plastic hinges in different structural members. In order to fully capture these effects, nonlinear analysis should be performed for each seismic load case.

---

[18] Moehle, "Performance-Based Seismic Design of Tall Buildings in the U.S.," 1.
[19] Moehle, "Nonlinear Analysis for Performance-Based Earthquake Engineering," 385.
[20] Moehle, "Performance-Based Seismic Design of Tall Buildings in the U.S.," 7.
[21] Moehle, "Nonlinear Analysis for Performance-Based Earthquake Engineering," 394.

4. Shortcomings of Performance-Based Earthquake Engineering

While PBEE has rapidly increased in development over the last decade, there still remain many undefined aspects of the process that require further research before full implementation in the structural engineering industry. At this point, PBEE's strongest application in the professional industry is in performance assessment, and it is currently being used for design improvements and retrofits. Unfortunately, PBEE currently lacks design strategies, which prevent it from having a significant role in the building design process. PEER hopes that further research will change this, however, and believes that the methodology can eventually "be used as a means of calibrating simplified procedures that might be used for the advancement of building codes."[22] Until a formalized procedure for PBEE in structural design is developed, its role in the structural engineering industry will be incomplete.

Since PBEE's implementation currently lies in performance assessment, building design is completed separately from this stage. Consequently, the selection of a structural system is governed by the strength requirements of the building; only after making this selection and a subsequent full design is seismic performance evaluated. The selected structure will typically require adjustments in order to meet prescribed performance criteria, though these adjustments are made to the fully designed structure and not the structural system.

Separating design and performance assessment is a suboptimal process, as "a poor initial conceptual design may be tuned to an extent that it fulfills the performance targets, but it likely will never become a good design."[23] Implementing performance criteria into the early design stages will likely yield optimal results, as subsequent performance assessment can serve as verification of the structural system. Rather than serviceability adjustments to smaller components of the structure after the

---

[22] Moehle and Deierlein, "A Framework Methodology for Performance-Based Earthquake Engineering," 10.
[23] Krawinkler, Zareian and Medina, 116.

system is selected, iterations on a larger scale with performance targets in mind can yield an efficient structural system. Unfortunately, this iteration is impractical under current PBEE methods for a few reasons.

As noted in Chapter 3, the occurrence of structural damage is more directly related to deformation than lateral force level for a yielding building.[24] Therefore, under seismic loads, nonlinear analysis is the preferred (and still developing) analytical tool as it more clearly and accurately captures the behavior of a building under large and varying forces. This procedure brings a building beyond elastic behavior and takes into account deterioration of structural elements and P-Δ effects, resulting in redistribution of forces throughout the structure.

While nonlinear analysis is a more accurate method for capturing a building's behavior, its implementation in current PBEE procedures makes selecting a structural system in the early design stages very difficult. Nonlinear analysis is computationally costly due to the many changes and iterations performed at different stages of loading. As a result, significant time is required to perform the nonlinear analysis for an entire building model. Since several iterations are likely required in order to develop an optimal structural system that meets all strength and performance requirements, implementing nonlinear analysis, while preferred, is very inefficient.

Compounding the issue of significant time associated with nonlinear analysis is that this duration of time is dependent on the size of the finite element model. In many cases, building models will have thousands of nodes and members, requiring very long iteration times. Additionally, the large number of model components makes determining which members have a significant effect on performance very difficult. Altering members and load distributions without fully knowing their effects on the system, and then running a computationally costly nonlinear analysis, makes for a

[24] Moehle, "Nonlinear Analysis for Performance-Based Earthquake Engineering," 385.

very frustrating process. Until this issue is resolved, implementing performance metrics in the early design stages is impractical.

5. Study Overview

Though formal PBEE design procedures have yet to be developed, this study hopes to contribute to this effort by addressing some of the problems listed in Chapter 4. As noted, one potential improvement in seismic design optimization is to integrate performance criteria consideration into the early design stages, allowing the selection of a structural system to be based on both strength and performance. However, this goal can only be attained through a solution that circumvents the problems associated with nonlinear analysis.

While nonlinear analysis is the more accurate and preferred method to measure structural performance under large seismic events, this study seeks to determine whether linear analysis is an adequate substitute for the initial stages of design. Should this be the case, structural system iteration would be a very realistic possibility due to the significant time savings associated with switching from complex computations to simpler ones due to the assumption of elastic behavior. In this scenario, nonlinear analysis for a full description of the building's performance can be executed after the initial and optimal structural system is selected.

The following study will be focused on the optimization of the structural systems of steel buildings. In determining whether utilizing linear analysis in the initial design stage is appropriate, two variables will be studied to see their effects in comparing linear and nonlinear analysis: building damping and earthquake intensity. By studying these parameters, it will become more apparent whether each has a significant effect, if any, on the comparison.

6. Analysis Procedure

6.1 The Model

6.1.1 Analysis Overview

In this study, a comparison of linear and nonlinear seismic performance is executed for a two-dimensional steel building. While a three-dimensional model would more accurately measure the behavior of a real building, the author seeks to determine whether a close relationship between linear and nonlinear analysis exists on the most rudimentary level. Should a relationship exist, a more complex and computationally intensive three-dimensional building can be studied.

To determine whether linear analysis is a suitable substitute for nonlinear analysis in the beginning stages of PBEE design, the percent error between linear and nonlinear analysis results will be calculated and evaluated. Additionally, results with varying parameters will also be compared to determine whether these parameters have an effect on the results, which will assist in developing future design recommendations.

6.1.2 Details of the Building Model

In this study, a 90-foot six-story steel frame building with three 20-foot bays and fixed base supports will be considered, modeled, and analyzed in SAP2000. An unfactored superimposed dead load of 20 psf and live load of 100 psf are applied on each beam, assuming a beam tributary depth of 20 feet. Upon application of gravity loads, a typical linear analysis was performed. Then, member sizes for the beams and columns were automatically designed by SAP2000 according to conventional code design procedures to reflect an appropriate building design. The building model with labeled member sizes can be seen in Figure 5.

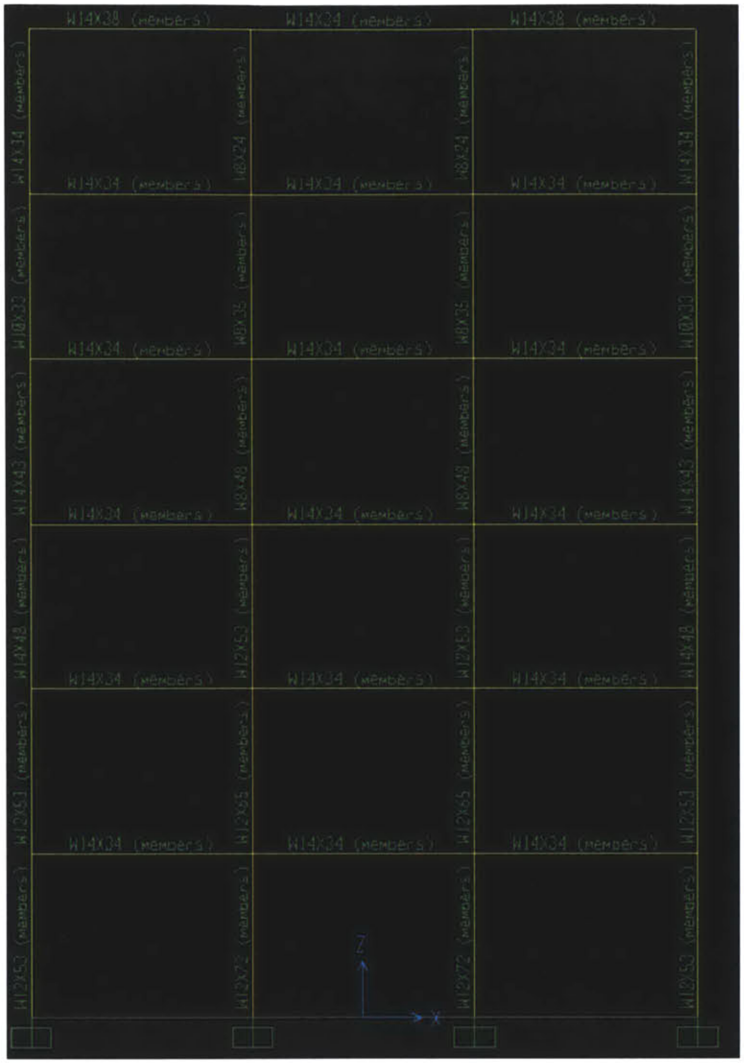


Figure 5: 2-D Building Model with Steel Member Labels

In addition to dimension, fixities, and typical member selection, a couple of additional details were added to the model to improve accuracy in modeling linear and nonlinear models. First, each floor was modeled as a rigid diaphragm by floor; this will be helpful in determining maximum interstory drift ratios, as the drift of every node per floor displaces the same amount. Second, plastic hinges were applied at both extremities of each steel member, which does not affect linear analysis and allows for nonlinear action under extreme loading in a nonlinear analysis. A photo of the model with plastic hinges can be seen in Figure 6.

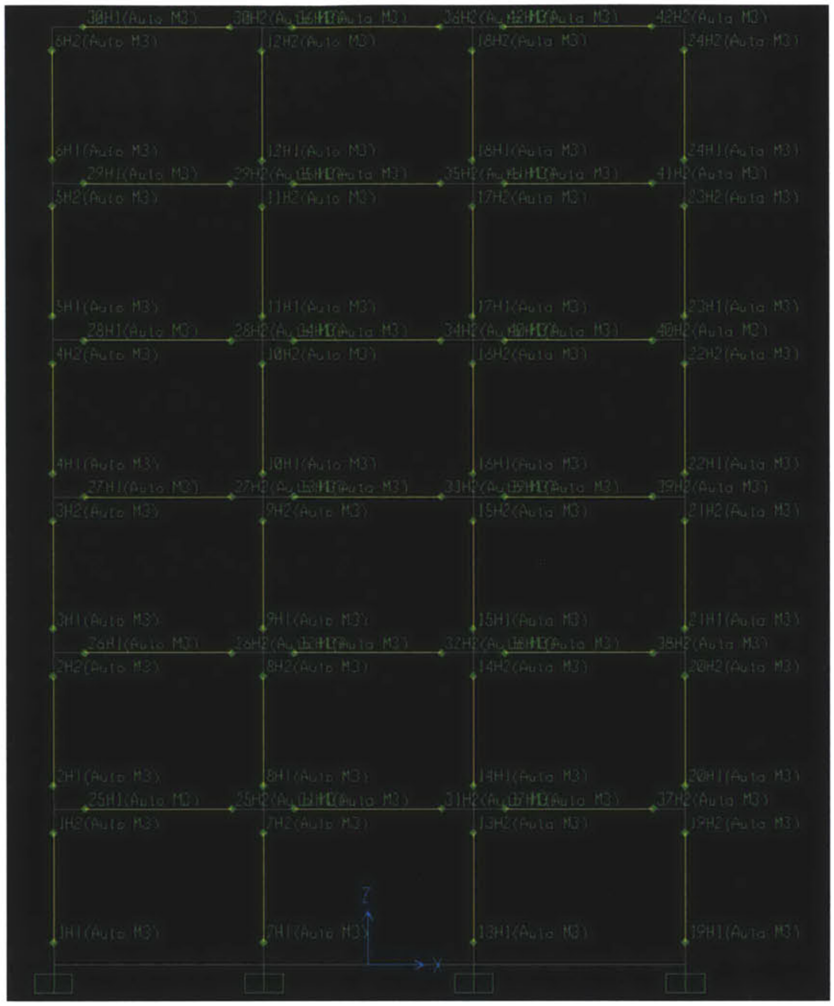


Figure 6: 2-D Building Model with Plastic Hinges

6.2 Earthquake Records

Due to the probabilistic nature of a building's response to a given earthquake intensity, three time-history records of California earthquakes were run through the SAP2000 building model: the San Fernando Earthquake of 1971, Loma Prieta Earthquake of 1989, and Northridge Earthquake of 1994. These records can be seen in Figure 7-Figure 9. In order to properly compare the building behavior under these different records, the earthquake records were scaled prior to each analysis so that all peak ground accelerations matched while keeping individual time-dependent characteristics. As this study seeks to determine the relationship between linear and nonlinear analyses under various seismic conditions, some records were scaled to unlikely intensities in order to see whether this relationship was intensity-dependent.

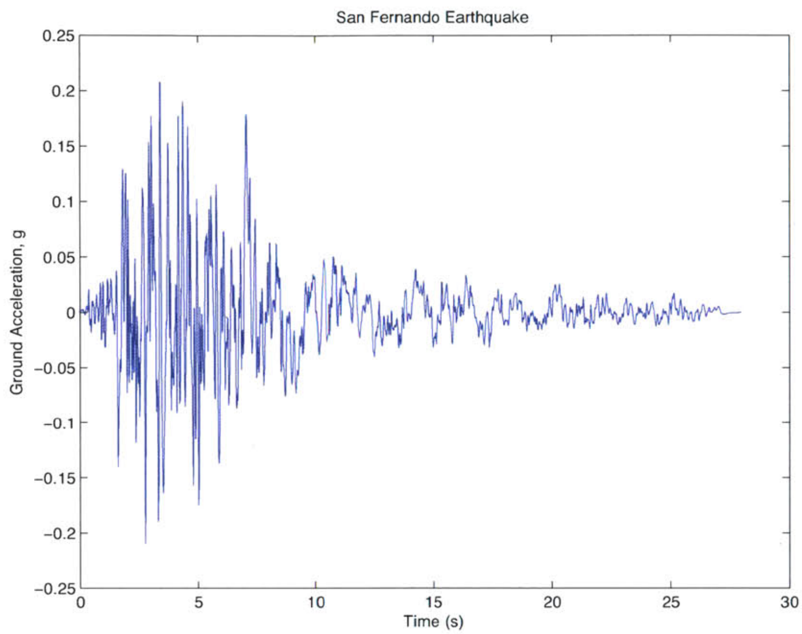


Figure 7: Record of San Fernando Earthquake

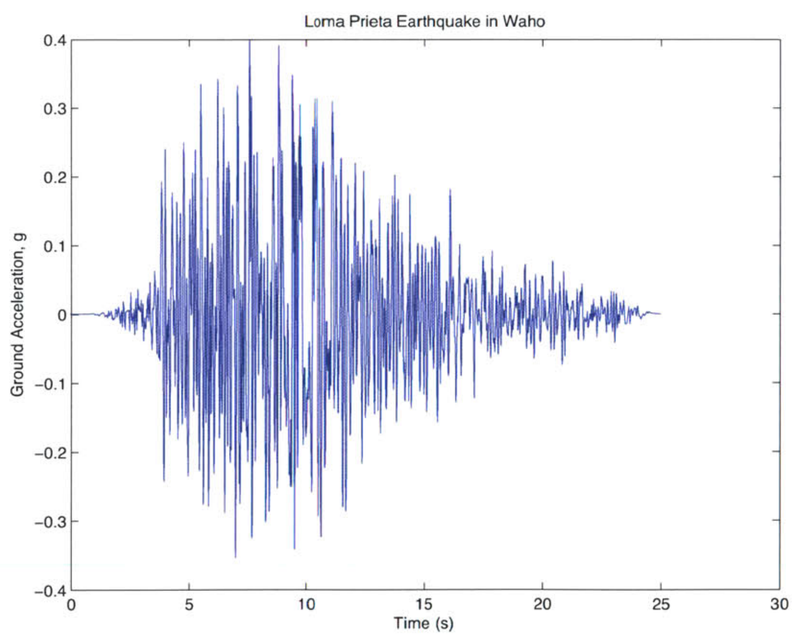


Figure 8: Record of Loma Prieta Earthquake

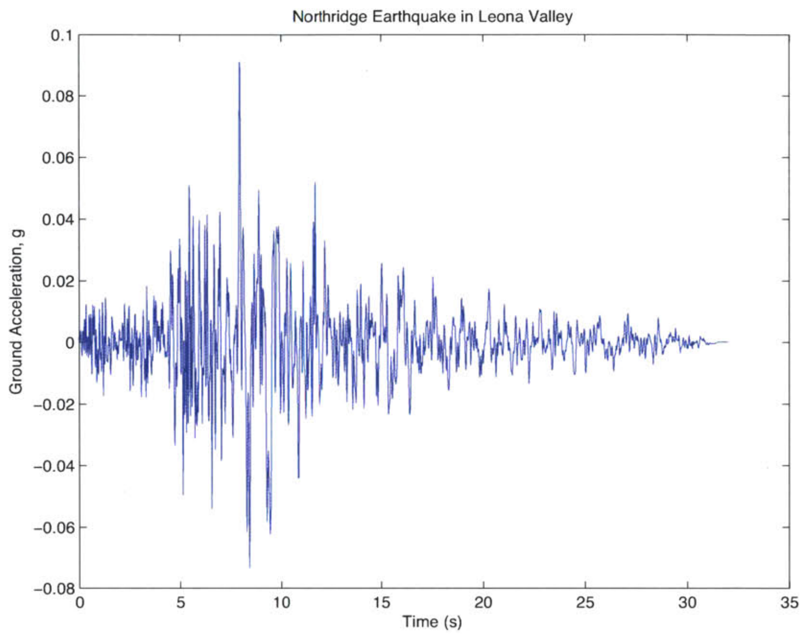


Figure 9: Record of Northridge Earthquake

6.3 Design Parameters

The overlying hope of this study is to determine whether linear analysis is an appropriate substitute for nonlinear analysis in the initial stages of design in order to achieve an optimal structural system. Within this general goal, the building's behavior under two variables was also observed in order to see whether the relationship between linear and nonlinear analysis depends on either. These variables are the damping of the structure and the earthquake intensity. It is the author's hope that, should these variables have a significant effect, design recommendations can be made from these relationships to advance PBEE's effort to have a larger role in building design.

Typically, concrete buildings have damping properties at around five percent of critical damping, while steel buildings commonly have damping in the range of two to three percent of critical damping.[25] By varying damping between two, five, and ten percent of critical damping in this study, it will be determined whether there is an optimal damping level for the purposes of using linear models in the early stages of design. In addition to varying damping, the intensity of the earthquakes will also be varied in order to see whether this has an effect on the linear-nonlinear relationship. Intensities will vary between 0.167g, 0.33g, 0.5g, 0.6g, 0.9g, g, 1.5g, 2g, and 3g. While an earthquake with a peak acceleration of 3g is rare, this value was chosen to see if a very severe earthquake has an effect on the observed relationship.

While several engineering demand parameters exist in PBEE, this study focused on the peak interstory drift ratio between all floors as the *EDP*. Due to the behavior of each floor as a diaphragm, it was only necessary to monitor one node per floor. *EDP* values for the linear and nonlinear analyses were compared for each set of variables.

6.4 Analysis Methods

When running an earthquake record through a structure in SAP2000, several linear and nonlinear analysis options exist. In this study, a linear time-history analysis was performed using the modal analysis procedure provided by SAP2000. This is the simplest and fastest way to perform a time-history seismic analysis. The nonlinear analysis was performed through means of direct integration. This method is more time-consuming than the modal "time-history type." However, due to the more intense computation completed in the direct integration case, results are more accurate and the data more reliable. The earthquake loads began with initial conditions after applying the gravity loads; this initial condition takes into account P-Δ effects. Figure 10 shows these options in a SAP2000 window.

[25] Moehle, "Performance-Based Seismic Design of Tall Buildings in the U.S.", 5.

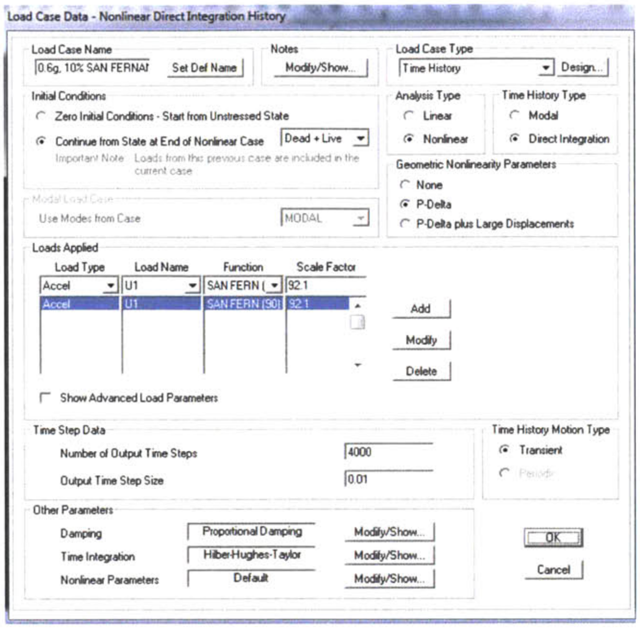


Figure 10: Display for Time History Load Case

6.5 Results

Results of the comparisons between linear and nonlinear analyses for the three earthquakes by varying both the building damping and earthquake intensities can be seen in Table 2Table 4; a full record of displacements and interstory drift ratios can be seen in the Appendix. In the tables, earthquake cases in which the building was unable to complete the analysis due to low damping and an overwhelming earthquake are denoted in black. While there are several interesting conclusions that can be drawn from the results, the fact that the errors for lower-level earthquake intensities are relatively small is one that is most relevant. For earthquake load cases in which the building does not form plastic hinges, noted in pink in the tables, linear and nonlinear results should be identical. This is not the case for the results, though this error can be attributed to the way the analysis was defined. For this SAP2000 model, building damping is specified only for the first two modes of the structure, and follows the "proportional damping" specification in the program. Had the damping been consistent for all modes, the results for linear and nonlinear analysis might converge, as it should.

Table 2: Percent Errors for Linear and Nonlinear Analyses for the San Fernando Earthquake

	2%	5%	10%
0.167g	35.22%	16.55%	7.41%
0.33g	32.93%	16.86%	7.41%
0.5g	42.60%	19.73%	8.34%
0.6g	91.87%	18.45%	4.82%
0.9g	64.14%	94.58%	5.76%
g	99.82%	99.53%	97.80%
1.5g			74.81%
No Plastic Hinge Formation			

Table 3: Percent Errors for Linear and Nonlinear Analyses for the Loma Prieta Earthquake

	2%	5%	10%
0.167g	22.73%	16.50%	11.93%
0.33g	22.73%	16.50%	11.93%
0.5g	22.73%	16.50%	11.93%
0.6g	36.55%	24.26%	11.93%
0.9g	43.58%	27.28%	11.93%
g	50.88%	27.12%	14.55%
1.5g		25.94%	12.92%
2g			4.49%
3g			99.39%
No Plastic Hinge Formation			

For earthquake intensities that create plastic hinges in the structure, it seems that the disparity between linear and nonlinear analysis remains relatively small until significantly larger earthquake intensities are experienced. In the case of the Loma Prieta Earthquake, the percent error between linear and nonlinear results remain at 11.93% through an intensity of 0.9g even though plastic hinges form beginning at an intensity of 0.5g. All three earthquakes show the same general behavior of the percent error only slightly increasing as the earthquake intensity increases until a certain magnitude, when the error increases dramatically. This is due to the fact that while plastic hinges form in the structure, there is not a large disparity in the linear and nonlinear analyses until the plastic hinges are fully developed in the structure, at which point the structure becomes unstable. Figure 9 shows the formation of plastic hinges for 10% building damping, when the percent error jumps from 12.32% for a g-level earthquake to 99.17% error for a 2g-level earthquake.

Table 4: Percent Errors for Linear and Nonlinear Analyses for the Northridge Earthquake

	2%	5%	10%
0.167g	11.16%	5.05%	6.87%
0.33g	11.16%	5.04%	6.87%
0.5g	10.82%	5.05%	6.87%
0.6g	9.10%	5.32%	7.28%
0.9g	2.91%	7.86%	8.64%
g	5.10%	13.50%	12.32%
1.5g		95.79%	7.63%
2g			99.17%
3g			70.88%
No Plastic Hinge Formation			

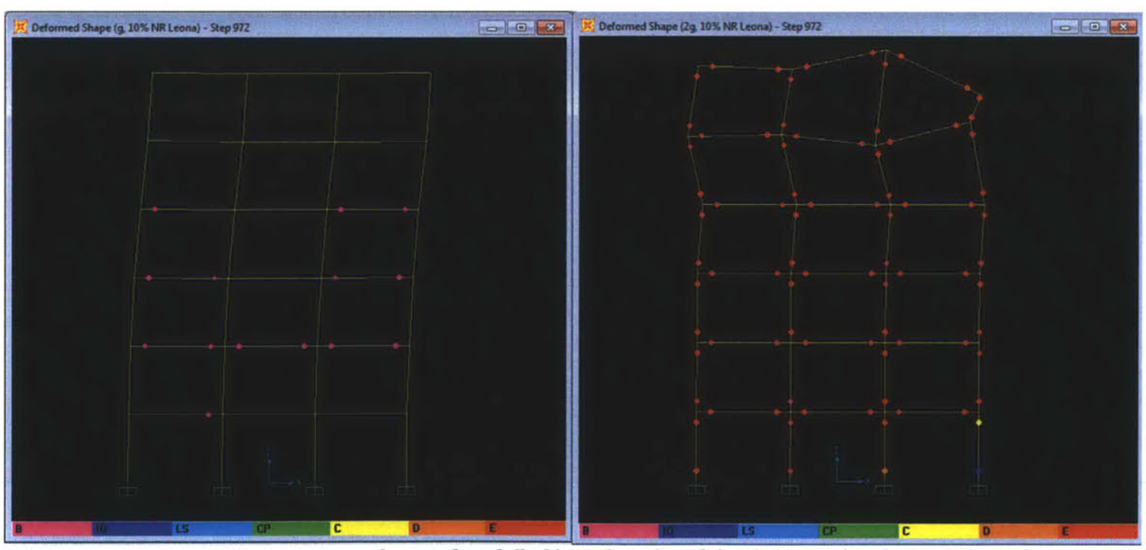


Figure 11: Plastic Hinges for a g-level (left) and 2g-level (right) Northridge Earthquake

In addition to earthquake intensity, the structure's damping was also studied in order to see if there is a correlation between error associated with both linear and nonlinear analyses and this parameter. While earthquake intensity may be the primary driver for diverging results, damping does have an effect due to the fact that plastic hinges form later in highly damped structures. Therefore, linear results will remain valid for a longer period of time with high damping.

While evaluating percent error is a good quantitative comparison of linear and nonlinear results, qualitatively seeing their comparison in a fragility function is also useful. A table of peak interstory drift ratios (story drift/story height) for the building in the Northridge Earthquake in Waho with 2% damping can be seen in Table 5. The linear and nonlinear results for each case can be plotted on the fragility function for a commercial building, seen in Figure 12. In the figure, the linear and nonlinear peak interstory drift ratios are close enough that damage estimates can approximately be found by using linear results in lieu of nonlinear ones. However, values will follow the same trend as the percent error trends, where higher damping and lower earthquake intensities yield a closer relationship between the two analyses.

Table 5: Peak Interstory Drift Ratio of Northridge Earthquake

	Linear (in/in)	Nonlinear (in/in)
g/6, 2% Waho	0.035	0.027
g/3, 2% Waho	0.069	0.055
g/2, 2% Waho	0.103	0.082
0.60g, 2% Waho	0.124	0.084
0.9g, 2% Waho	0.186	0.120
1g, 2% Waho	0.207	0.129

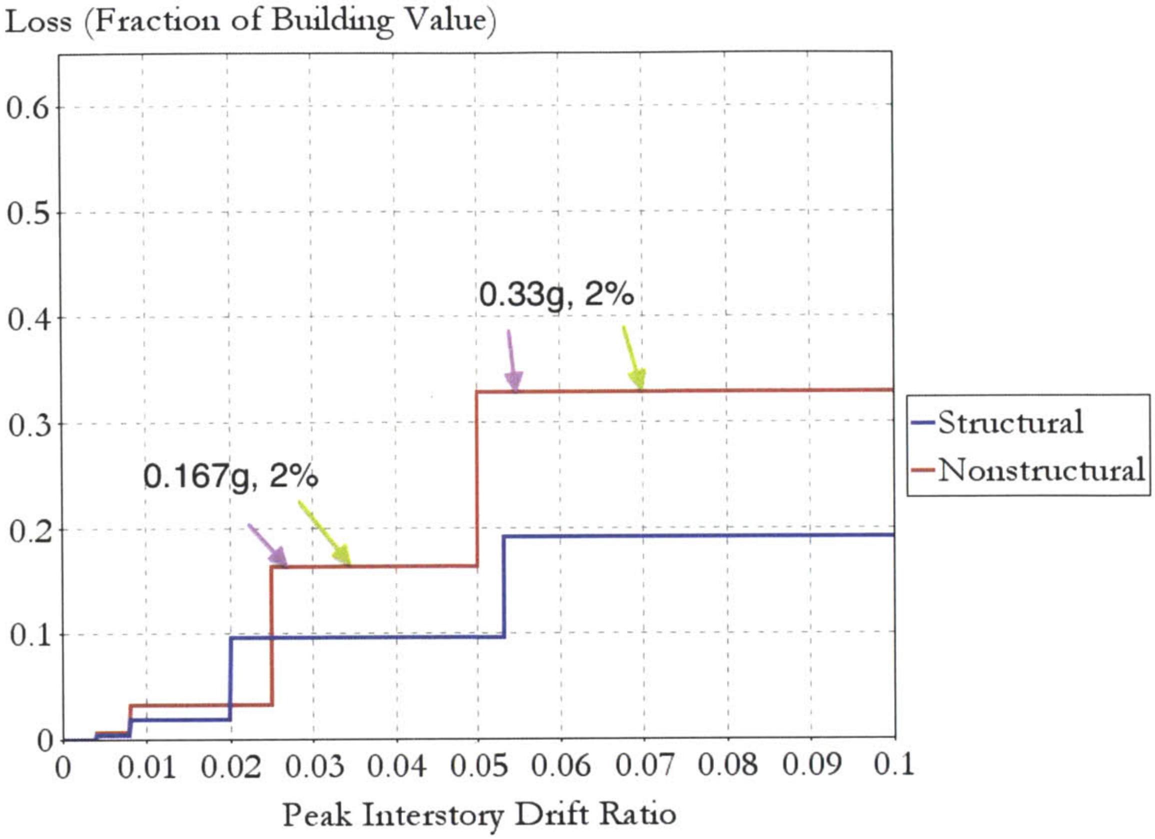


Figure 12: Fragility Function for a Commercial Building with Results from the Northridge Earthquake[26]

[26] HAZUS

7. Conclusion

Recent developments in earthquake engineering have the industry trending towards procedures that involve performance requirements in addition to traditional collapse prevention. This emphasis began in earnest when the Northridge Earthquake hit Southern California in 1994, causing severe structural damage in many of the structures in the area. The latest iteration of performance-based earthquake engineering, championed by PEER, has been developed over the last decade.

While PEER's current PBEE methodology is a good tool for verifying that a building meets building performance criteria developed by the engineers and stakeholders, it currently lacks a significant presence in the early design stages of structural design. Rather than driving the structural system design, current practice involves selecting a structural system for strength requirements and then checking and tuning the structure for serviceability afterwards. This usually results in a suboptimal design compared to one where performance is integrated into the design process and system selection from the beginning.

The biggest obstacle to creating an optimal design procedure in which the structural system is evaluated and iterated with performance criteria in mind is the fact that nonlinear analysis, the preferred and more accurate method in seismic analysis, is computationally costly. Due to the computational cost, running a nonlinear analysis takes a lot of time, and iterations using this analysis would be highly inefficient.

However, this study shows that it is possible to use linear analysis in lieu of nonlinear analysis in the early design stages in order to get relatively accurate results, depending on the building and earthquake properties. Results from this study show that highly damped structures are more optimal in the structural system iteration procedure due to the fact that large earthquakes are required in order for plastic hinges to form in the steel members. As the error associated with using linear analysis instead of nonlinear analysis depends on plastic hinge formation in the structure, having a highly damped structure will allow for the use of linear analysis for a larger range of earthquake intensities. As nonlinear analysis is a better procedure in accurately predicting a building's performance, it should still be used on the building after the structural system is chosen and the entire building is detailed.

8. Bibliography

Deierlein, Greg, Andrei Reinhorn and Michael Willford. "Nonlinear Structural Analysis for Seismic Design." NEHRP Seismic Design Technical Brief No. 4. 2010.

Dogruel, Seda and Gary Dargush. "A Framework for Multi-Hazard Design and Retrofit of Passively Damped Structures." AEI 2008: Building Integration Solutions. American Society of Civil Engineers, 2008.

Federal Emergency Management Agency. HAZUS: FEMA's Methodology for Estimating Potential Losses from Disasters. <http://www.fema.gov/plan/prevent/hazus/>.

Hamburger, Ronaldo. A Framework for Performance-Based Earthquake Resistive Design. <http://nisee.berkeley.edu/lessons/hamburger.html>.

Holmes, William, et al. "Seismic Performance Objectives for Tall Buildings." Pacific Earthquake Engineering Research Center, 2008.

Krawinkler, Helmut, et al. "Decision Support for Conceptual Performance-Based Design." Earthquake Engineering and Structural Dynamics 35 (2006): 115-133.

Lin, Jean. "Preparing for the "Big One": Saving Lives through Earthquake Mitigation in Los Angeles, California." Office of Policy and Development Research, n.d.

Miyamoto, H.Kit and Amir Gilani. "Design of a New Steel-Framed Building Using ASCE 7 Damper Provisions." Structures 2008: Crossing Borders. American Society of Civil Engineers, 2008.

Moehle, Jack and Greg Deierlein. "A Framework Methodology for Performance-Based Earthquake Engineering." 13th World Conference on Earthquake Engineering. Vancouver, 2004.

Moehle, Jack. "Nonlinear Analysis for Performance-Based Earthquake Engineering." The Structural Design of Tall and Special Buildings 2005: 385-400.

Moehle, Jack. "Performance-Based Seismic Design of Tall Buildings in the U.S." Proceedings of the 2008 World Conference on Earthquake Engineering. n.d. 1-8.

Moehle, Jack, et al. "An Application of PEER Performance-Based Earthquake Engineering Methodology." Research Digest No. 2005-1 (2005): 1-4.

Pacific Earthquake Engineering Research Center. PEER Mission Goals. <http://peer.berkeley.edu/about/mission_goals.html>.

Pacific Earthquake Engineering Research Center. What is PEER? <http://peer.berkeley.edu/about/what_is_peer.html>.

Rojas, Hugo, Shahram Pezeshk and Christopher Foley. "Automatic Risk-Based Seismic Design Method of Optimal Structural and Non-Structural System Performance." 18th Analysis and Computation Specialty Conference. American Society of Civil Engineers, 2008.

Structural Engineers Association of California (SEAOC). Recommended Lateral Force Requirements and Commentary. San Francisco: SEAOC, 1960.

Zareian, Farzin and Helmut Krawinkler. "Assessment of Probability of Collapse and Design for Collapse Safety." Earthquake Engineering and Structural Dynamics 36 (2007): 1901-1914.

9. Appendix

Summary of Percent Errors for San Fernando Earthquake

	2%	5%	10%
g/6	35.22%	16.55%	7.41%
g/3	32.93%	16.86%	7.41%
g/2	42.60%	19.73%	8.34%
0.6g	91.87%	18.45%	4.82%
0.9g	64.14%	94.58%	5.76%
g	99.82%	99.53%	97.80%
1.5g			74.81%

No Plastic Hinge Formation

Peak Interstory Drift Ratios for San Fernando Earthquake

	Linear (in/in)	Nonlinear (in/in)
g/6, 2% SAN FERNANDO 90d	0.083	0.069
g/3, 2% SAN FERNANDO 90d	0.166	0.137
g/2, 2% SAN FERNANDO 90d	0.249	0.191
0.6g, 2% SAN FERNANDO 90d	0.299	48.838
0.9g, 2% SAN FERNANDO 90d	0.448	15.800
1g, 2% SAN FERNANDO 90d	0.498	610.167

	Linear (in/in)	Nonlinear (in/in)
g/6, 5% SAN FERNANDO 90d	0.055	0.054
g/3, 5% SAN FERNANDO 90d	0.111	0.108
g/2, 5% SAN FERNANDO 90d	0.166	0.157
0.6g, 5% SAN FERNANDO	0.200	0.191
0.9g, 5% SAN FERNANDO	0.300	172.003
1g, 5% SAN FERNANDO 90d	0.332	634.505

	Linear (in/in)	Nonlinear (in/in)
g/6, 10% SAN FERNANDO 90	0.033	0.039
g/3, 10% SAN FERNANDO 90	0.067	0.079
g/2, 10% SAN FERNANDO 90	0.100	0.118
0.6g, 10% SAN FERNANDO	0.137	0.138
0.9g, 10% SAN FERNANDO	0.205	0.212
g, 10% SAN FERNANDO	0.228	225.767
1.5g, 10% SAN FERNANDO	0.341	10.996

Percent Error for 2% Damping for San Fernando Earthquake

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/6, 2% SAN FERNANDO 90d	1.09	0.74	47.82%
23	Translation	g/6, 2% SAN FERNANDO 90d	3.85	2.66	44.52%
34	Translation	g/6, 2% SAN FERNANDO 90d	7.07	5.09	39.12%
45	Translation	g/6, 2% SAN FERNANDO 90d	10.15	7.65	32.75%
56	Translation	g/6, 2% SAN FERNANDO 90d	12.97	10.25	26.54%
67	Translation	g/6, 2% SAN FERNANDO 90d	14.94	12.39	20.58%
	Average				35.22%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/3, 2% SAN FERNANDO 90d	2.19	1.48	48.18%
23	Translation	g/3, 2% SAN FERNANDO 90d	7.70	5.48	40.56%
34	Translation	g/3, 2% SAN FERNANDO 90d	14.15	10.53	34.34%
45	Translation	g/3, 2% SAN FERNANDO 90d	20.30	15.76	28.81%
56	Translation	g/3, 2% SAN FERNANDO 90d	25.94	20.80	24.73%
67	Translation	g/3, 2% SAN FERNANDO 90d	29.87	24.70	20.94%
	Average				32.93%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/2, 2% SAN FERNANDO 90d	3.28	2.03	61.43%
23	Translation	g/2, 2% SAN FERNANDO 90d	11.55	7.60	51.89%
34	Translation	g/2, 2% SAN FERNANDO 90d	21.22	14.85	42.89%
45	Translation	g/2, 2% SAN FERNANDO 90d	30.45	22.32	36.43%
56	Translation	g/2, 2% SAN FERNANDO 90d	38.91	29.31	32.75%
67	Translation	g/2, 2% SAN FERNANDO 90d	44.81	34.41	30.22%
	Average				42.60%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	0.6g, 2% SAN FERNANDO 90d	3.94	119.32	96.70%
23	Translation	0.6g, 2% SAN FERNANDO 90d	13.86	4393.94	99.68%
34	Translation	0.6g, 2% SAN FERNANDO 90d	25.48	100.70	74.70%
45	Translation	0.6g, 2% SAN FERNANDO 90d	36.56	8790.85	99.58%
56	Translation	0.6g, 2% SAN FERNANDO 90d	46.72	254.48	81.64%
67	Translation	0.6g, 2% SAN FERNANDO 90d	53.80	5032.73	98.93%
	Average				91.87%

Percent Error for 2% Damping for San Fernando Earthquake

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	0.9g, 2% SAN FERNANDO 90d	5.91	8.13	27.27%
23	Translation	0.9g, 2% SAN FERNANDO 90d	20.79	51.54	59.67%
34	Translation	0.9g, 2% SAN FERNANDO 90d	38.21	42.19	9.44%
45	Translation	0.9g, 2% SAN FERNANDO 90d	54.82	2632.48	97.92%
56	Translation	0.9g, 2% SAN FERNANDO 90d	70.05	1053.81	93.35%
67	Translation	0.9g, 2% SAN FERNANDO 90d	80.67	2843.99	97.16%
	Average				64.14%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	1g, 2% SAN FERNANDO 90d	6.57	33257.28	99.98%
23	Translation	1g, 2% SAN FERNANDO 90d	23.09	17333.19	99.87%
34	Translation	1g, 2% SAN FERNANDO 90d	42.45	7236.83	99.41%
45	Translation	1g, 2% SAN FERNANDO 90d	60.90	109830.00	99.94%
56	Translation	1g, 2% SAN FERNANDO 90d	77.82	104873.10	99.93%
67	Translation	1g, 2% SAN FERNANDO 90d	89.62	40043.17	99.78%
	Average				99.82%

Percent Error for 5% Damping for San Fernando Earthquake

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/6, 5% SAN FERNANDO 90d	0.72	0.54	32.77%
23	Translation	g/6, 5% SAN FERNANDO 90d	2.52	2.00	26.06%
34	Translation	g/6, 5% SAN FERNANDO 90d	4.64	3.91	18.75%
45	Translation	g/6, 5% SAN FERNANDO 90d	6.67	5.93	12.46%
56	Translation	g/6, 5% SAN FERNANDO 90d	8.57	8.01	6.97%
67	Translation	g/6, 5% SAN FERNANDO 90d	9.95	9.73	2.27%
Average					16.55%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/3, 5% SAN FERNANDO 90d	1.44	1.07	34.09%
23	Translation	g/3, 5% SAN FERNANDO 90d	5.05	3.96	27.32%
34	Translation	g/3, 5% SAN FERNANDO 90d	9.28	7.83	18.53%
45	Translation	g/3, 5% SAN FERNANDO 90d	13.34	11.90	12.08%
56	Translation	g/3, 5% SAN FERNANDO 90d	17.14	16.04	6.88%
67	Translation	g/3, 5% SAN FERNANDO 90d	19.90	19.46	2.27%
Average					16.86%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/2, 5% SAN FERNANDO 90d	2.15	1.53	40.97%
23	Translation	g/2, 5% SAN FERNANDO 90d	7.57	5.73	32.14%
34	Translation	g/2, 5% SAN FERNANDO 90d	13.92	11.60	20.04%
45	Translation	g/2, 5% SAN FERNANDO 90d	20.01	17.90	11.79%
56	Translation	g/2, 5% SAN FERNANDO 90d	25.71	23.81	7.98%
67	Translation	g/2, 5% SAN FERNANDO 90d	29.85	28.30	5.46%
Average					19.73%

12	Translation	0.6g, 5% SAN FERNANDO	2.58	1.82	41.65%
23	Translation	0.6g, 5% SAN FERNANDO	9.09	6.93	31.18%
34	Translation	0.6g, 5% SAN FERNANDO	16.71	14.17	17.96%
45	Translation	0.6g, 5% SAN FERNANDO	24.03	21.97	9.36%
56	Translation	0.6g, 5% SAN FERNANDO	30.87	29.15	5.90%
67	Translation	0.6g, 5% SAN FERNANDO	35.97	34.37	4.66%
Average					18.45%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	0.9g, 5% SAN FERNANDO	3.88	13.03	70.25%
23	Translation	0.9g, 5% SAN FERNANDO	13.63	76.83	82.26%
34	Translation	0.9g, 5% SAN FERNANDO	25.06	14621.20	99.83%
45	Translation	0.9g, 5% SAN FERNANDO	36.03	895.17	95.98%
56	Translation	0.9g, 5% SAN FERNANDO	46.29	30960.50	99.85%
67	Translation	0.9g, 5% SAN FERNANDO	53.93	327.60	83.54%
Average					88.62%

Percent Error for 5% Damping for San Fernando Earthquake

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	1g, 5% SAN FERNANDO 90d	4.31	1365.87	99.68%
23	Translation	1g, 5% SAN FERNANDO 90d	15.14	114210.88	99.99%
34	Translation	1g, 5% SAN FERNANDO 90d	27.84	99426.79	99.97%
45	Translation	1g, 5% SAN FERNANDO 90d	40.02	9079.56	99.56%
56	Translation	1g, 5% SAN FERNANDO 90d	51.42	7928.11	99.35%
67	Translation	1g, 5% SAN FERNANDO 90d	59.70	4409.08	98.65%
Average					99.53%

Percent Error for 10% Damping for San Fernando Earthquake

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/6, 10% SAN FERNANDO 90	0.43	0.40	7.56%
23	Translation	g/6, 10% SAN FERNANDO 90	1.52	1.46	3.78%
34	Translation	g/6, 10% SAN FERNANDO 90	2.78	2.80	0.82%
45	Translation	g/6, 10% SAN FERNANDO 90	4.00	4.24	5.66%
56	Translation	g/6, 10% SAN FERNANDO 90	5.14	5.79	11.15%
67	Translation	g/6, 10% SAN FERNANDO 90	5.99	7.09	15.51%
Average					7.41%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/3, 10% SAN FERNANDO 90	0.86	0.80	7.56%
23	Translation	g/3, 10% SAN FERNANDO 90	3.03	2.92	3.78%
34	Translation	g/3, 10% SAN FERNANDO 90	5.56	5.61	0.82%
45	Translation	g/3, 10% SAN FERNANDO 90	7.99	8.47	5.66%
56	Translation	g/3, 10% SAN FERNANDO 90	10.29	11.58	11.15%
67	Translation	g/3, 10% SAN FERNANDO 90	11.97	14.17	15.51%
Average					7.41%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/2, 10% SAN FERNANDO 90	1.30	1.18	10.03%
23	Translation	g/2, 10% SAN FERNANDO 90	4.55	4.33	4.94%
34	Translation	g/2, 10% SAN FERNANDO 90	8.34	8.46	1.33%
45	Translation	g/2, 10% SAN FERNANDO 90	11.99	12.84	6.63%
56	Translation	g/2, 10% SAN FERNANDO 90	15.43	17.46	11.58%
67	Translation	g/2, 10% SAN FERNANDO 90	17.96	21.27	15.55%
Average					8.34%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	0.6g, 10% SAN FERNANDO	1.56	1.36	14.54%
23	Translation	0.6g, 10% SAN FERNANDO	5.46	5.01	8.94%
34	Translation	0.6g, 10% SAN FERNANDO	10.09	9.91	1.86%
45	Translation	0.6g, 10% SAN FERNANDO	15.03	15.17	0.92%
56	Translation	0.6g, 10% SAN FERNANDO	20.25	20.56	1.53%
67	Translation	0.6g, 10% SAN FERNANDO	24.60	24.88	1.13%
Average					4.82%

Percent Error for 10% Damping for San Fernando Earthquake

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	0.9g, 10% SAN FERNANDO	2.34	2.09	11.76%
23	Translation	0.9g, 10% SAN FERNANDO	8.18	7.85	4.25%
34	Translation	0.9g, 10% SAN FERNANDO	15.14	15.65	3.29%
45	Translation	0.9g, 10% SAN FERNANDO	22.54	24.03	6.21%
56	Translation	0.9g, 10% SAN FERNANDO	30.36	32.20	5.72%
67	Translation	0.9g, 10% SAN FERNANDO	36.88	38.16	3.34%
	Average				5.76%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g, 10% SAN FERNANDO	2.59	49.63	94.77%
23	Translation	g, 10% SAN FERNANDO	9.09	212.85	95.73%
34	Translation	g, 10% SAN FERNANDO	16.81	17955.67	99.91%
45	Translation	g, 10% SAN FERNANDO	25.04	2028.34	98.77%
56	Translation	g, 10% SAN FERNANDO	33.73	40638.03	99.92%
67	Translation	g, 10% SAN FERNANDO	40.98	1805.05	97.73%
	Average				97.80%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	1.5g, 10% SAN FERNANDO	3.89	3.62	7.62%
23	Translation	1.5g, 10% SAN FERNANDO	13.64	28.99	52.96%
34	Translation	1.5g, 10% SAN FERNANDO	25.22	763.04	96.69%
45	Translation	1.5g, 10% SAN FERNANDO	37.55	1754.57	97.86%
56	Translation	1.5g, 10% SAN FERNANDO	50.58	1599.23	96.84%
67	Translation	1.5g, 10% SAN FERNANDO	61.46	1979.32	96.90%
	Average				74.81%

Summary of Percent Errors for Loma Prieta Earthquake in Waho

	2%	5%	10%
g/6	22.73%	16.50%	11.93%
g/3	22.73%	16.50%	11.93%
g/2	22.73%	16.50%	11.93%
0.6g	36.55%	24.26%	11.93%
0.9g	43.58%	27.28%	11.93%
g	50.88%	27.12%	14.55%
1.5g		25.94%	12.92%
2g			4.49%
3g			99.39%
No Plastic Hinge Formation			

Peak Interstory Drift Ratios for Loma Prieta Earthquake in Waho

	Linear (in/in)	Nonlinear (in/in)
g/6, 2% Waho	0.035	0.027
g/3, 2% Waho	0.069	0.055
g/2, 2% Waho	0.103	0.082
0.60g, 2% Waho	0.124	0.084
0.9g, 2% Waho	0.186	0.120
1g, 2% Waho	0.207	0.129

	Linear (in/in)	Nonlinear (in/in)
g/6, 5% Waho	0.027	0.022
g/3, 5% Waho	0.055	0.044
g/2, 5% Waho	0.082	0.066
0.6g, 5% Waho	0.132	0.100
0.9g, 5% Waho	0.148	0.106
1g, 5% Waho	0.164	0.118
1.5g, 5% Waho	0.247	0.188

	Linear (in/in)	Nonlinear (in/in)
g/6, 10% Waho	0.020	0.017
g/3, 10% Waho	0.040	0.033
g/2, 10% Waho	0.060	0.050
0.6g, 10% Waho	0.072	0.060
0.9g, 10% Waho	0.109	0.089
1g, 10% Waho	0.120	0.097
1.5g, 10% Waho	0.181	0.148
2g, 10% Waho	0.241	0.221
3g, 10% Waho	0.362	143.757

Percent Error for 2% Damping for Loma Prieta Earthquake in Waho

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/6, 2% Waho	0.44	0.36	23.89%
23	Translation	g/6, 2% Waho	1.46	1.22	19.75%
34	Translation	g/6, 2% Waho	2.64	2.19	20.50%
45	Translation	g/6, 2% Waho	3.82	3.11	22.90%
56	Translation	g/6, 2% Waho	5.02	4.09	22.83%
67	Translation	g/6, 2% Waho	6.22	4.92	26.50%
	Average				22.73%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/3, 2% Waho	0.88	0.71	23.89%
23	Translation	g/3, 2% Waho	2.92	2.44	19.75%
34	Translation	g/3, 2% Waho	5.27	4.37	20.50%
45	Translation	g/3, 2% Waho	7.65	6.22	22.90%
56	Translation	g/3, 2% Waho	10.04	8.17	22.83%
67	Translation	g/3, 2% Waho	12.44	9.83	26.50%
	Average				22.73%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/2, 2% Waho	1.32	1.06	23.89%
23	Translation	g/2, 2% Waho	4.37	3.65	19.75%
34	Translation	g/2, 2% Waho	7.89	6.55	20.50%
45	Translation	g/2, 2% Waho	11.44	9.31	22.90%
56	Translation	g/2, 2% Waho	15.02	12.23	22.83%
67	Translation	g/2, 2% Waho	18.61	14.72	26.50%
	Average				22.73%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	0.60g, 2% Waho	1.58	1.13	40.20%
23	Translation	0.60g, 2% Waho	5.25	4.02	30.42%
34	Translation	0.60g, 2% Waho	9.47	7.27	30.31%
45	Translation	0.60g, 2% Waho	13.73	10.14	35.46%
56	Translation	0.60g, 2% Waho	18.03	13.35	35.02%
67	Translation	0.60g, 2% Waho	22.35	15.11	47.89%
	Average				36.55%

Percent Error for 2% Damping for Loma Prieta Earthquake in Waho

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	0.9g, 2% Waho	2.37	1.56	52.50%
23	Translation	0.9g, 2% Waho	7.88	5.64	39.67%
34	Translation	0.9g, 2% Waho	14.21	10.31	37.87%
45	Translation	0.9g, 2% Waho	20.61	15.17	35.85%
56	Translation	0.9g, 2% Waho	27.07	19.33	40.00%
67	Translation	0.9g, 2% Waho	33.54	21.56	55.55%
	Average				43.58%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	1g, 2% Waho	2.63	1.61	63.49%
23	Translation	1g, 2% Waho	8.74	5.85	49.56%
34	Translation	1g, 2% Waho	15.77	10.71	47.31%
45	Translation	1g, 2% Waho	22.88	16.30	40.35%
56	Translation	1g, 2% Waho	30.04	20.79	44.52%
67	Translation	1g, 2% Waho	37.23	23.26	60.02%
	Average				50.88%

Percent Error for 5% Damping for Loma Prieta Earthquake in Waho

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/6, 5% Waho	0.32	0.28	12.70%
23	Translation	g/6, 5% Waho	1.12	0.98	13.94%
34	Translation	g/6, 5% Waho	2.08	1.80	15.53%
45	Translation	g/6, 5% Waho	3.01	2.59	16.09%
56	Translation	g/6, 5% Waho	3.95	3.38	16.84%
67	Translation	g/6, 5% Waho	4.94	3.99	23.87%
Average					16.50%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/3, 5% Waho	0.63	0.56	12.70%
23	Translation	g/3, 5% Waho	2.24	1.97	13.94%
34	Translation	g/3, 5% Waho	4.16	3.60	15.53%
45	Translation	g/3, 5% Waho	6.02	5.19	16.09%
56	Translation	g/3, 5% Waho	7.89	6.75	16.84%
67	Translation	g/3, 5% Waho	9.88	7.98	23.87%
Average					16.50%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/2, 5% Waho	0.95	0.84	12.70%
23	Translation	g/2, 5% Waho	3.35	2.94	13.94%
34	Translation	g/2, 5% Waho	6.23	5.39	15.53%
45	Translation	g/2, 5% Waho	9.01	7.76	16.09%
56	Translation	g/2, 5% Waho	11.81	10.10	16.84%
67	Translation	g/2, 5% Waho	14.79	11.94	23.87%
Average					16.50%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	0.6g, 5% Waho	1.51	1.26	20.27%
23	Translation	0.6g, 5% Waho	5.37	4.43	21.21%
34	Translation	0.6g, 5% Waho	9.97	8.04	24.01%
45	Translation	0.6g, 5% Waho	14.43	11.60	24.36%
56	Translation	0.6g, 5% Waho	18.91	15.19	24.47%
67	Translation	0.6g, 5% Waho	23.69	18.05	31.26%
Average					24.26%

Percent Error for 5% Damping for Loma Prieta Earthquake in Waho

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	0.9g, 5% Waho	1.70	1.42	20.22%
23	Translation	0.9g, 5% Waho	6.04	5.00	20.92%
34	Translation	0.9g, 5% Waho	11.22	9.06	23.80%
45	Translation	0.9g, 5% Waho	16.24	12.65	28.34%
56	Translation	0.9g, 5% Waho	21.27	16.29	30.57%
67	Translation	0.9g, 5% Waho	26.65	19.06	39.81%
Average					27.28%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	1g, 5% Waho	1.89	1.55	21.98%
23	Translation	1g, 5% Waho	6.70	5.49	22.14%
34	Translation	1g, 5% Waho	12.45	9.99	24.65%
45	Translation	1g, 5% Waho	18.02	14.34	25.65%
56	Translation	1g, 5% Waho	23.61	18.32	28.92%
67	Translation	1g, 5% Waho	29.58	21.22	39.37%
Average					27.12%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	1.5g, 5% Waho	2.84	2.20	28.96%
23	Translation	1.5g, 5% Waho	10.06	7.93	26.90%
34	Translation	1.5g, 5% Waho	18.69	14.91	25.33%
45	Translation	1.5g, 5% Waho	27.06	22.41	20.73%
56	Translation	1.5g, 5% Waho	35.45	28.93	22.51%
67	Translation	1.5g, 5% Waho	44.41	33.84	31.23%
Average					25.94%

Percent Error for 10% Damping for Loma Prieta Earthquake in Waho

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/6, 10% Waho	0.23	0.20	12.63%
23	Translation	g/6, 10% Waho	0.80	0.72	10.59%
34	Translation	g/6, 10% Waho	1.47	1.35	8.87%
45	Translation	g/6, 10% Waho	2.12	1.97	7.56%
56	Translation	g/6, 10% Waho	2.83	2.56	10.65%
67	Translation	g/6, 10% Waho	3.62	2.99	21.28%
	Average				11.93%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/3, 10% Waho	0.46	0.41	12.63%
23	Translation	g/3, 10% Waho	1.60	1.45	10.59%
34	Translation	g/3, 10% Waho	2.95	2.71	8.87%
45	Translation	g/3, 10% Waho	4.24	3.94	7.56%
56	Translation	g/3, 10% Waho	5.66	5.12	10.65%
67	Translation	g/3, 10% Waho	7.24	5.97	21.29%
	Average				11.93%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/2, 10% Waho	0.69	0.61	12.63%
23	Translation	g/2, 10% Waho	2.40	2.17	10.59%
34	Translation	g/2, 10% Waho	4.41	4.05	8.87%
45	Translation	g/2, 10% Waho	6.34	5.90	7.56%
56	Translation	g/2, 10% Waho	8.47	7.65	10.65%
67	Translation	g/2, 10% Waho	10.84	8.94	21.29%
	Average				11.93%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	0.6g, 10% Waho	0.83	0.74	12.63%
23	Translation	0.6g, 10% Waho	2.88	2.60	10.59%
34	Translation	0.6g, 10% Waho	5.29	4.86	8.87%
45	Translation	0.6g, 10% Waho	7.62	7.08	7.56%
56	Translation	0.6g, 10% Waho	10.17	9.19	10.65%
67	Translation	0.6g, 10% Waho	13.01	10.73	21.29%
	Average				11.93%

Percent Error for 10% Damping for Loma Prieta Earthquake in Waho

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	0.9g, 10% Waho	1.24	1.10	12.63%
23	Translation	0.9g, 10% Waho	4.32	3.91	10.60%
34	Translation	0.9g, 10% Waho	7.94	7.30	8.87%
45	Translation	0.9g, 10% Waho	11.43	10.63	7.55%
56	Translation	0.9g, 10% Waho	15.26	13.79	10.64%
67	Translation	0.9g, 10% Waho	19.53	16.11	21.29%
	Average				11.93%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	1g, 10% Waho	1.38	1.23	12.62%
23	Translation	1g, 10% Waho	4.79	4.24	13.06%
34	Translation	1g, 10% Waho	8.82	7.85	12.28%
45	Translation	1g, 10% Waho	12.69	11.44	10.87%
56	Translation	1g, 10% Waho	16.94	14.88	13.87%
67	Translation	1g, 10% Waho	21.68	17.40	24.59%
	Average				14.55%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	1.5g, 10% Waho	2.07	1.87	10.64%
23	Translation	1.5g, 10% Waho	7.20	6.60	9.15%
34	Translation	1.5g, 10% Waho	13.23	11.99	10.34%
45	Translation	1.5g, 10% Waho	19.05	17.14	11.15%
56	Translation	1.5g, 10% Waho	25.43	22.30	14.02%
67	Translation	1.5g, 10% Waho	32.55	26.63	22.22%
	Average				12.92%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	2g, 10% Waho	2.76	2.58	7.06%
23	Translation	2g, 10% Waho	9.60	9.25	3.75%
34	Translation	2g, 10% Waho	17.64	17.07	3.38%
45	Translation	2g, 10% Waho	25.39	25.03	1.43%
56	Translation	2g, 10% Waho	33.90	33.19	2.14%
67	Translation	2g, 10% Waho	43.39	39.73	9.21%
	Average				4.49%

Percent Error for 10% Damping for Loma Prieta Earthquake in Waho

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	3g, 10% Waho	4.14	8089.06	99.95%
23	Translation	3g, 10% Waho	14.39	4472.80	99.68%
34	Translation	3g, 10% Waho	26.46	23625.68	99.89%
45	Translation	3g, 10% Waho	38.08	25876.28	99.85%
56	Translation	3g, 10% Waho	50.84	1906.96	97.33%
67	Translation	3g, 10% Waho	65.07	17872.16	99.64%
Average					99.39%

Summary of Percent Errors of Interstory Drifts for Northridge Earthquake in Leona Valley

	2%	5%	10%
g/6	11.16%	5.05%	6.87%
g/3	11.16%	5.04%	6.87%
g/2	10.82%	5.05%	6.87%
0.6g	9.10%	5.32%	7.28%
0.9g	2.91%	7.86%	8.64%
g	5.10%	13.50%	12.32%
1.5g		95.79%	7.63%
2g			99.17%
3g			70.88%
No Plastic Hinge Formation			

Peak Interstory Drift Ratios for Northridge Earthquake in Leona Valley

	Linear (in/in)	Nonlinear (in/in)
g/6, 2% NR Leona	0.030	0.031
g/3, 2% NR Leona	0.061	0.062
g/2, 2% NR Leona	0.091	0.093
0.6g, 2% NR Leona	0.109	0.110
0.9g, 2% NR Leona	0.164	0.151
1g, 2% NR Leona	0.182	0.182

	Linear (in/in)	Nonlinear (in/in)
g/6, 5% NR Leona	0.024	0.022
g/3, 5% NR Leona	0.048	0.044
g/2, 5% NR Leona	0.072	0.066
0.6g, 5% NR Leona	0.087	0.079
0.9g, 5% NR Leona	0.130	0.114
1g, 5% NR Leona	0.144	0.125
1.5g, 5% NR Leona	0.216	51.901

	Linear (in/in)	Nonlinear (in/in)
g/6, 10% NR Leona	0.021	0.020
g/3, 10% NR Leona	0.042	0.040
g/2, 10% NR Leona	0.064	0.059
0.6g, 10% NR Leona	0.076	0.071
0.9g, 10% NR Leona	0.115	0.106
1g, 10% NR Leona	0.127	0.114
1.5g, 10% NR Leona	0.191	0.175
2g, 10% NR Leona	0.255	54.724
3g, 10% NR Leona	0.382	43.496

Percent Error of Interstory Drifts for 2% Damping for Northridge Earthquake in Leona Valley

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/6, 2% NR Leona	0.32	0.38	14.89%
23	Translation	g/6, 2% NR Leona	1.15	1.36	15.31%
34	Translation	g/6, 2% NR Leona	2.14	2.52	14.85%
45	Translation	g/6, 2% NR Leona	3.19	3.65	12.76%
56	Translation	g/6, 2% NR Leona	4.40	4.75	7.38%
67	Translation	g/6, 2% NR Leona	5.47	5.57	1.79%
	Average				11.16%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/3, 2% NR Leona	0.65	0.76	14.89%
23	Translation	g/3, 2% NR Leona	2.31	2.73	15.31%
34	Translation	g/3, 2% NR Leona	4.29	5.04	14.85%
45	Translation	g/3, 2% NR Leona	6.37	7.30	12.76%
56	Translation	g/3, 2% NR Leona	8.80	9.50	7.38%
67	Translation	g/3, 2% NR Leona	10.93	11.13	1.79%
	Average				11.16%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/2, 2% NR Leona	0.97	1.13	13.88%
23	Translation	g/2, 2% NR Leona	3.46	4.04	14.36%
34	Translation	g/2, 2% NR Leona	6.43	7.54	14.72%
45	Translation	g/2, 2% NR Leona	9.56	10.96	12.79%
56	Translation	g/2, 2% NR Leona	13.20	14.25	7.38%
67	Translation	g/2, 2% NR Leona	16.40	16.70	1.79%
	Average				10.82%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	0.6g, 2% NR Leona	1.16	1.31	11.55%
23	Translation	0.6g, 2% NR Leona	4.11	4.73	13.06%
34	Translation	0.6g, 2% NR Leona	7.69	8.87	13.32%
45	Translation	0.6g, 2% NR Leona	11.49	12.89	10.90%
56	Translation	0.6g, 2% NR Leona	15.85	16.79	5.60%
67	Translation	0.6g, 2% NR Leona	19.69	19.72	0.19%
	Average				9.10%

Percent Error of Interstory Drifts for 2% Damping for Northridge Earthquake in Leona Valley

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	0.9g, 2% NR Leona	1.73	1.72	0.69%
23	Translation	0.9g, 2% NR Leona	6.16	6.15	0.19%
34	Translation	0.9g, 2% NR Leona	11.53	11.41	0.99%
45	Translation	0.9g, 2% NR Leona	17.23	16.82	2.44%
56	Translation	0.9g, 2% NR Leona	23.78	22.69	4.76%
67	Translation	0.9g, 2% NR Leona	29.52	27.23	8.41%
Average					2.91%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	1g, 2% NR Leona	1.95	2.03	4.35%
23	Translation	1g, 2% NR Leona	6.93	7.42	6.65%
34	Translation	1g, 2% NR Leona	12.87	14.01	8.15%
45	Translation	1g, 2% NR Leona	19.11	20.59	7.15%
56	Translation	1g, 2% NR Leona	26.40	27.53	4.11%
67	Translation	1g, 2% NR Leona	32.80	32.74	0.18%
Average					5.10%

Percent Error of Interstory Drifts for 5% Damping for Northridge Earthquake in Leona Valley

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/6, 5% NR Leona	0.25	0.24	4.61%
23	Translation	g/6, 5% NR Leona	0.93	0.91	2.48%
34	Translation	g/6, 5% NR Leona	1.81	1.76	2.56%
45	Translation	g/6, 5% NR Leona	2.73	2.62	4.30%
56	Translation	g/6, 5% NR Leona	3.67	3.42	7.32%
67	Translation	g/6, 5% NR Leona	4.33	3.97	9.05%
Average					5.05%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/3, 5% NR Leona	0.51	0.49	4.59%
23	Translation	g/3, 5% NR Leona	1.86	1.82	2.47%
34	Translation	g/3, 5% NR Leona	3.62	3.53	2.56%
45	Translation	g/3, 5% NR Leona	5.47	5.24	4.29%
56	Translation	g/3, 5% NR Leona	7.34	6.84	7.31%
67	Translation	g/3, 5% NR Leona	8.66	7.94	9.04%
Average					5.04%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/2, 5% NR Leona	0.76	0.73	4.60%
23	Translation	g/2, 5% NR Leona	2.79	2.73	2.47%
34	Translation	g/2, 5% NR Leona	5.43	5.29	2.56%
45	Translation	g/2, 5% NR Leona	8.20	7.87	4.29%
56	Translation	g/2, 5% NR Leona	11.01	10.26	7.31%
67	Translation	g/2, 5% NR Leona	12.99	11.91	9.05%
Average					5.05%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	0.6g, 5% NR Leona	0.92	0.87	5.58%
23	Translation	0.6g, 5% NR Leona	3.38	3.27	3.28%
34	Translation	0.6g, 5% NR Leona	6.53	6.35	2.80%
45	Translation	0.6g, 5% NR Leona	9.84	9.44	4.24%
56	Translation	0.6g, 5% NR Leona	13.17	12.31	6.96%
67	Translation	0.6g, 5% NR Leona	15.59	14.30	9.05%
Average					5.32%

Percent Error of Interstory Drifts for 5% Damping for Northridge Earthquake in Leona Valley

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	0.9g, 5% NR Leona	1.38	1.32	4.91%
23	Translation	0.9g, 5% NR Leona	5.07	4.90	3.40%
34	Translation	0.9g, 5% NR Leona	9.79	9.36	4.59%
45	Translation	0.9g, 5% NR Leona	14.76	13.69	7.80%
56	Translation	0.9g, 5% NR Leona	19.75	17.61	12.14%
67	Translation	0.9g, 5% NR Leona	23.39	20.46	14.33%
Average					7.86%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	1g, 5% NR Leona	1.52	1.40	8.99%
23	Translation	1g, 5% NR Leona	5.59	5.14	8.71%
34	Translation	1g, 5% NR Leona	10.86	9.73	11.56%
45	Translation	1g, 5% NR Leona	16.41	14.21	15.47%
56	Translation	1g, 5% NR Leona	22.01	18.24	20.70%
67	Translation	1g, 5% NR Leona	25.99	22.49	15.57%
Average					13.50%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	1.5g, 5% NR Leona	2.3	159.4	98.55%
23	Translation	1.5g, 5% NR Leona	8.4	799.5	98.94%
34	Translation	1.5g, 5% NR Leona	16.3	4580.7	99.64%
45	Translation	1.5g, 5% NR Leona	24.6	1069.3	97.70%
56	Translation	1.5g, 5% NR Leona	32.9	9342.2	99.65%
67	Translation	1.5g, 5% NR Leona	39.0	197.2	80.24%
Average					95.79%

Percent Error of Interstory Drifts for 10% Damping for Northridge Earthquake in Leona Valley

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/6, 10% NR Leona	0.21	0.19	8.7%
23	Translation	g/6, 10% NR Leona	0.78	0.73	6.7%
34	Translation	g/6, 10% NR Leona	1.51	1.43	5.8%
45	Translation	g/6, 10% NR Leona	2.31	2.18	6.0%
56	Translation	g/6, 10% NR Leona	3.16	2.95	6.9%
67	Translation	g/6, 10% NR Leona	3.82	3.57	7.1%
	Average				6.9%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/3, 10% NR Leona	0.42	0.39	8.7%
23	Translation	g/3, 10% NR Leona	1.55	1.45	6.7%
34	Translation	g/3, 10% NR Leona	3.03	2.86	5.8%
45	Translation	g/3, 10% NR Leona	4.62	4.36	6.0%
56	Translation	g/3, 10% NR Leona	6.32	5.91	6.9%
67	Translation	g/3, 10% NR Leona	7.64	7.13	7.1%
	Average				6.9%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	g/2, 10% NR Leona	0.63	0.58	8.7%
23	Translation	g/2, 10% NR Leona	2.33	2.18	6.7%
34	Translation	g/2, 10% NR Leona	4.54	4.29	5.8%
45	Translation	g/2, 10% NR Leona	6.94	6.54	6.0%
56	Translation	g/2, 10% NR Leona	9.47	8.86	6.9%
67	Translation	g/2, 10% NR Leona	11.46	10.70	7.1%
	Average				6.9%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	0.6g, 10% NR Leona	0.77	0.69	10.40%
23	Translation	0.6g, 10% NR Leona	2.81	2.62	7.55%
34	Translation	0.6g, 10% NR Leona	5.46	5.15	5.95%
45	Translation	0.6g, 10% NR Leona	8.32	7.85	5.88%
56	Translation	0.6g, 10% NR Leona	11.35	10.64	6.69%
67	Translation	0.6g, 10% NR Leona	13.77	12.84	7.22%
	Average				7.28%

Percent Error of Interstory Drifts for 10% Damping for Northridge Earthquake in Leona Valley

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	0.9g, 10% NR Leona	1.15	1.02	12.49%
23	Translation	0.9g, 10% NR Leona	4.22	3.87	9.06%
34	Translation	0.9g, 10% NR Leona	8.19	7.66	6.95%
45	Translation	0.9g, 10% NR Leona	12.47	11.66	6.96%
56	Translation	0.9g, 10% NR Leona	17.02	15.76	7.98%
67	Translation	0.9g, 10% NR Leona	20.64	19.05	8.40%
	Average				8.64%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	1g, 10% NR Leona	1.28	1.09	17.6%
23	Translation	1g, 10% NR Leona	4.69	4.14	13.2%
34	Translation	1g, 10% NR Leona	9.10	8.27	10.1%
45	Translation	1g, 10% NR Leona	13.86	12.59	10.0%
56	Translation	1g, 10% NR Leona	18.91	16.98	11.4%
67	Translation	1g, 10% NR Leona	22.94	20.55	11.6%
	Average				12.3%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	1.5g, 10% NR Leona	1.9	1.7	9.70%
23	Translation	1.5g, 10% NR Leona	7.0	6.5	7.65%
34	Translation	1.5g, 10% NR Leona	13.6	12.9	6.05%
45	Translation	1.5g, 10% NR Leona	20.8	19.6	5.75%
56	Translation	1.5g, 10% NR Leona	28.4	26.4	7.43%
67	Translation	1.5g, 10% NR Leona	34.4	31.5	9.20%
	Average				7.63%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	2g, 10% NR Leona	2.55	181.64	98.59%
23	Translation	2g, 10% NR Leona	9.38	915.10	98.97%
34	Translation	2g, 10% NR Leona	18.20	2040.13	99.11%
45	Translation	2g, 10% NR Leona	27.72	8667.32	99.68%
56	Translation	2g, 10% NR Leona	37.83	4223.47	99.10%
67	Translation	2g, 10% NR Leona	45.88	9850.29	99.53%
	Average				99.17%

			Linear (in)	Nonlinear (in)	Percent Error
12	Translation	3g, 10% NR Leona	3.83	5.03	23.88%
23	Translation	3g, 10% NR Leona	14.07	12.99	8.33%
34	Translation	3g, 10% NR Leona	27.28	7829.27	99.65%
45	Translation	3g, 10% NR Leona	41.56	4498.85	99.08%
56	Translation	3g, 10% NR Leona	56.72	1315.19	95.69%
67	Translation	3g, 10% NR Leona	68.79	5229.93	98.68%
	Average				70.88%