

NONLINEAR ANALYSIS OF MULTISTORY STRUCTURES USING NONLIN

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ABSTRACT

This thesis presents the results of a study of the effect of variations of systemic parameters on the structural response of multistory structures subjected to Incremental Dynamic Analysis. A five-story building was used in this study. Three models were used to represent buildings located in Berkeley, CA, New York, NY, and Charleston, SC. The systemic parameters studied are post-yield stiffness, degrading stiffness and degrading strength. A set of single-record IDA curves was obtained for each systemic parameter. Two ground motions were used in this study to generate the single-record IDA curves. These ground motions were scaled to the design spectral acceleration prior to the applications. The effect of vertical acceleration was examined in this analysis. "NONLIN", a program capable of performing nonlinear dynamic analysis, was updated to perform most of the analysis in this study. The damage measure used in this study was the maximum interstory drift. Some trends were observed for the post-yield stiffness and the degrading strength. However, no trend was observed for the degrading stiffness. The change in structural response due to vertical acceleration and P-delta effect has been studied.

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Chapter 1 Introduction

1.1 Background

Building codes require that structures be designed to withstand a certain intensity of ground acceleration, with the intensity of the ground motion depending on the seismic hazard. Because of the high forces imparted to the structure by the earthquake, the structures are usually designed to have some yielding. The goal of earthquake engineering is to minimize loss of life due to the collapse of the yielding structure. However, the costs involved in replacing and rehabilitating structures damaged by the relatively moderate Loma Prieta and Northridge earthquakes have proven that the “Life-Safe” building design approaches are economically inefficient (Vamvatsikos 2002). As a result, the principle of “Performance Based Earthquake Engineering” (PBEE), which promotes the idea of designing structures with higher levels of performance standards across multiple limit states, has been proposed. In association with PBEE principles, a new analysis approach, called Incremental Dynamic Analysis (IDA), has been developed to assist the engineer in evaluating the performance of structures.

IDA was first introduced by Bertero in 1997 and a computer algorithm for implementing IDA was presented by Vamvatsikos and Cornell (Spears 2004). By using IDA, engineers not only can estimate the safety of structure under certain level of seismic loads but also ensure that the designed structure meets a designated level of serviceability.

Throughout the past century, no significant earthquake has occurred in the Central and Eastern United States (CEUS) (Spears 2004). Additionally, based on the relatively low

occurrence rate of deadly earthquakes, buildings and infrastructures in the CEUS have been designed to mainly withstand gravity and wind load only. Usually, the seismic and wind loads for structures located in the non-coastal areas is less critical than gravity, and therefore gravity loads dominate the design. Structural designs controlled by gravity are referred to as Gravity Load Design (GLD). In GLD, structures tend to have lower lateral strength and stiffness than structures designed for earthquake or wind. However, the total weight (gravity load) of buildings in the CEUS is not significantly different than the weight of the same building situated in the Western United States (WUS). Due to the relatively low lateral resistance of CEUS buildings, the influence of the geometric effect, known as P-Delta effects, are likely to be more significant in CEUS buildings than in WUS structures.

The P-Delta effects can also be affected by vertical accelerations. In particular, if the vertical accelerations are imposing maximum compressive forces in columns at the same time that the lateral displacements are approaching a maximum, dynamic instability may occur. Based on this concern, Spears (2004) conducted research on the influence of vertical accelerations on structural collapse of buildings situated in the CEUS. In his research, only simple single degree of freedom structures were analyzed. From his research, it was discovered that vertical accelerations can affect the maximum lateral displacements and in some circumstances, increase the likelihood of structural collapse.

1.2 Objective and Purpose

The purpose of this thesis is to further investigate the effect of vertical acceleration on the structural response under seismic loads. Multistory structural models with vertical flexibilities

and degrading strength and stiffness properties were used for this analysis. Incremental Dynamic Analysis was performed to determine the sensitivity of a variety of parameters to the seismic behavior.

The majority of the analysis was performed by the program NONLIN (Charney and Barngrover, 2004). NONLIN is specifically designed to perform nonlinear dynamic analysis on simplified models of structural systems. In the latest version of NONLIN (Version 7), there is a Multiple Degree of Freedom Model (MDOF) that has the ability to analyze only single-story structures. Furthermore, Version 7 cannot accommodate vertical ground accelerations. For this reason, a new analytical model was created in NONLIN to allow the analysis of multistory structures subjected to simultaneous horizontal and vertical ground motions. This new model also provides for the inclusion of degrading stiffness and strength. The first part of this thesis describes the new model, and the verification of the model. Also described in the first part of the thesis are various other enhancements that were added to NONLIN, not all of which were directly utilized in the analysis of the CEUS structures. For example, a new utility for evaluating the damped modal characteristics of structures was added to NONLIN, but was not used in the research. These utilities added to NONLIN but not directly used in the research were requested by the sponsor of the project.

Once the new version of NONLIN was available, the principal objectives of the study were to:

- Investigate the effect of vertical acceleration on the dynamic stability of structures
- Evaluate the effect of deteriorating stiffness and strength of the structural components

- Determine whether the vertical acceleration and the deteriorating inelastic structural properties should be included in the analysis

1.3 Organization of the Thesis

Chapter 2 focuses on a literature review, and explains the need for the development of NONLIN and Incremental Dynamic Analysis. The description of the revised Single Degree of Freedom (SDOF) model in NONLIN is discussed in Chapter 3. The development of the Nonproportional Damping tool and the comparison between damped mode shape and the undamped mode shape is discussed in Chapter 4. Chapter 5 presents the development of the new multistory model, and explains the theory behind the program. The verification of the multistory model is given in Chapter 6. The variation of parameter IDA of a sample 5-story structure is presented given in Chapter 7. The summary of the IDA and ideas for future research are given in Chapter 8.

Chapter 2 Literature Review

2.1 Incremental Dynamic Analysis (IDA)

To conduct the research on the influence of vertical acceleration on structures, a large number of analyses have to be run, and a tremendous amount of output has to be evaluated. Incremental Dynamic Analysis (IDA) is a systematic methodology for performing and evaluating the results of a large number of analyses.

2.1.1 History and Background of IDA

The idea of Incremental Dynamic Analysis was first introduced by Bertero in 1977. It has been further developed by many researchers, and was adopted by the Federal Emergency Management Agency (FEMA 2000a). IDA is described as the state-of-the-art method to determine global collapse capacity (Vamvatsikos 2002). By using IDA, engineers can study and understand structural response under a variety of ground motions and ground motion intensities. A good estimation of the dynamic capacity of structures can be obtained. The range of structural demands anticipated under certain level of ground motion records can also be found. By using all the data obtained from IDA, engineers can readily evaluate the adequacy of a particular design.

In general, Incremental Dynamic Analysis is a series of nonlinear dynamic analyses of a particular structure subjected to a suite of ground motions of varying intensities. The goal of IDA is to provide information on the behavior of a structure, from elastic response, to inelastic response, and finally, to collapse. (Vamvatsikos 2002). In IDA, the quantification of the response of the structure is provided by a variety of Damage Measures (DM) which correspond to

systematically increasing ground motion Intensity Measures (IM). Plots of Damage Measures versus Intensity Measures are called IDA plots.

There are two conventional types of IDA, which are Single Record IDA and Multiple Record IDA. The Single Record IDA refers to the dynamic analysis of a single structure with a single scaled ground motion. In contrast, Multiple Record IDA refers to the IDA of a single structure with multiple scaled ground motions. In addition to these two conventional types of IDA, there is another type of IDA in which the structures can have a single varying structural parameter, under a single ground motion. For example, a series of IDA plots of DM versus IM may be plotted for a single structure subjected to a single ground motion, but with each plot representing a particular initial stiffness.

As mentioned above, the ground motion has to be scaled before it can be used in IDA. There are several methods to scale the ground motions. In general, the ground motions are scaled to a base intensity measure. The base intensity measure is usually a spectral acceleration. The most common base intensity measures are peak ground acceleration, or the 5% damped spectral pseudoacceleration at the structure's first mode period of vibration.

Once the base intensity is obtained, individual response histories are run at equally spaced intervals, or Intensity Measures. For example, a single ground motion IDA may consist of response histories run at 0.05 to 2.0 times the base intensity, at 0.05 increments.

Peak result quantities, or Damage Measures, are obtained from each response history. The damage measure is the maximum response or damage to the structure due to the ground acceleration. The damage measure can be the maximum base shear, total acceleration, nodal displacement, interstory drift, damage index, etc. The selection of the damage measure depends on the component of interest. For example, to assess the nonstructural damage, the peak total acceleration can be a good choice (Vamvatsikos 2002). For damage on the structural frame, the inelastic joint rotation or rotational ductility demand can be very good options for the DM.

2.1.2 General Properties in IDA

The slope of the IDA curve is an important indicator of the structural response. On the IDA curve, there is usually a very distinct region for elastic response. In the elastic response region, the slope of the IDA curve is linear, meaning that the damage measure is directly proportional to the intensity measure in that region. When the slope becomes nonlinear, it represents the fact that the structure undergoes nonlinear behavior. An IDA plot obtained from NONLIN is shown in Figure 2.1.

There are two definitions for the capacity of the structure under IDA. The first one is the DM-based rule. Damage Measure is an indication of the damage to structures. The idea of a DM-based rule is that if the damage measure reaches certain values, the limit state will be exceeded. FEMA 350 has guidelines for the definition of DM-based limit states for immediate occupancy and global collapse. The advantages of DM-based rules are simplicity and effortlessness in implementation. DM-based rules are an especially accurate indication for the performance level

of structures. However, for determination of structural collapse, DM-base rules can be a good indicator only if the structure is modeled very precisely.

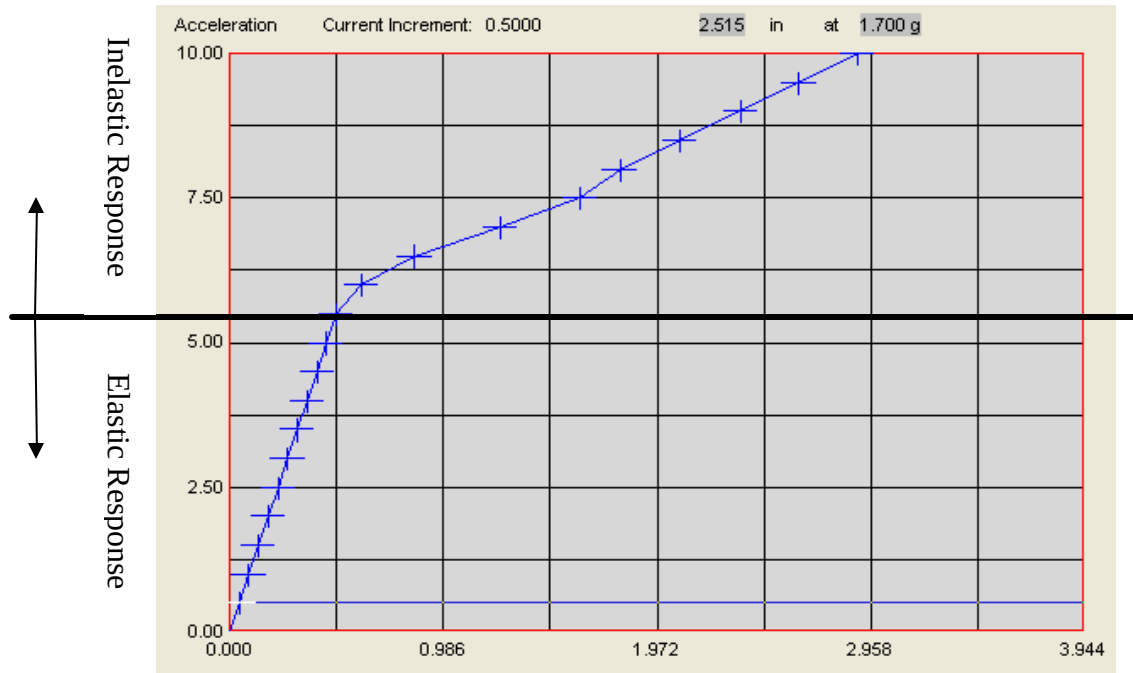


Figure 2.1 Example of IDA curve

The second limit state is an IM-based rule. The IM-based rule is a better assessment of structural collapse. In the IM-based rule, the IDA curve is divided into two regions. The upper region represents collapse and the lower region represents non-collapse. The collapse region can be clearly defined by an IM-based rule. However, the difficulty is to define the point that separates the two regions in a consistent pattern (Vamvatsikos 2002). Based on FEMA (2000a), the last point on the IDA curve with a tangent slope equal to 20% of the elastic slope is defined as the capacity point. This capacity point is used to separate the collapse and non-collapse region.

Figure 2.2 shows a sample of an IDA plot. Notice that there are certain points on the IDA curve that satisfy the limit state based on DM and a similar condition happens to the limit state based on IM. This is due to the structural resurrection (Vamvatsikos 2002). Structural resurrection means that the structural damage is decreased when the intensity of ground motion is increased. For a DM-based rule, the lowest value is conservatively used as the limit state point. For an IM-based rule, the last point of the curve with a slope equal to 20% of the elastic slope is to be used as the capacity points.

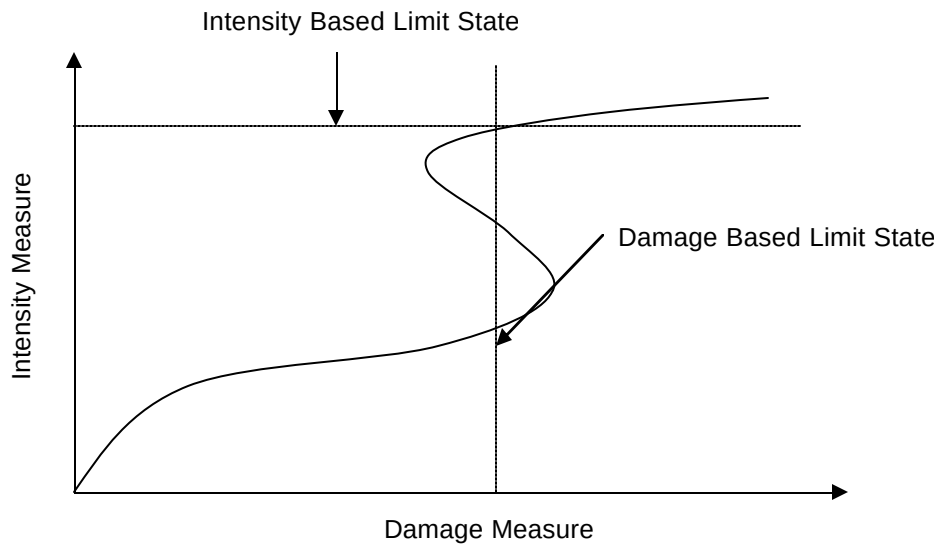


Figure 2.2 Sample of IDA plot

When the response of the structure is in the elastic range, the intensity measure will be the same for all ground motions. However, for intensity beyond the elastic range, the structural response will be different for different ground motions. The difference is called “Dispersion”. Figure 2.3 illustrates the IDA dispersion (Spears 2004). The dispersion represents the certainty of

IDA analysis. In order to assertively draw a conclusion from an IDA analysis, many earthquake ground motions are required.

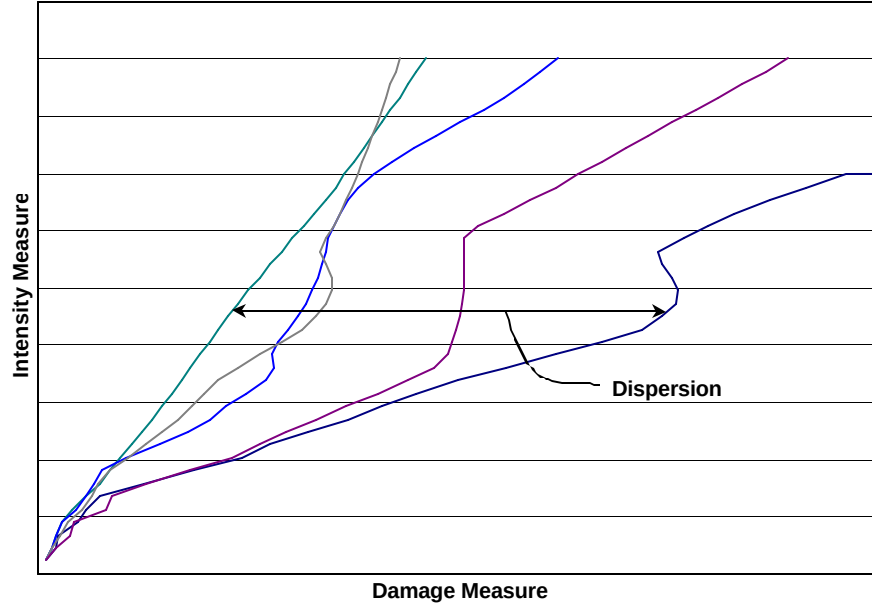


Figure 2.3 IDA Dispersion (Spears 2004)

2.1.3 Damage Index

The Damage Index (DI) is often used as a Damage Measure. Many damage indices have been developed by researchers. One of the most popular damage indices is the “Park and Ang index”. The Park and Ang index (Park et al. 1985) is developed for damage evaluation of reinforced concrete buildings. The equation for the Park and Ang Index is shown in Equation 2.1 (Spears 2004).

$$DI = \frac{|u|_{\max}}{u_{ult}} + b \frac{HE}{R_y u_{ult}} \quad (2.1)$$

where HE is the total dissipated hysteretic energy,

β is a calibration factor, taken as 0.15,

R_y is the yield force,

$|u_{max}|$ is the maximum cyclic displacement,

u_{ult} is the maximum deformation capacity under monotonically increasing lateral deformation, which can be taken as $4u_y$.

2.2 P-Delta Effect and Vertical Acceleration on Structures

The P-delta effect is an important issue in structural engineering. The lateral stiffness of a cable will be increased by a large tension force, while a large compressive force on a long rod will decrease the lateral stiffness of the rod (Wilson 2002). According to Wilson, for static analysis, the changes in displacement and member forces caused by the P-delta effect for a well designed structure should be less than 10%.

The analysis without P-delta effect is called “first order analysis”, while the analysis with P-delta effect is known as “second order analysis”. Figure 2.4 demonstrates the P-delta effect on a compression member with a moment applied at the ends of the member. M_o is the moment on the member based on the non-deformed shape. The P-delta moment refers to the additional moment generated by the deformed shape of the member.

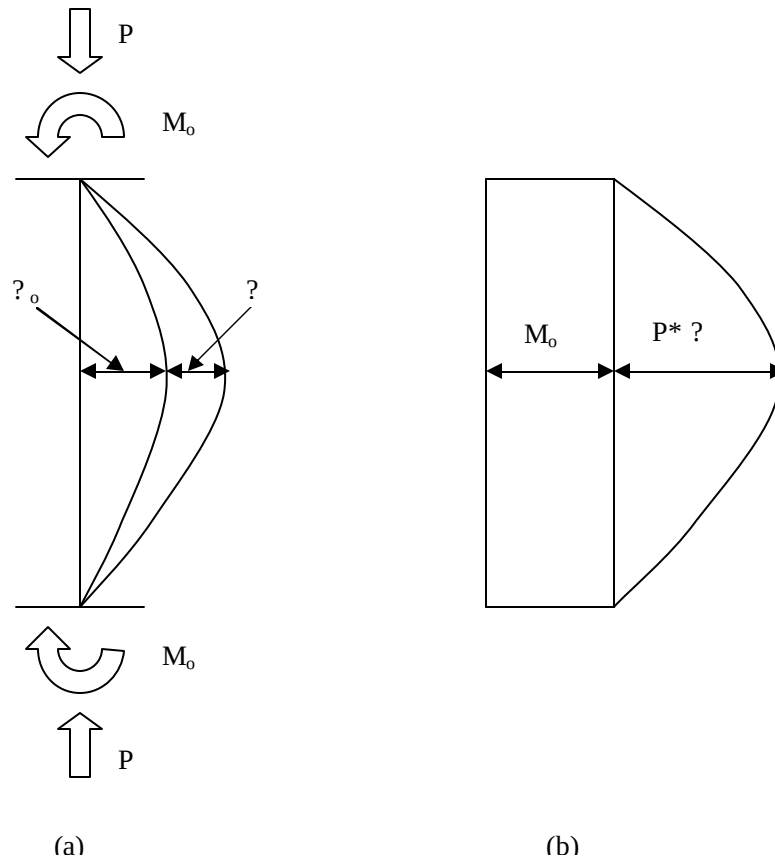


Figure 2.4 (a) Free Body Diagram of member with P-delta Effect (b) Moment Diagram of member with P-delta effect

For static analysis, the P-delta effect usually increases the lateral displacement of the structure. For dynamic analysis, the P-delta effect depends on the loading history and the original fundamental period of vibration of the structure. Depending on the ground motion, P-delta effect may result in an increase or decrease in the lateral displacement. Unlike static analysis, the P-delta effect in dynamic analysis can significantly change the response of the structures. Figure 2.5 shows the response history of the top story lateral displacement of a three-story structure subjected to a sine wave ground motion. One of the curves represents the time history of the

response of the structure without considering the P-delta effect, and the other curve represents the structural response with P-delta effects considered in the analysis. When the response of structure is in the elastic range, the P-delta effect is usually small (Bernal 1987). However, for structural response beyond the elastic limit, the P-delta effect becomes significant. Present earthquake engineering philosophy allows structures to yield under the design level of ground acceleration; therefore it is necessary to include the P-delta effect in the analysis.

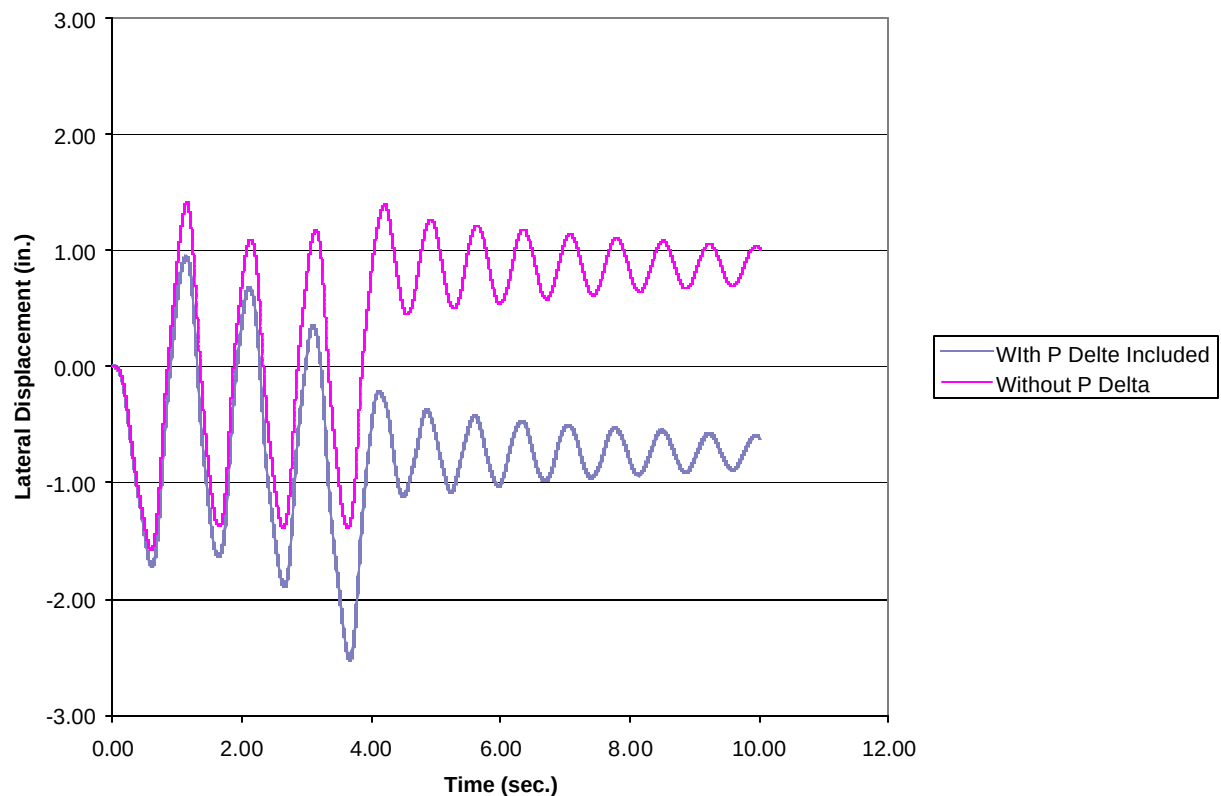


Figure 2.5 P-Delta Effect on Structure Responses

The P-delta effect can be accounted by reducing the lateral stiffness of the structures. The reduction of stiffness is called geometric stiffness. The equation of geometric stiffness (K_g) is

shown in Equation 2.2. In Equation 2.2, P is the axial force on the compression member and h is the member height. In general, the axial force on the column is proportional to the weight of the structure. The effective stiffness (K_e) is shown in Equation 2.3.

$$K_g = \frac{P}{h} \quad (2.2)$$

$$K_e = K - K_g \quad (2.3)$$

2.3 Vertical acceleration due to ground acceleration

Vertical accelerations are usually not explicitly considered in seismic analyses. Before the 1994 Northridge Earthquake, the peak vertical accelerations obtained from ground motion attenuation relationships underestimated the true magnitude of the vertical accelerations. In the Northridge Earthquake, which occurred in January, 1994, the vertical-to-horizontal peak acceleration ratio (V/H) recorded was much higher than the expected ratio based on the attenuation relationships (Lew and Hundson 1999). The V/H ratio depends on the distance from the source to the site being considered. It means that when the site is far away from the epicenter, the magnitude of the vertical acceleration is relatively small compared with the horizontal motion. The main reason for the underestimation of the V/H ratio was that the attenuation relationship used was based on the regression of the entire range of epicentral distances and magnitudes (Papazoglou and Elnashai 1996).

High peak vertical accelerations were recorded in many recent earthquakes. In the 1994 Northridge earthquake, the peak vertical accelerations recorded were as high as 1.18g and the V/H ratio was 1.79 (Papazoglou and Elnashai 1996). In the 1986 Kalamata earthquake in Greece, items were found to be displaced horizontally without any evidence of friction at the interface in the earthquake station (Papazoglou and Elnashai 1996). This means that the vertical acceleration was as high as gravity.

Field evidence shows that vertical accelerations can cause compression failures in columns. In the Northridge Earthquake, interior columns of a moment resisting frame parking garage failed in direct compression (Papazoglou and Elnashai 1996). The failure caused the total collapse of the parking structure.

Vertical acceleration also caused columns to fail in combined shear and compression. For example, the Holiday Inn Hotel located 7 km from the epicenter experienced shear split failure on the exterior columns in the 1994 Northridge Earthquake. This indicates that vertical accelerations can indirectly cause failure to the structures (Papazoglou and Elnashai 1996).

Dynamic amplification of vertical accelerations can be very high. Vertical natural frequencies of structures are usually very high because columns are much stiffer in the axial direction than the transverse direction. Papazoglou analyzed the effect of the fundamental vertical natural period of vibration on a 3-bay 8-story coupled wall-frame reinforced concrete structure and found the period to be 0.075s. Usually, the predominant periods for near field

vertical ground motion are between 0.05 s to 0.15s. This implies that large amplification on vertical acceleration is expected for strong near field ground motion.

2.4 Motivation of Research

Many researchers have conducted research using IDA analysis. De (2004) studied the influence of the effect of the variation of the systemic parameters on the structural response of single degree of freedom systems. In his study, several conclusions were made:

1. Increasing the stiffness often resulted in lower peak displacement. But for the inelastic region, the peak displacement did not have the same pattern.
2. Damping in general reduced the maximum response.
3. Geometric stiffness generally increased the peak response.

Spears (2004) conducted a study on the influence of vertical acceleration on a SDOF model with bilinear behavior. The results he obtained have shown that the lateral displacements were influenced most at the point just before collapse. In general, he concluded that vertical accelerations may or may not influence the lateral displacements of the structures. Therefore, he recommended that vertical acceleration be included in the analysis, based on conservative reasons.

However, there were some limitations in both De's and Spears' studies. In both studies, only a single degree of freedom structure was used. Usually, the first mode dominates the response in most structures. However, in some structures, the higher mode response may play an

important factor. Therefore, it is important to include the higher modes to estimate the true response of the structure.

In addition, the degrading strength and degrading stiffness characteristics of most structural elements were not applied in De's or Spears' analyses. Degrading strength and degrading stiffness can completely change the response of the structure. When degrading properties are included, it is possible that the structure will degrade to the predominant periods of the ground motion and cause resonance. Therefore, the findings they obtained may not represent the behavior of realistic structures. For example, if the natural period of a structure is 0.7 sec and the predominant period of a ground motion is 1 sec, degradation of stiffness may change the natural period of the structure to a higher value which gives a larger response than a non-degraded structure.

Moreover, in Spears' study, the amplification of the vertical acceleration on the structure could not be included because only SDOF models were used. However, researchers have found large amplification on the axial force on columns of a multistory structure. It was found that the upper floors' accelerations can be several times higher than in the lower stories (Bozorgnia et al. 1998).

Based on the limitations of the previous research, it is prudent to conduct a study using Incremental Dynamic Analysis for a structure that has multiple stories with degrading strength and degrading stiffness and with the vertical accelerations included in the analysis.

Chapter 3 Description of NONLIN Version 8

3.1 Introduction

As mentioned previously, the research conducted for this thesis relies heavily on NONLIN. Therefore, it is necessary to describe this program. NONLIN, initially created by Charney and Barngrover (2004), is a program designed to perform simple nonlinear dynamic analysis. The purpose of the development of NONLIN was to provide a tool to facilitate the understanding of the fundamentals concepts of earthquake engineering. NONLIN version 8.0 was developed as an update of NONLIN version 7.0. The objective of the update is to further develop the program by providing several new advanced features, and by modifying certain existing portions of the program to be more user-friendly. In NONLIN Version 8, there are five models in the program:

1. Single Degree of Freedom (SDOF) Model
2. Multiple Degree of Freedom (MDOF) Model
3. Dynamic Response Tool (DRT)
4. Complex Mode Response Tool (CRT)
5. Multistory Model.

The Single Degree of Freedom Model and the Dynamic Response Tool, which existed in Version 7, were extensively modified. The Complex Mode Tool and the Multistory Model are newly developed for NONLIN Version 8. The Multiple Degree of Freedom Model, present in Version 7, has not been modified for version 8 of the program.

The description of the updated SDOF and DRT are given in this chapter. The CRT and Multistory Model are described in Chapters 4 and 5, respectively.

3.2 Single Degree of Freedom (SDOF) model

The SDOF routines provide nonlinear dynamic analysis for single degree of freedom structural systems. The updates have improved the numerical integration techniques, and modifications have been done on the solver to handle more advanced hysteretic properties. The updates will ultimately be used in the Incremental Dynamic Analysis (IDA) routines. There are three major updates for the SDOF model, which are the addition of unsymmetrical structural properties, provision for hysteretic models of deteriorating inelastic behavior, and systemic parameter variation in Incremental Dynamic Analysis.

3.2.1 Unsymmetrical Structural Properties

The original SDOF model can handle fully elastic, elastic-plastic, and yielding systems with an arbitrary level of secondary stiffness; however, there are some limitations. The original SDOF model can only handle structures with symmetric structural properties, which have equal positive and negative yield strengths and equal initial and secondary stiffness. However, not all single degree of freedom structures have symmetric structural properties. For example, a non-symmetric reinforced column may have more reinforcing bars on one side than the other. Therefore, it is essential to update the SDOF model to have the ability to analyze structures with unsymmetrical structural properties.

The newly modified SDOF model has the ability to handle structures with unsymmetrical properties. Users are required to input the positive yield strength, negative yield strength, elastic stiffness, positive secondary stiffness, and negative secondary stiffness for NONLIN to perform the nonlinear analysis. The force-deformation relationship of the unsymmetrical structural properties is illustrated in Figure 3.1, and the system properties input for the SDOF model is shown in Figure 3.2.

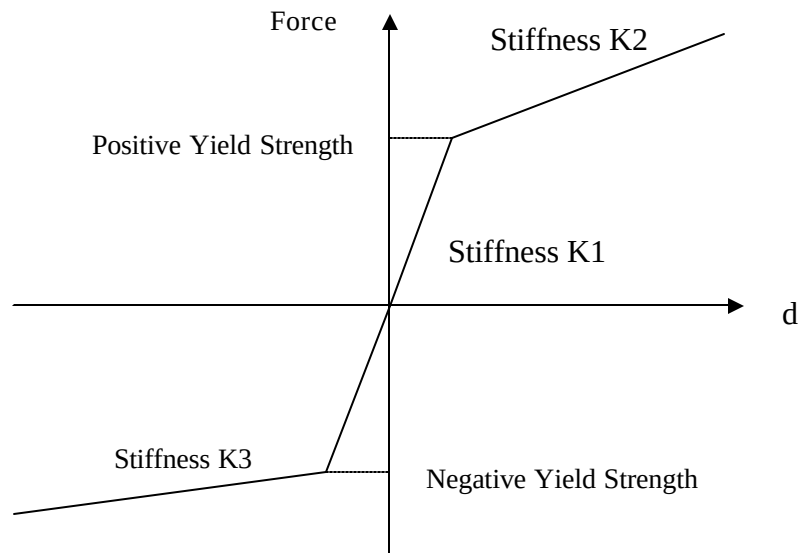


Figure 3.1 Unsymmetrical Hysteretic Model in SDOF Model

The image shows a 'Properties' dialog box with a dropdown menu set to 'Non Degrading'. It contains several input fields for material properties, each preceded by a small icon representing a force-displacement curve. The fields are: STIFFNESS K1, STIFFNESS K2, STIFFNESS K3, STRENGTH POS., STRENGTH NEG., and STIFFNESS KG. Below these fields are three radio buttons: 'LINEAR ANALYSIS' (unselected), 'NONLINEAR ANALYSIS' (selected), and 'INCLUDE P-DELTA' (unchecked).

Figure 3.2 Input Table for Yield Strengths and Stiffness

By inputting different values for the secondary stiffness and yield strength in the input table in Figure 3.2, the unsymmetrical structural properties can be modeled. Figure 3.3 and Figure 3.4 show two examples of force-displacement curves obtained from the newly modified NONLIN program.

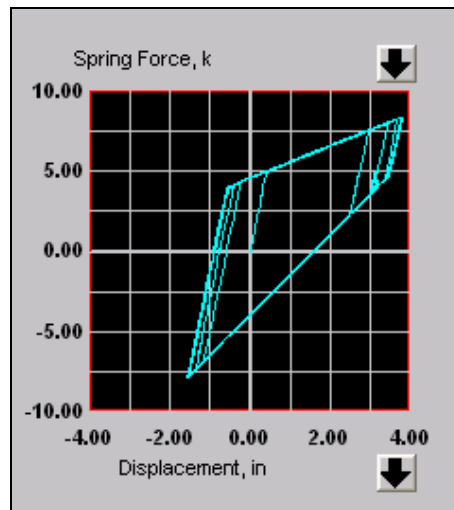


Figure 3.3 Force-Displacement Curve of a Structure With Unsymmetrical Secondary Stiffness

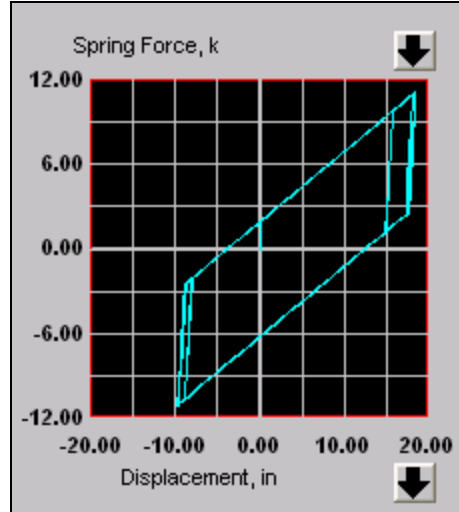


Figure 3.4 Force-Displacement Curve of a Structure With Unsymmetrical Yield Strength

3.2.2 Degrading Structural Properties for SDOF model

The cost to design earthquake resistant structures to remain elastic is much higher than inelastic design. Hence, structures are designed to yield under strong ground motion. For strong and long duration ground motions, structures usually undergo numerous cycles of deformation. When the deformation is beyond the yielding limit, deterioration in stiffness and strength is expected.

3.2.2.1 Hysteretic Models for Deteriorating Inelastic Structures

Yielding can cause degradation in stiffness and strength of a structure. The changes in stiffness and strength can cause an increase in the lateral displacement of the structure and increase the chance of structural collapse. The inelastic behavior of degradation in stiffness and strength can be modeled by the hysteretic models developed by Sivaelvan and Reinhorn (1999). Sivaelvan and Reinborn developed two types of deteriorating hysteretic behavior, which are the

polygonal hysteretic model (PHM) and the smooth hysteretic model (SHM). The deteriorating nonlinear behavior used in the SDOF model of NONLIN is the polygonal hysteretic model. The PHM is chosen because of the simplicity in handling the various parameters, including initial stiffness, cracking, yielding, stiffness and strength degrading, and crack and gap closures.. The polygonal hysteretic model follows “Points” and “Branches” which govern the various stages and the transitions of the elements. The backbone curve of the PHM is the same as the bilinear model.

The elastic stiffness is reduced when the inelastic displacement increases. The pivot rule was found to be an accurate model of the stiffness degradation (Park et al. 1987). The pivot rule assumes that during the load-reversal, the reloading stiffness is targeted to a pivot point on the elastic branch at a distance on the opposite side. The illustration of the stiffness degradation is presented in Figure 3.5. The stiffness degradation terms (R_k) are obtained from the geometrical relationship in Figure 3.5 and is shown in Equation 3.1 (Sivaselvan and Reinhorn, 1999). The elastic stiffness after yielding is given in Equation 3.2.

$$R_K^+ = \frac{M_{cur} + aM_y}{K_0 f_{cur} + aM_y} \quad (3.1)$$

where M_{cur} is the current moment;

f_{cur} is the current curvature;

K_0 is the initial elastic stiffness;

a is the stiffness degradation parameter;

M_y is the positive or negative yield moment.

$$K_{cur} = R_k K_0 \quad (3.2)$$

where R_k is the stiffness reduction factor

K_0 is the initial elastic stiffness

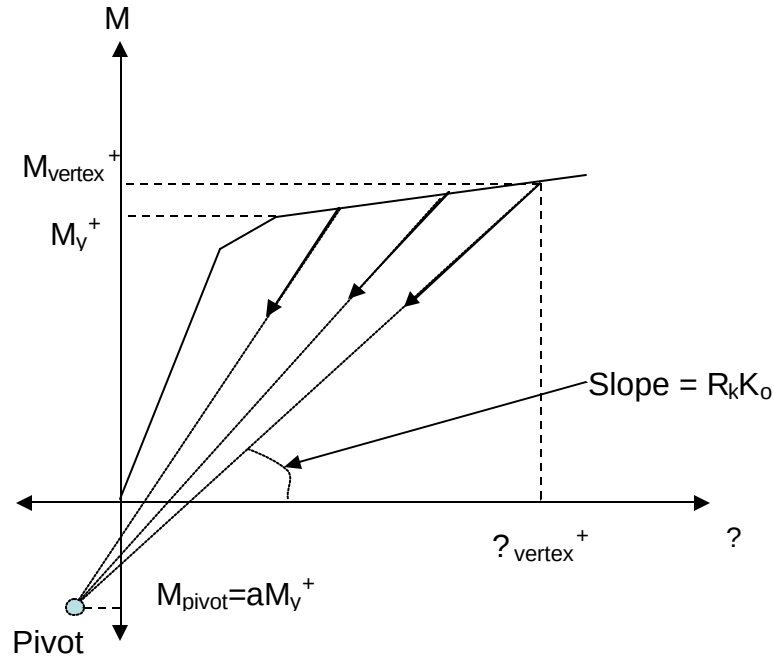


Figure 3.5 Modeling of Stiffness Degradation (Sivaselvan and Reinhorn, 1999)

The schematic diagram of the strength degradation model is given in Figure 3.6 (Sivaselvan and Reinhorn, 1999). The strength of the elements is reduced in each cycle of yielding. The rule for strength degradation is given in Equation 3.3 (Sivaselvan and Reinhorn, 1999).

$$M_y^{+/-} = M_{y0}^{+/-} \left[1 - \left(\frac{f_{\max}^{+/-}}{f_u^{+/-}} \right)^{\frac{1}{\beta_1}} \right] \left[1 - \frac{b_2}{1 - b_2} \cdot \frac{H}{H_{ult}} \right] \quad (3.3)$$

where $M_y^{+/-}$ represents the degraded positive or negative yield moment;

$M_{y0}^{+/-}$ is the initial positive or negative yield moment;

$f_{\max}^{+/-}$ is the maximum positive and negative curvature;

$f_u^{+/-}$ is the ultimate positive and negative curvature;

H is the hysteretic energy dissipated;

H_{ult} is the hysteretic energy dissipated when loaded monotonically to the ultimate curvature without any degradation;

β_1 is the ductility-based strength degradation parameter;

β_2 is the energy based strength degradation parameter.

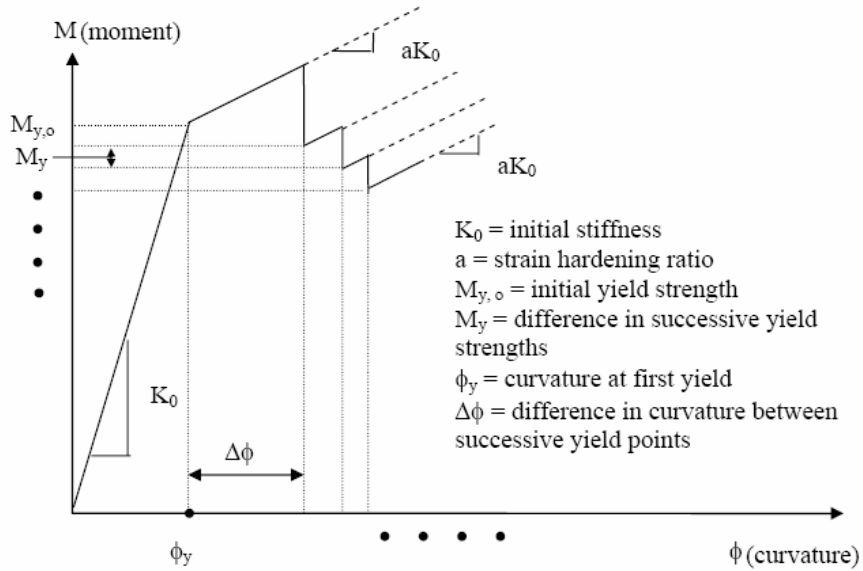


Figure 3.6 Schematic representation of strength degradation (Sivaselvan and Reinhorn, 1999)

3.2.2.2 Degrading Model in NONLIN

The Polygonal Hysteretic Model is used as the deteriorating hysteretic inelastic behavior in NONLIN. Figure 3.7 shows the input table for the parameters of the hysteretic model. The default input for the degrading parameters does not have any significant degrading properties. The range of variable “Alpha” is from 1 to 300. The range of “Beta 1” and “Beta 2” are from 0 to 1. The input for the “Positive Ductility” and “Negative Ductility” cannot be less than 1. The effect of stiffness degradation can be minimized if “Alpha” is input as a higher number. The effect of strength degradation can be minimized when “Beta 1” and “Beta 2” are small and the “Positive Ductility” and “Negative Ductility” are high.

Hysteresis	
INITIAL STIFFNESS K1	500.00
SECONDARY STIFFNESS K2	5.00
SECONDARY STIFFNESS K3	5.00
POSITIVE YIELD STRENGTH	100.00
NEGATIVE YIELD STRENGTH	100.00

Common Parameters for Multilinear and Smooth Models	
Pos. Ultimate Ductility	30.00
Neg. Ultimate Ductility	30.00
Alpha	20.00
Beta-1	0.01
Beta-2	0.01

Multilinear Model	
GAMMA	0.10
Bilinear Type: ☒ Bilinear ☐ Pinching ☐ Vertex	

Figure 3.7 Input Table for the Deteriorating Inelastic Behavior

When appropriate values are input, the true inelastic behavior can be modeled. Figure 3.8 shows the force-displacement curve of a structure with high degradation in stiffness under cyclic load, obtained from the new SDOF model of NONLIN program. Figure 3.9 shows the force-displacement curve of a structure with high strength degradation under cyclic load, obtained from the new SDOF model of NONLIN.

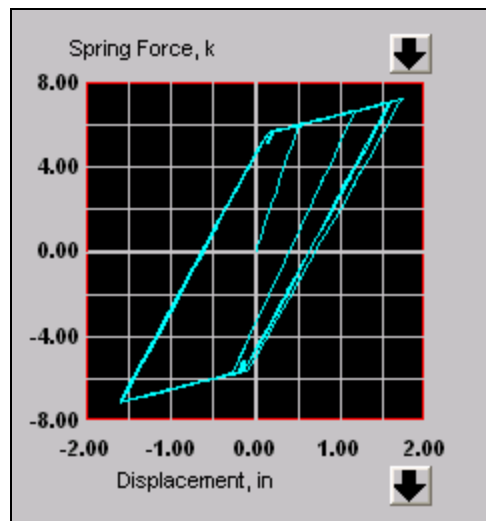


Figure 3.8 Force-displacement curve of a structure with high stiffness degradation

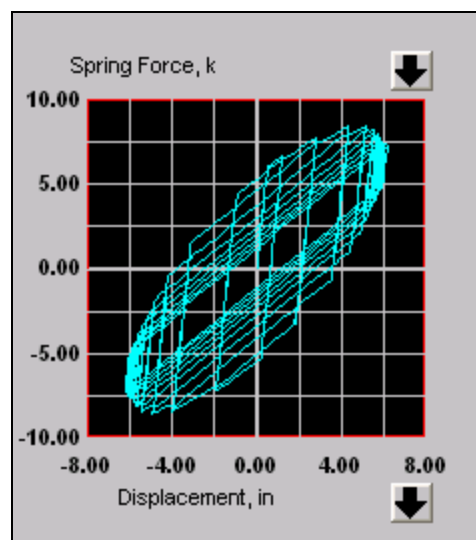


Figure 3.9 Force-displacement curve of a structure with high strength degradation

3.2.3 IDA Tool of the SDOF model

NONLIN allows for almost automatic Incremental Dynamic Analysis of single degree of freedom structures. It has been updated to handle the unsymmetrical and the hysteretic deteriorating inelastic behavior as discussed in section 3.2.1 and section 3.2.2. Another update is the creation of a new type of IDA method which allows for incremental variation of structural properties. The new type of IDA is called “Multiple Structural Parameter IDA”. This is a very useful tool to evaluate the sensitivity of a damage measure to a small change in systemic properties.

In the new IDA tool, there are five parameters that can be varied, which are mass, damping, elastic stiffness, geometric stiffness, and yield strength. Figure 3.10 shows the input table for the variation parameters. “% of Variation” is the percentage of variation of the assigned parameter. “Number of increments” is the number of increments used in the IDA. Figure 3.11 shows an example of an IDA curve with variation in stiffness.

The screenshot shows a software dialog box titled "IDA Method: Multiple Structural Parameters". It features a list of "Available EarthQuakes" on the left, including files like 49OLY.ACC, 49OLY1.ACC, and IMPVAL1.ACC. A "Selected EarthQuakes" table is on the right, with columns for Earthquakes, Factor, PGA, and Plot?. Below this, a "Parameters" section has a dropdown menu set to "Stiffness", with input fields for "% of Variation" and "Number of Increments". At the bottom, there are buttons for "Auto Scale", "Run", "Update Plot", "Clear List", "RS Plot", and "Close".

Earthquakes	Factor	PGA	Plot?
			☐

Parameters: **Stiffness**

% of Variation:

Number of Increments:

Figure 3.10 Input Table for the Multiple Structural Parameter

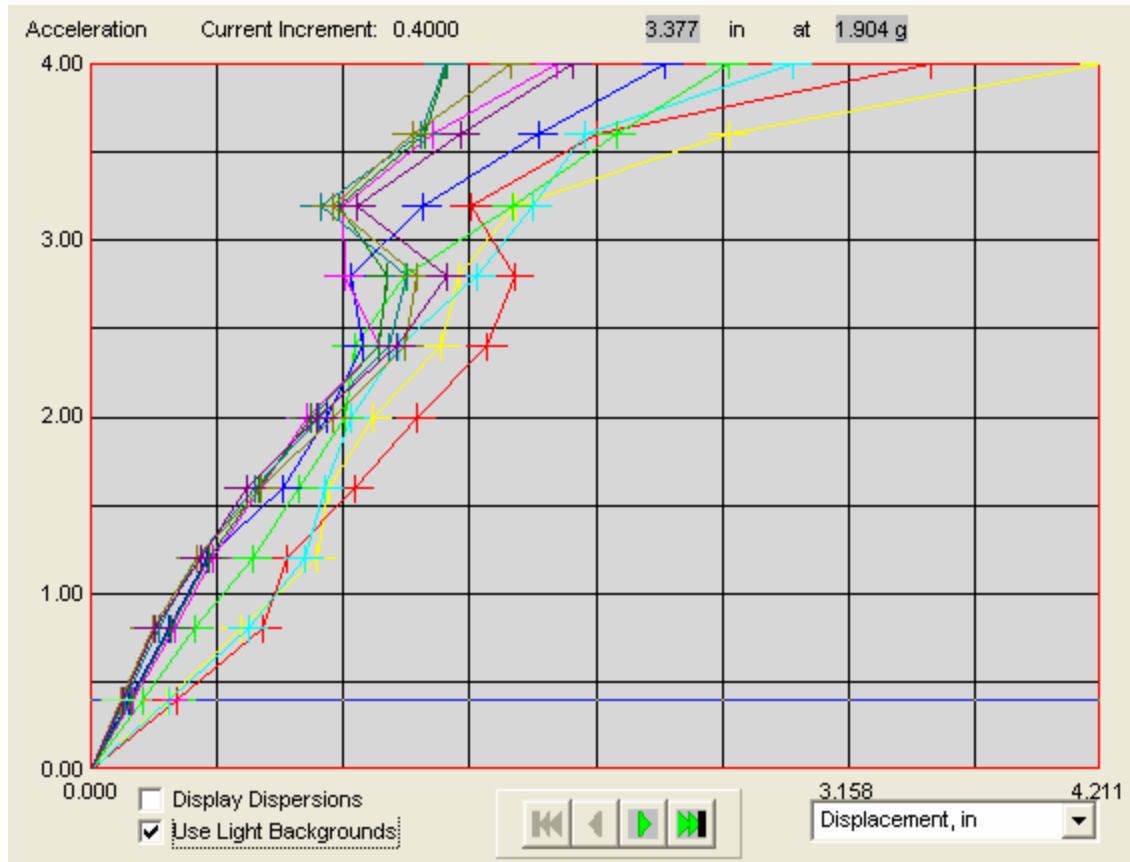


Figure 3.11 Example of IDA Plot with Variation in Primary Stiffness

3.4 Dynamic Response Tool

The Dynamic Response Tool (DRT) is a utility to illustrate the fundamental concepts of structural dynamics in real time. This illustration is carried out with a multistory shear frame subject to sinusoidal ground excitation. Both the properties of the shear frame and the ground motion may be altered by the user to see how such parameters affect the dynamic response. The purpose of the update of the DRT is to provide a more efficient tool for users to obtain and visualize the dynamic properties.

To improve the DRT to become a more efficient tool and to help the user to save calculation time, the following items have been added:

1. Mode Shape Normalization Options

Two normalization options have been added to the new DRT tool. The normalization options are unity top story displacement, and $f_n^T \cdot M \cdot f_n = 1$. The normalization options can be found on the left hand side of the window in Figure 3.1.

2. Calculation of Modal Properties

The new DRT calculates modal participation factors (MDF), effective mass, cumulative effective mass, and cumulative % of effective mass automatically. The new DRT tool also has the option to show and animate all the calculated mode shapes of the structure.

The modal properties table obtained from DRT is shown in Figure 3.12. In addition, the animation of the structural response was modified to become a smooth cubic curve rather than the straight line curve implemented in Version 7. A snapshot of the mode shape animation can be found in Figure 3.13.

Mode	1.000	2.000	3.000
Period (sec)	0.998	0.356	0.247
Freq (Hz)	1.002	2.807	4.056
Freq (rad/sec)	6.294	17.635	25.483
M.P.F.	1.220	-0.280	0.060
Eff.Mass.	6.856	0.562	0.083
Cum. Eff. Mass	6.856	7.417	7.500
% of Eff. Mass	91.408	98.896	100.000

Figure 3.12 Modal Properties obtained from Dynamic Response Tool

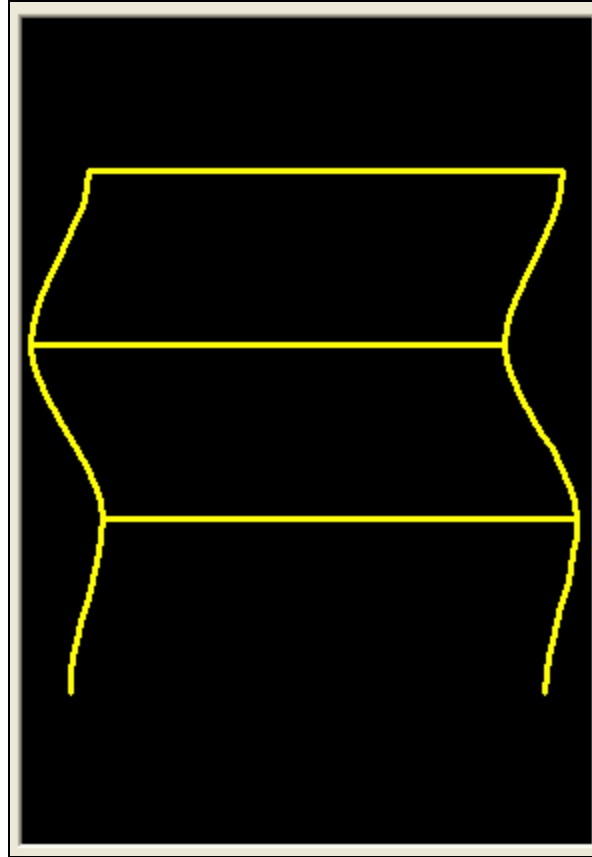


Figure 3.13 Mode Shape Animation Obtained From DRT

In NONLIN version 7, there is a Fast Fourier Transformation (FFT) plot in the Dynamic Response Tool. In the older version, the amplitude of the forcing frequency was normalized to the maximum forcing amplitude. This may cause confusion in visualizing the forcing magnitude. Therefore, in NONLIN version 8, the normalization of the FFT plot has been removed and replaced by a zoom option that provides the user the option to change the view of the FFT plot. The new FFT plot is shown in Figure 3.14.

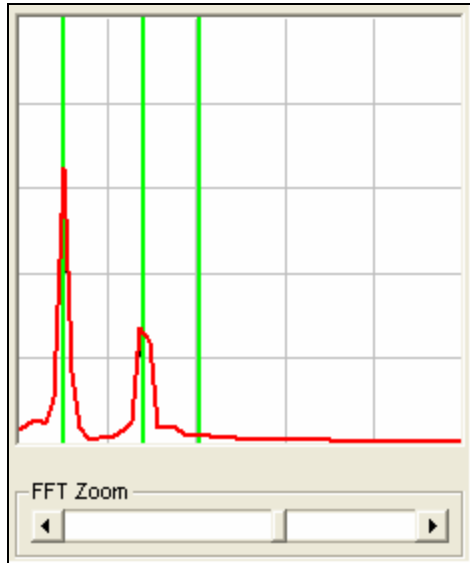


Figure 3.14 FFT Plot in NONLIN Version 8

Chapter 4 Damping in Structure

4.1 Damping in Structure

In structural dynamics, there are three important properties of structures. They are mass, damping, and stiffness of the structure. Damping can be classified as natural damping and added damping. Natural damping is the damping inherent in the structure, while added damping refers to the damping that is added to the structure by either an active or a passive device.

4.1.1 Natural Damping

Natural damping can be determined by performing a free vibration analysis. For single degree of freedom (SDOF) structures, free vibration tests can be performed to find out the damping ratio (z_N). For example, a free vibration analysis can be done to calculate the damping ratio of a cantilever. Note that Equation 4.1 assumes small damping ratios. For damping that is not small (greater than 10%), the damping ratio shall be found by using Equation 4.2. Equation 4.1 is developed based on $\sqrt{1 - z_N^2} \approx 1$.

$$z_N = \frac{1}{2pJ} \ln \frac{a_i}{a_{i+j}} \quad (4.1)$$

$$\frac{a_i}{a_{i+1}} = \exp \left(\frac{2 \cdot p \cdot z_N}{\sqrt{1 - z_N^2}} \right) \quad (4.2)$$

where a_i is the acceleration at peak i;

a_{i+j} is the acceleration at peak i + j;

a_{i+1} is the acceleration at peak i + 1.

The damping constant (C_N), which is used in the numerical analysis, is equal to a function of damping ratio, mass, and stiffness of the structure as presented in Equation 4.3. It is important to note that C_N is just a mathematical representation of some assumed damping ratio. The actual structure does not have a dashpot as represented by C_N .

$$C_N = 2 \cdot z_N \cdot \sqrt{m \cdot k} \quad (4.3)$$

For Multiple Degree of Freedom (MDOF) structures, it is more difficult to find the natural damping constant of the structure, although free vibration analysis can be done to obtain the actual damping constant. For a structure that has not been built, however, it is impossible to obtain the damping constant. Therefore, the damping ratio (z_N) is usually estimated based on data from similar structures.

4.1.2 Added Damping

Dampers are sometimes added to a structure to increase the damping. Increase in damping can usually reduce the displacement in the structure and therefore reduce the damage in the structure. The damping added (C_A) by the damper is not related to the structural properties of the original structure (Charney 2005). The damping coefficients (z_A) for the damper are usually obtained by laboratory testing.

4.2 Damping Matrix in Multiple Degree of Freedom Structure

To analyze structures that have multiple degrees of freedom, the mass, stiffness, and damping matrices have to be formed. The mass and stiffness matrices can be diagonalized using the undamped mode shapes of the structure. However, for the damping matrix, it may or may not be diagonalized by the mode shape.

For structures that have no added damping, there are two distinct ways to calculate the response of the structure. The first option is to decouple the equations of motion using the undamped mode shapes, and then simply assign a modal damping ratio to each uncoupled equation.

The second way is to form the damping matrix as a linear combination of the mass and stiffness matrices. This ensures that the damping matrix can be diagonalized by the mode shape (because the mass and stiffness are diagonalized). This type of damping is called Rayleigh Proportional damping. Any structure that has a damping matrix that can be diagonalized by the undamped mode shapes is said to have classical damping. Rayleigh Proportional Damping is by definition classical.

In Rayleigh Damping, the damping matrix (C) is equal to the sum of the product of mass matrix (M) and the constant (a) and the product of stiffness matrix (K) and the constant (β).

$$C = a \cdot M + b \cdot K \quad (4.4)$$

To calculate the mass proportional constant (α) and the stiffness proportional constant (β), the damping ratios of two modes have to be known. As discussed before, damping for a MDOF structure is very difficult to determine and is not related to the structural properties.

Once the damping ratios are known, the constants α and β can be found using the matrix relationship in Equation 4.5 as presented by Clough and Penzien (1993).

$$\begin{Bmatrix} z_i \\ z_j \end{Bmatrix} = \frac{1}{2} \cdot \begin{bmatrix} \frac{1}{w_i} & w_i \\ \frac{1}{w_j} & w_j \end{bmatrix} \cdot \begin{bmatrix} a \\ b \end{bmatrix} \quad (4.5)$$

The damping ratio for a mode other than the i^{th} and j^{th} mode can be found by Equation 4.6.

$$z_n = \frac{1}{2w_n} a + \frac{w_n}{2} b \quad (4.6)$$

For structures that have added damping, it is not likely to be able to diagonalize the damping matrix by the mode shapes of the structures. For example, a viscous elastic damper may only be added to one story of a structure and, therefore the damping will not be proportional to the mass and stiffness matrices. When structures cannot be diagonalized by their mode shapes, they are said to have non-classical damping. For these situations, the damping matrices are formed by direct assembly, similar to the stiffness matrices.

4.3 Mode Shapes of the Structure

Mode shapes of a structure are very important to MDOF structural dynamics because they are essential to perform modal analysis. Modal participation factors and effective modal mass are calculated from the mode shapes of the structures. Then the number of modes required for the analysis will be determined. After that, modal analysis will be performed.

4.3.1 Undamped Mode Shapes of the Structure

To find out the mode shapes of a structure that has no damping, the mass and stiffness matrices have to be formed. The characteristic equation is formed and the eigenvectors of the characteristic equation are the mode shapes of the structure.

$$\det \left[k - \omega_n^2 \cdot m \right] = 0 \quad (4.7)$$

As discussed in the previous section, the undamped mode shapes can be used to decouple the equations of motion for structures that have proportional damping. However, for structures that have non-proportional damping, another approach has to be used.

4.3.2 Damped Mode Shapes of the Structure

When damping cannot be decoupled, the undamped mode shapes cannot truly represent the mode shapes of the structure. To obtain the damped mode shapes, the state space matrix has to be formed (Lang and Lee 1991). The state space matrix is presented in Equation 4.8.

$$H = \begin{bmatrix} -M^{-1} \cdot C & -M^{-1} \cdot K \\ I & 0 \end{bmatrix} \quad (4.8)$$

where M is the mass matrix;

C is the damping matrix;

K is the stiffness matrix;

I is the identity matrix;

0 is the zero matrix

The size of the state space matrix is 2 times the number of degrees of freedom of the structure. When all modes are underdamped, the eigenvalues of the state space matrix will occur in complex conjugate pairs. The complex eigenvalues (l_D) are given by in Equation 4.9. The real parts of the eigenvalues are negative, which represents the decay of the motion. Equation 4.10 shows the simplified version of Equations 4.10.

$$l_D = -z_D \cdot w_D \pm i\sqrt{1-z_D^2} w_D \quad (4.9)$$

where z_D is the damping ratio;

w_D is the damped frequency.

$$l_D = A \pm iB \quad (4.10)$$

The damped frequency and the damping ratio can be found in Equations 4.11 and 4.12, respectively.

$$w_D = \sqrt{A^2 + B^2} \quad (4.11)$$

$$z_D = \frac{-A}{\sqrt{A^2 + B^2}} \quad (4.12)$$

It is important to note that the term $i\sqrt{1-z_D^2}$ becomes real when the mode is overdamped, which makes Equation 4.12 not applicable

For structures that have no damping, all the coordinates in each mode will be in phase or 180 degrees out of phase. However, for structures that have non-proportional damping, the different modal coordinates will have a variety of phase relationships. To visualize the phase relationship of each degree of freedom, a complex plane plot can be employed.

4.4 Complex Mode Tool in NONLIN

To illustrate the difference between the responses of a multistory structure with a damped mode shape and an undamped mode shape, the Complex-Mode Response Tool (CRT) is created.

In the DRT tool, a previously developed model in NONLIN, a Multi-Degree-of-Freedoms (MDOF) structure is analyzed by using the undamped mode shapes. The equations of motion are first decoupled, and then assigned a specific damping ratio to each modal equation (Charney 2005).

In the newly developed CRT tool, rather than using the traditional method, a more complicated method is used to calculate the mode shape. In the CRT tool, users are required to input the stiffness, mass, and damping constant for each level of the structure. By inputting those values, the CRT tool forms the mass, stiffness, and damping matrices. After that, the state space matrix is formed. The eigenvalues of the state space matrix are found internally, followed by the eigenvectors. Then, the complex mode shape, magnitude and phase of each degree of freedom, are calculated and presented in a table in the CRT output table.

4.4.1 Input for CRT

The number of stories and the mass, stiffness and damping for each story are required to calculate the complex mode shape of the multistory model. Figure 4.1 depicts the CRT input windows in NONLIN.

Fundamental Properties			
Number of Stories: 3			
Story	K	M	C
3	200.00	1.00	10
2	300.00	2.00	20
1	400.00	3.00	50

Figure 4.1 System Properties Input for CRT tool in NONLIN

4.4.2 Result for CRT

As mentioned before, for a proportionally damped structure, there is no difference between the damped and undamped mode shapes. However, for a structure that has non-proportional damping, the damped and undamped mode shapes will be different. In the result

table of CRT, the damped properties and undamped properties are utilized as shown in Figure 4.2. Note that the values below are based on the numbers shown in Figure 4.1.

Undamped Properties			
Mode	1.000	2.000	3.000
Period (sec)	0.954	0.424	0.306
Freq (Hz)	1.048	2.357	3.264
Freq (rad/sec)	6.585	14.808	20.511
% of Damping	32.518	59.061	64.744
Damped Properties			
Mode	1.000	2.000	3.000
Period (sec)	0.929	0.378	0.353
Freq (Hz)	1.077	2.643	2.833
Freq (rad/sec)	6.765	16.607	17.803
% of Damping	31.814	55.577	71.814

Figure 4.2 Output table for the damped and undamped properties

The phase relationship of each degree of freedom in each mode shape can be seen by plotting the coordinates of the eigenvectors (mode shape) in the complex plane. The complex plane plot is integrated in CRT. When the motion of a story is in-phase with another story, the complex plot will align together. Figure 4.3 demonstrates the complex plot in CRT.

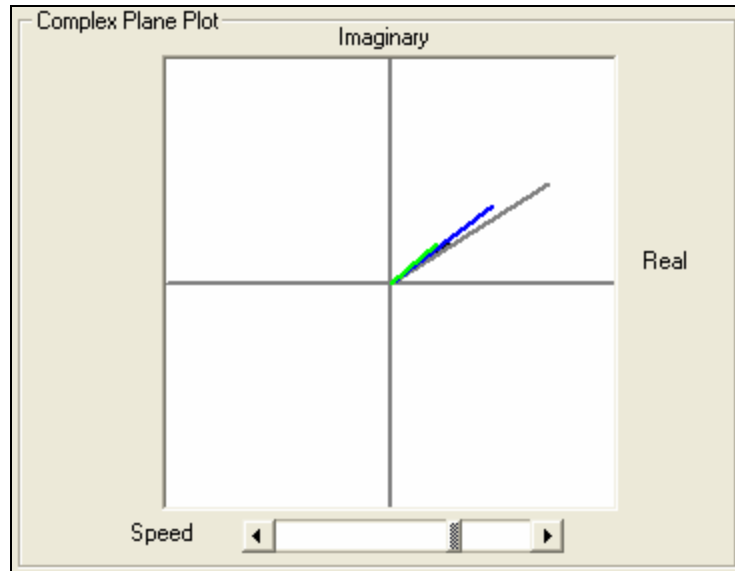


Figure 4.3 Complex Plane Plot

4.5 Comparison between Damped Mode Shape and Undamped Mode Shape

As mentioned in the first chapter, the goal of this research is to analyze the effect of vertical acceleration on structural response. A new multistory model is to be created. The model has the ability to model structures with highly non-proportional damping. One of the purposes of the creation of the CRT is to investigate and to demonstrate the difference between the damped mode shape and the undamped mode shape. In this section, the mode shape of a three-story structure is analyzed using the Complex Mode Response Tool (CRT). The schematic model of the three-story structure is shown in Figure 4.4. The structural properties of the three-story structure are shown in Table 4.1.

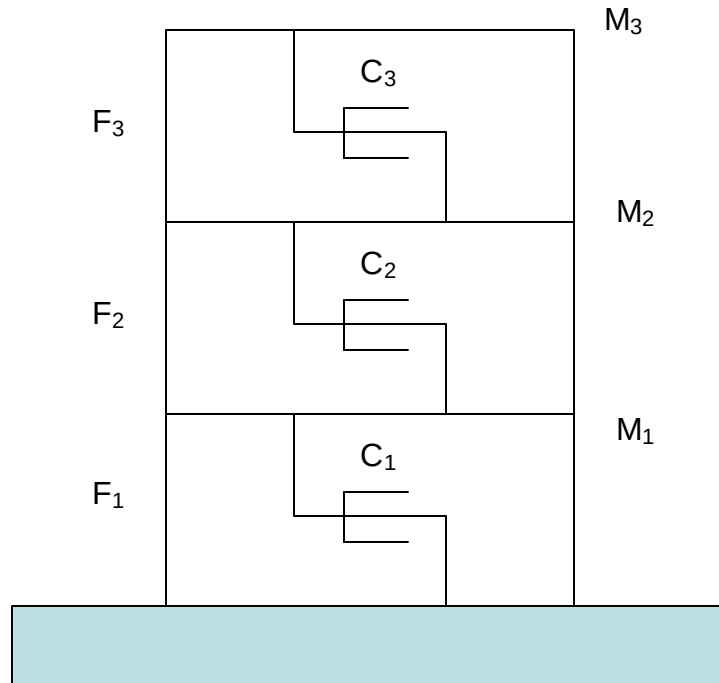


Figure 4.4 Model for Comparison

Table 4.1 Structural Properties of Model for Comparison

Story	Stiffness	Mass	Damping
3	200	2	0
2	300	2	10
1	400	2	20

By inputting the structural properties, the damped and undamped mode shapes are calculated. The damped and undamped properties are shown in Figure 4.5.

Undamped Properties			
Mode	1.000	2.000	3.000
Period (sec)	1.110	0.444	0.290
Freq (Hz)	0.901	2.251	3.443
Freq (rad/sec)	5.662	14.142	21.632
% of Damping	9.436	11.785	36.053
< >			
Damped Properties			
Mode	1.000	2.000	3.000
Period (sec)	1.102	0.434	0.299
Freq (Hz)	0.907	2.303	3.343
Freq (rad/sec)	5.699	14.467	21.007
% of Damping	9.448	10.040	38.125
< >			

Figure 4.5 Comparison between Damped and Undamped Properties

By comparing the modal properties, the difference in period and the percentage of critical damping can be observed. The phase relationship can also be seen in the complex plane plot. The complex plane plot for the first undamped mode is on the left hand side of Figure 4.6. The damped mode is on the right hand side of Figure 4.6.

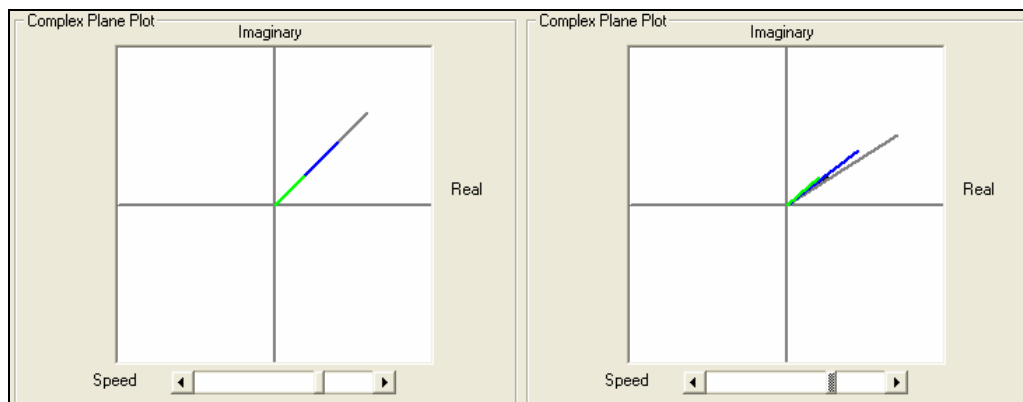


Figure 4.6 Complex Plane Plot for Undamped and Damped Mode Shape of First Mode

For the complex plane plot of the undamped mode, the lines for all stories are aligned together. This means that the displacements for every floor are in phase. However, for the

complex plane plot for the damped mode, the lines are not aligned together, which means that the motions are not in phase. The complex plane plot for the third mode of the undamped mode is on the left hand side of Figure 4.7. The damped mode is on the right hand side of Figure 4.7.

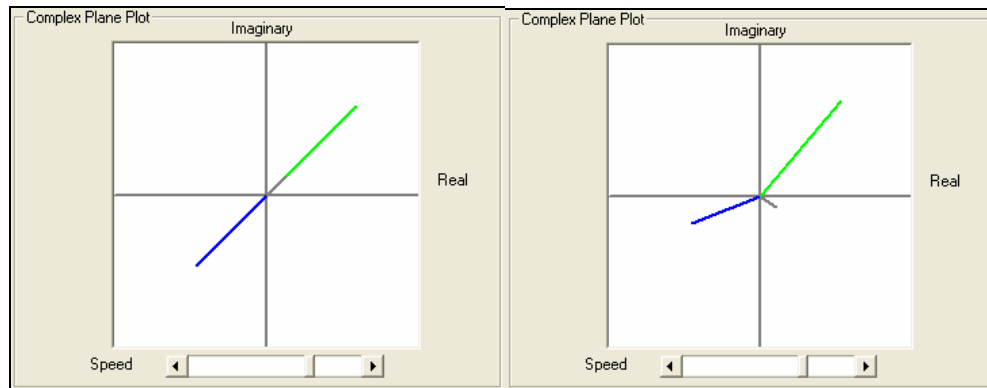


Figure 4.7 Complex Plane Plot for Undamped and Damped Mode Shape of Third Mode

For modal analysis, classical damping is assumed. The damping is required to be proportional to the mass and stiffness. However, for structures that have added damping, the assumption may not be correct. As presented in Figure 4.5 and Figure 4.6, there is significant difference between the damped mode and the undamped mode.

In the CRT Tool, there is an animation option that can show the damped mode shape of the structure in real time. Figure 4.8 shows snapshots of the animation of the second mode shape. It is interesting to see that the mode shape looks very similar to the third mode of an undamped shape. For structures that have non-proportional damping, non-classical analysis has to be used to analyze the response. The full coupled equation of motion have to be solved. Because of these reasons, the direct integration method is used to analyze the response of the structure in the newly developed multistory model.

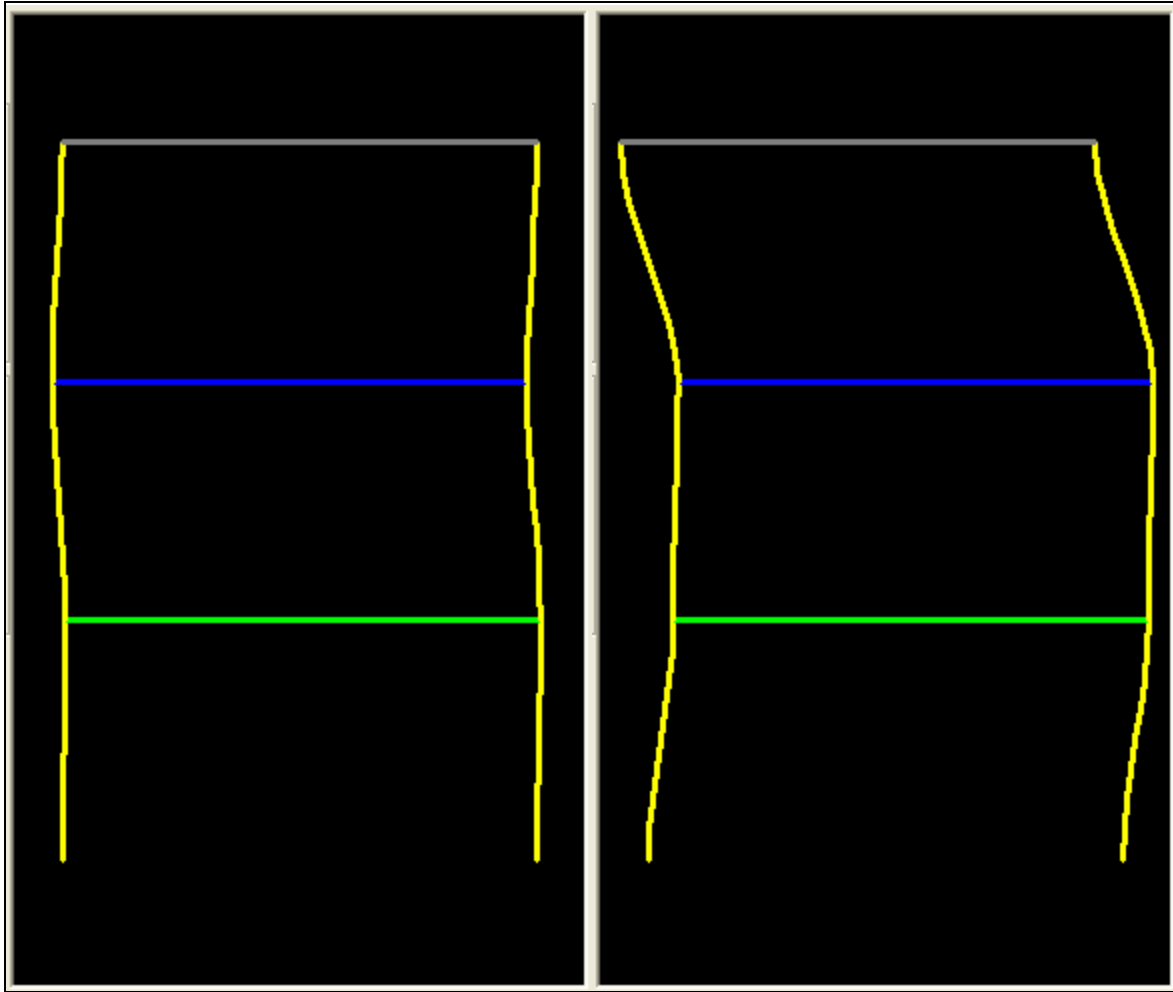


Figure 4.8 Snapshot for Second Damped Mode

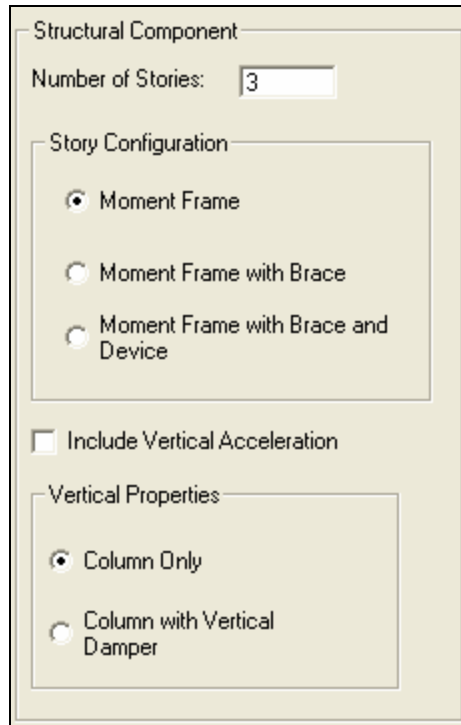
Chapter 5 Multistory Model in NONLIN

5.1 Purpose of the Development of the Multistory Model

In NONLIN version 7.0, there is a Multiple-Degree of Freedom (MDOF) Model which can handle a single story structure with a base isolator, a passive energy device, and/or base isolators. This model can be used to analyze the performance of the structure under dynamic loads. In the IDA analysis presented in the later part of this thesis, a multistory model is required. Hence, the single story model was extended to a multiple story model. As discussed in the earlier chapters, one of the goals of this thesis is to evaluate the effect of vertical acceleration on structural response. Therefore, it is also essential to incorporate the vertical acceleration in the analysis. This chapter contains a detailed description of all the features included the new multiple-story model in NONLIN. The explanation and the formulation of the program are also included.

5.2 The Description of Elements of the Multistory Model

The Multistory Model can be assessed by clicking the “Model” category under the main screen of NONLIN. There are three different types of story configuration, which are moment frame, moment frame with brace, and moment frame with brace and device. If vertical acceleration is included in the analysis, the column axial stiffness is also required. The configuration of the structure can be selected on the main screen of the “Multistory Model” of NONLIN as shown in Figure 5.1. The Multistory Model can handle any structure that is less than or equal to ten stories high. Before going into details of the story configurations, it is essential to discuss the assumption of the behavior of “Moment Frame”, “Brace”, “Device”, and “Column”.



The image shows a software window titled "Structural Component". It contains the following elements:

- A label "Number of Stories:" followed by a text input field containing the value "3".
- A section titled "Story Configuration" containing three radio button options:
 - ☒ Moment Frame
 - ☐ Moment Frame with Brace
 - ☐ Moment Frame with Brace and Device
- A checkbox labeled "Include Vertical Acceleration" which is currently unchecked.
- A section titled "Vertical Properties" containing two radio button options:
 - ☒ Column Only
 - ☐ Column with Vertical Damper

Figure 5.1 Structural Configuration Selection Window

5.2.1 Moment Frame

“Moment Frame” provides lateral stiffness by the flexural resistance of the columns. When the applied force is within the yielding limit, then the “Moment Frame” will behave elastically. However, when the rotation is beyond the yielding limit, then plastic hinges will form in the columns and the stiffness will not remain linear. It is very important to note that in actual buildings, yielding typically occurs in the girders, not in the columns as assumed by NONLIN. However, this is not an important distinction because only the story inelastic behavior is required in the simplified model in NONLIN. Modeling of actual hinges in girders is significantly more complicated, but such complex modeling is not required for the parameter analysis conducted in this thesis.

Three options are available in NONLIN to model the hysteretic behavior in the moment frame, which are “Linear”, “Bilinear”, and “Multi-linear”. When the moment frame is assumed to be “Linear”, no yielding is allowed. When the moment frame is assumed to be “Bilinear”, it will yield when the force on the moment frame is beyond the yield strength; however, the elastic and post-yield stiffness will remain the same even after numerous cycles of loadings. When the moment frame is assumed to be “Multi-linear”, it will behave nonlinearly if the force is beyond the yield limit, and the elastic stiffness and yield strength will degrade after each yielding event. The theory of the stiffness and strength degradation is discussed in Section 3.2.2.

5.2.2 Brace

The Brace is connected between two stories. The Brace provides lateral stiffness to a structure by the resistance in deformation of the brace length in the brace axial direction. Usually, braces are oversized. Therefore, the braces are assumed to have linearly elastic behavior.

5.2.3 Device

The device is connected between the frame and the brace as illustrated in Figure 5.2. The device consists of a stiffness component and damping component acting in parallel.

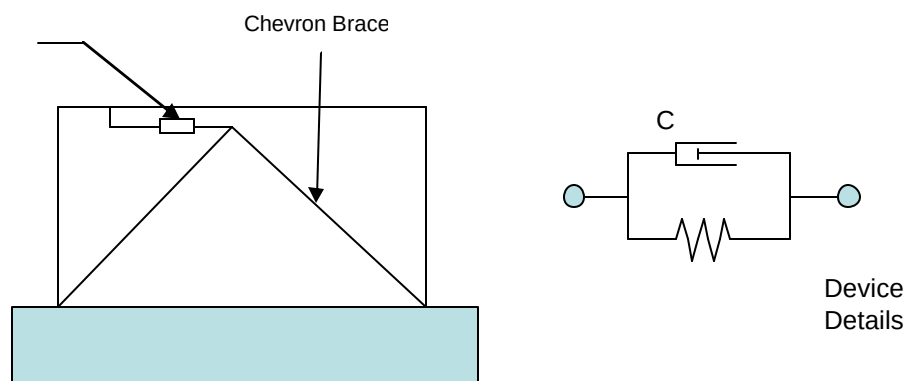


Figure 5.2 Device used in NONLIN

In Figure 5.2, the stiffness portion of the device is designated with a “K” and the added damping portion of the device is designated with a “C”. The stiffness portion of the device can have the same type of hysteretic behavior as described in the moment frame element or have linear elastic behavior. For the damping portion, the force-velocity relationship in the damper is shown in Equation 5.1.

$$f = C \cdot |v|^x \cdot \text{sign}(v) \quad (5.1)$$

where

C is the damping coefficient

x is the damping exponent

v is the deformational velocity in the damping part of the element

$\text{sign}(v)$ is the signum function of the deformational velocity in the device

The typical range of the damping exponent lies between 0.4 and 2.0. When the damping exponent is assigned to be 1.0, then the damper will have a linear force-velocity relationship, which is called a linear viscous elastic damper. When the damping exponent is assigned values other than unity, the damper will have a nonlinear force-velocity relationship, and is known as a nonlinear viscous damper. Kinetic energy dampers which represent sudden impacts can be modeled if the damping exponent is taken as 2.0 (Charney 2005). Damping exponents in the range of 0.5 to 0.8 are typical in seismic applications.

In the Multistory Model, the device can have hysteretic behavior, damping behavior, or both the hysteretic device and the damping behavior. In reality, the devices used usually have either the hysteretic properties or the damping properties. When the device only has the hysteretic behavior, the damping component is removed. When the device only has the damping behavior, the hysteretic behavior is removed.

5.2.4 Columns

Columns are assumed to remain elastic in the axial direction because they are usually much stronger in the axial direction than in the lateral direction.

5.3 Description of the Story Configuration

The following is a list of the built-in models in the multistory models in NONLIN:

- (1) Simple moment frame
- (2) Braced frame
- (3) Braced frame with device
- (4) Moment frame with vertical acceleration
- (5) Braced frame with vertical acceleration
- (6) Braced frame with device and vertical acceleration

5.3.1 Moment Frame Model

The schematic model of the two-story moment frame structure is shown in Figure 5.3. The mass matrix is formed by adding the mass in the appropriate location and the mass matrix is

given in Equation 5.2. The letters $m_{F,1}$, $m_{F,2}$ denote the mass of the frame of story 1 and story 2, respectively.

$$M = \begin{bmatrix} m_{F,1} & 0 \\ 0 & m_{F,2} \end{bmatrix} \quad (5.2)$$

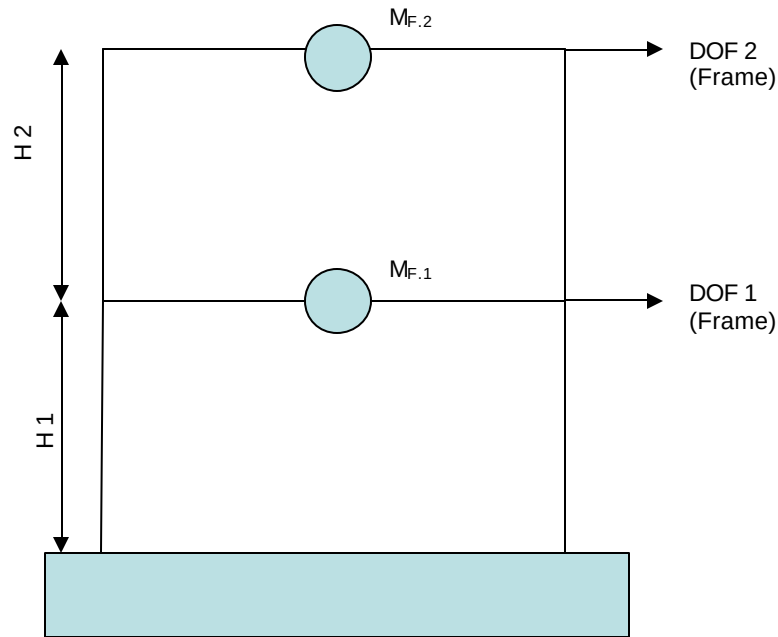


Figure 5.3 Two-Story Model Frame Model

The global stiffness matrix is derived by assembling the individual stiffness of each frame. The global stiffness matrix formed for the two-story moment frame is shown in Equation 5.3. The letters $K_{F,1}$, $K_{F,2}$ denote the lateral stiffness of story 1 and story 2 of the frame, respectively.

$$K = \begin{bmatrix} K_{F,1} + K_{F,2} & -K_{F,2} \\ -K_{F,2} & K_{F,2} \end{bmatrix} \quad (5.3)$$

When the P delta effect is included in the analysis, the geometric stiffness (K_g) has to be formed. When the vertical acceleration is not included in the analysis, the initial weight of the structure is used to calculate the geometric effect.

The formation of geometric stiffness that uses the initial weight of the structure is shown in Equation 5.4. The letters $W_{story,1}$, $W_{story,2}$, $h_{story,1}$, $h_{story,2}$ denote the weight of story 1, the weight of story 2, the height of story 1 and the height of story 2, respectively. When the P-delta effect is included in the analysis, the effective stiffness is equal to the sum of the global stiffness and the geometric stiffness.

$$K_G = \begin{bmatrix} \left(\begin{array}{cc} -W_{story,1}/h_{story,1} & -W_{story,2}/h_{story,2} \\ W_{story,2}/h_{story,2} \end{array} \right) & \left(\begin{array}{c} W_{story,2}/h_{story,2} \end{array} \right) \\ \left(\begin{array}{c} W_{story,2}/h_{story,2} \end{array} \right) & \left(\begin{array}{c} -W_{story,2}/h_{story,2} \end{array} \right) \end{bmatrix} \quad (5.4)$$

5.3.2 Brace Frame Model

The formation of the stiffness, damping, mass, and geometric stiffness for the brace frame model is very similar to the simple moment frame model. The brace is assumed to be massless. The schematic drawing of a two-story brace frame model is shown in Figure 5.4. The mass matrix of the brace frame model is shown in Equation 5.5. The letters $m_{F,1}$, $m_{F,2}$ denote the mass of the frame of story 1 and story 2, respectively.

$$M = \begin{bmatrix} m_{F.1} & 0 \\ 0 & m_{F.2} \end{bmatrix} \quad (5.5)$$

The global stiffness matrix formed for the two-story brace frame is shown in Equation 5.6. The letters $K_{F.1}$, $K_{F.2}$, $K_{B.1}$, $K_{B.2}$ denote the stiffness of the frame of story 1, frame of story 2, brace of story 1, and brace of story 2, respectively. The geometric stiffness of the brace frame is the same as for the moment frame as shown in Equation 5.4.

$$K = \begin{bmatrix} (K_{F.1} + K_{F.2} + K_{B.1} + K_{B.2}) & (-K_{F.2} - K_{B.2}) \\ (-K_{F.2} - K_{B.2}) & (K_{F.2} + K_{B.2}) \end{bmatrix} \quad (5.6)$$

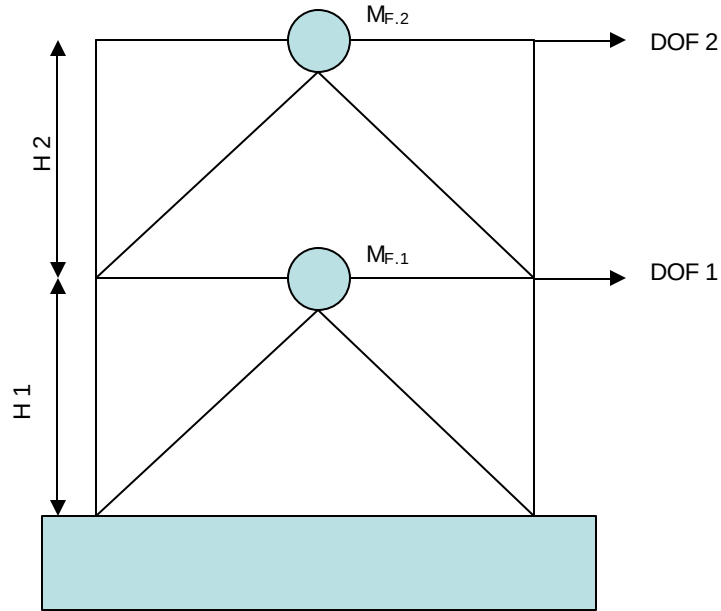


Figure 5.4 Two-Story Model Brace Frame Model

5.3.3 Brace Frame with Device Model

The brace frame with device model is the most complicated model in NONLIN. The schematic drawing of the brace frame with device model is shown in Figure 5.5.

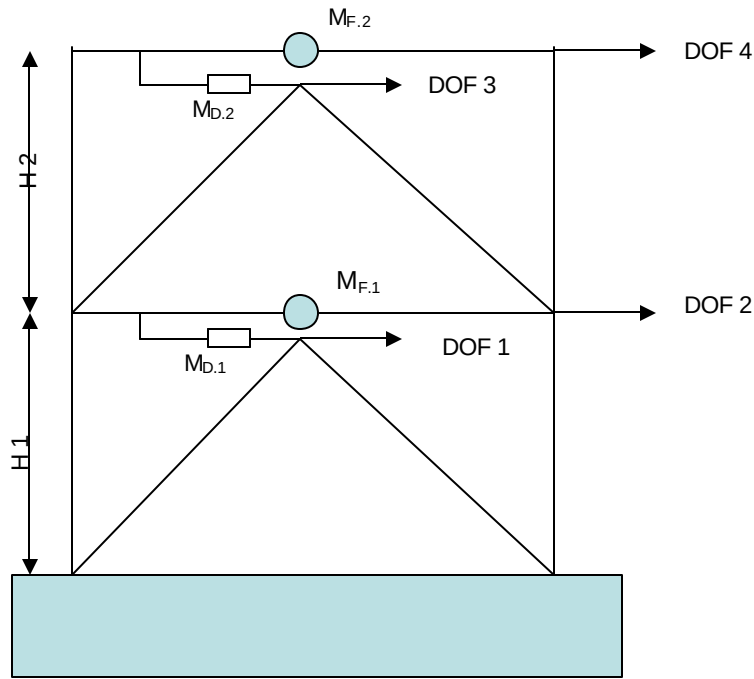


Figure 5.5 Two Story Brace Frame with Device Model

The mass matrix M is formed by lumping the masses in the appropriate location similar to the simple moment frame model. The mass matrix of the two-story brace frame (with device) is shown in Equation (5.7). The letters $m_{D,1}$, $m_{D,2}$, $m_{F,1}$, $m_{F,2}$ denote the mass of device 1, the mass of device 2, the mass of frame 1, and the mass of frame 2, respectively.

$$M = \begin{bmatrix} m_{D,1} & 0 & 0 & 0 \\ 0 & m_{F,1} & 0 & 0 \\ 0 & 0 & m_{D,2} & 0 \\ 0 & 0 & 0 & m_{F,2} \end{bmatrix} \quad (5.7)$$

The stiffness matrix is shown in Equation 5.8. The letters $K_{F,1}$, $K_{F,2}$, $K_{D,1}$, $K_{D,2}$, $K_{B,1}$, $K_{B,2}$ denote the stiffness of frame 1, frame 2, device 1, device 2, brace 1, and brace 2 respectively.

$$K = \begin{bmatrix} K_{B,1} + K_{D,1} & -K_{D,1} & 0 & 0 \\ -K_{D,1} & K_{D,1} + K_{F,1} + K_{B,2} + K_{F,2} & -K_{B,2} & -K_{F,2} \\ 0 & -K_{B,2} & K_{B,2} + K_{D,2} & -K_{D,2} \\ 0 & -K_{F,2} & -K_{D,2} & K_{D,2} + K_{F,2} \end{bmatrix} \quad (5.8)$$

The P-delta effect only contributes a significant effect on the moment frame. Therefore, the geometric stiffness is only calculated for the degree of freedom that has a moment frame. The geometric stiffness for the brace frame with device model is shown in Equation 5.9. The letters $W_{story,1}$, $W_{story,2}$, $h_{story,1}$, $h_{story,2}$ denote the weight of story 1, the weight of story 2, the height of story 1, and the height of story 2, respectively.

$$K_G = \begin{bmatrix} 0 & 0 & 0 & 0 \\ 0 & \begin{pmatrix} -W_{story,1}/h_{story,1} & -W_{story,2}/h_{story,2} \end{pmatrix} & 0 & \begin{pmatrix} 0 \\ W_{story,2}/h_{story,2} \end{pmatrix} \\ 0 & 0 & 0 & 0 \\ 0 & \begin{pmatrix} W_{story,2}/h_{story,2} \end{pmatrix} & 0 & \begin{pmatrix} -W_{story,2}/h_{story,2} \end{pmatrix} \end{bmatrix} \quad (5.9)$$

When a damping device is assigned, the damping matrix has to be modified. A damper creates damping in the structure in addition to the natural damping. The added damping matrix for the two-story brace frame with device model is shown in Equation 5.10. The letters $C_{D,1}$, $C_{D,2}$ denote the damping coefficient of damper 1, and damper 2, respectively.

$$C_{Added} = \begin{bmatrix} C_{D,1} & -C_{D,1} & 0 & 0 \\ -C_{D,1} & C_{D,1} & 0 & 0 \\ 0 & 0 & C_{D,2} & -C_{D,2} \\ 0 & 0 & -C_{D,2} & C_{D,2} \end{bmatrix} \quad (5.10)$$

5.3.4 Moment Frame with Vertical Accelerations

When vertical acceleration is included in the analysis, the stiffness and mass matrices have to be changed to include the vertical degrees of freedom. The schematic drawing of a two-story moment frame with vertical acceleration is shown in Figure 5.6.

The mass matrix of the two-story moment frame with vertical acceleration is shown in Equation 5.11. The letters $m_{F,1}$, $m_{F,2}$ denote the mass of the frame 1 and the mass of frame 2, respectively. Note that DOF1 and DOF3 have the same mass because it is the mass of the same story.

$$M = \begin{bmatrix} m_{F,1} & 0 & 0 & 0 \\ 0 & m_{F,2} & 0 & 0 \\ 0 & 0 & m_{F,1} & 0 \\ 0 & 0 & 0 & m_{F,2} \end{bmatrix} \quad (5.11)$$

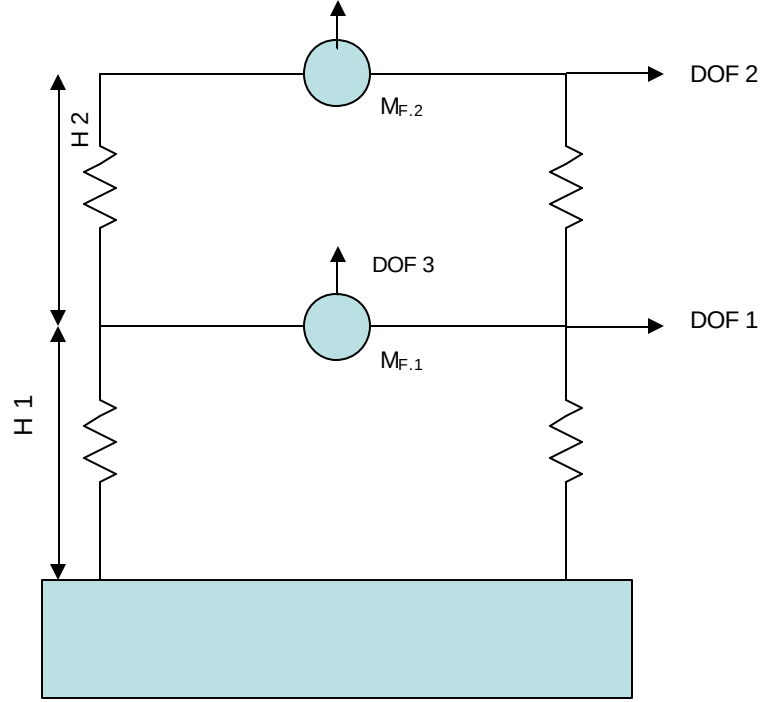


Figure 5.6 Two Story Moment Frame with Vertical Acceleration

The stiffness matrix of the two-story moment frame with vertical acceleration is shown in Equation 5.12. The letters $K_{F.1}$, $K_{F.2}$, $K_{C.1}$, $K_{C.2}$ denote the lateral stiffness of story 1, the lateral stiffness of story 2, the vertical stiffness of story 1, and the vertical stiffness of story 2, respectively.

$$K = \begin{bmatrix} K_{F.1} + K_{F.2} & -K_{F.2} & 0 & 0 \\ -K_{F.2} & K_{F.2} & 0 & 0 \\ 0 & 0 & K_{C.1} + K_{C.2} & -K_{C.2} \\ 0 & 0 & -K_{C.2} & K_{C.2} \end{bmatrix} \quad (5.12)$$

When the vertical acceleration is included in the analysis, the geometric stiffness is updated in every time step due to the change in the compression force on the column caused by the vertical acceleration. Therefore, in order to obtain the effect of vertical acceleration on the

lateral displacement, it is important to add the effect of variation in the geometric stiffness. The geometric stiffness is shown in Equation 5.13. The letters $A_{story,1}$, $A_{story,2}$ denote the total axial force in columns of story 1 and story 2, respectively. The letters $h_{story,1}$, $h_{story,2}$ denote the height of story 1 and story 2, respectively. This axial force includes the weight of the system. Note that compression force is assumed to be positive for the axial force.

$$K_G = \begin{bmatrix} -A_{story,1}/h_{story,1} & -A_{story,2}/h_{story,2} & A_{story,2}/h_{story,2} & 0 & 0 \\ A_{story,2}/h_{story,2} & -A_{story,2}/h_{story,2} & -A_{story,2}/h_{story,2} & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 \end{bmatrix} \quad (5.13)$$

5.3.5 Brace Frame with Vertical Acceleration

The schematic drawing of a two-story moment frame with vertical acceleration is shown in Figure 5.7.

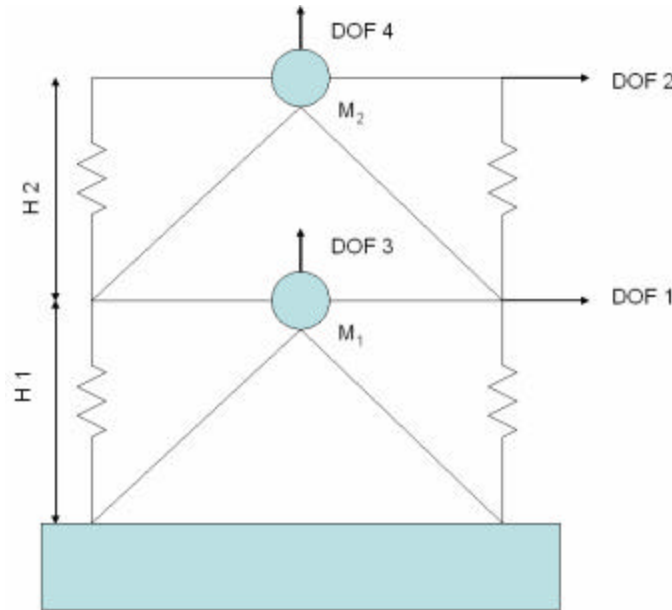


Figure 5.7 Two Story Brace Frame with Vertical Acceleration

The mass matrix of the two story brace frame with vertical acceleration is the same as for the two story moment frame and is shown in Equation 5.11.

The stiffness matrix of the two story moment frame with vertical acceleration is shown in Equation 5.14. The letters $K_{F.1}$, $K_{F.2}$, $K_{C.1}$, $K_{C.2}$, $K_{B.1}$, $K_{B.2}$ denote the lateral stiffness of story 1, the lateral stiffness of story 2, the vertical stiffness of story 1, the vertical stiffness of story 2, the lateral stiffness of the brace of story 1 and the lateral stiffness of the brace of story 2, respectively. Note that the braces are assumed not to contribute any vertical stiffness to the structure. The geometric stiffness of the brace frame model is the same as for the moment frame model and is shown in Equation 5.13.

$$K = \begin{bmatrix} K_{F.1} + K_{F.2} + K_{B.1} + K_{B.2} & -K_{F.2} - K_{B.2} & 0 & 0 \\ -K_{F.2} - K_{B.2} & K_{F.2} + K_{B.2} & 0 & 0 \\ 0 & 0 & K_{C.1} + K_{C.2} & -K_{C.2} \\ 0 & 0 & -K_{C.2} & K_{C.2} \end{bmatrix} \quad (5.14)$$

5.3.6 Brace Frame with Device and Vertical Acceleration

The schematic drawing of a two story moment frame with vertical acceleration is shown in Figure 5.8. The mass matrix of the two story moment frame with vertical acceleration is shown in Equation 5.15. The letters $m_{F.1}$, $m_{F.2}$, $m_{D.1}$, $m_{D.2}$ denote the mass of the frame 1, frame 2, device 1, device 2, respectively.

$$M = \begin{bmatrix} m_{D,1} & 0 & 0 & 0 & 0 & 0 \\ 0 & m_{F,1} & 0 & 0 & 0 & 0 \\ 0 & 0 & m_{D,2} & 0 & 0 & 0 \\ 0 & 0 & 0 & m_{F,2} & 0 & 0 \\ 0 & 0 & 0 & 0 & m_{F,1} & 0 \\ 0 & 0 & 0 & 0 & 0 & m_{F,2} \end{bmatrix} \quad (5.15)$$

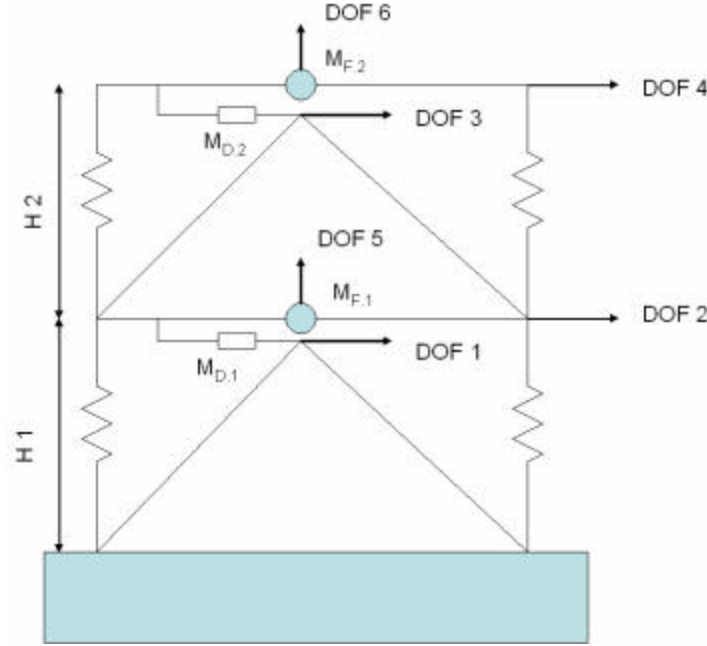


Figure 5.8 Two Story Moment Frame with Vertical Acceleration

The stiffness matrix of the two story brace frame with device and vertical acceleration is shown in Equation 5.16. The letters $K_{F,1}$, $K_{F,2}$, $K_{C,1}$, $K_{C,2}$, $K_{B,1}$, $K_{B,2}$ denote the stiffness of the frame of story 1, story 2, the column of story 1 and story 2, the brace of story 1 and story 2, respectively. Note that the braces are assumed not to contribute any vertical stiffness to the structure.

$$K = \begin{bmatrix} K_{B,1} + K_{D,1} & -K_{D,1} & 0 & 0 & 0 & 0 \\ -K_{D,1} & K_{D,1} + K_{F,1} + K_{B,2} + K_{F,2} & -K_{B,2} & -K_{F,2} & 0 & 0 \\ 0 & -K_{B,2} & K_{B,2} + K_{D,2} & -K_{D,2} & 0 & 0 \\ 0 & -K_{F,2} & -K_{D,2} & K_{D,2} + K_{F,2} & 0 & 0 \\ 0 & 0 & 0 & 0 & K_{C,1} + K_{C,2} & -K_{C,2} \\ 0 & 0 & 0 & 0 & -K_{C,2} & K_{C,2} \end{bmatrix} \quad (5.16)$$

The geometric stiffness of the brace frame model is the same as for the moment frame model and is shown in Equation 5.17. The letters $A_{story,1}$ and $A_{story,2}$ denote the axial force in columns of story 1 and story 2, respectively. The letters $h_{story,1}$ and $h_{story,2}$ denote the height of story 1 and story 2, respectively. Note that the compression force is assumed to be positive for the axial force.

$$K_G = \begin{bmatrix} 0 & -A_{story,1}/h_{story,1} & 0 & -A_{story,2}/h_{story,2} & 0 & 0 & 0 & 0 \\ 0 & A_{story,1}/h_{story,1} & 0 & A_{story,2}/h_{story,2} & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \end{bmatrix} \quad (5.17)$$

The added damping matrix for the two story brace frame with device and vertical acceleration is shown in Equation 5.18. The letters $C_{D,1}$ and $C_{D,2}$ denote the damping coefficient of damper 1 and damper 2, respectively.

$$C_{Added} = \begin{bmatrix} C_{D,1} & -C_{D,1} & 0 & 0 & 0 & 0 \\ -C_{D,1} & C_{D,1} & 0 & 0 & 0 & 0 \\ 0 & 0 & C_{D,2} & -C_{D,2} & 0 & 0 \\ 0 & 0 & -C_{D,2} & C_{D,2} & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \end{bmatrix} \quad (5.18)$$

5.4 Natural Damping in the Multistory Model

In the multistory model, natural damping is estimated by Rayleigh proportional damping. The damping ratios of the 1st and *n*th mode are assigned and the mass proportional factor and stiffness proportional factor are calculated. Then the natural damping matrix is formed by summing the mass proportional damping and the stiffness proportional damping. For detailed information of the calculation of Rayleigh proportional damping, please refer to section 4.2.

5.5 Numerical Evaluation of Dynamic Response

When the excitation is a simple function, a closed form solution can be found. However, for earthquake ground motions, it is impossible to find a closed form solution. Modal analysis can be used if damping is proportional to the mass and the stiffness and the response is in the elastic range. However, the model used in NONLIN has the added damper which generates highly non-proportional damping. The response of the structure may behave nonlinearly if the displacement is beyond the elastic range. Therefore, the only method to obtain the accurate solution is a numerical method. The equation of motion used for the numerical analysis is shown in Equation 5.19.

$$M \cdot \ddot{u} + [C_N + C_A] \cdot \dot{u} + [K_T + K_G] \cdot u = -M \cdot \ddot{a}_{g(hor.)} \cdot R_h - M \cdot \ddot{a}_{g(ver.)} \cdot R_v \quad (5.19)$$

where M is the mass;

C_N and C_A are the natural and added damping, respectively;

K_T and K_G are the tangent and geometric stiffness respectively;

$a_{g(hor.)}$ and $a_{g(ver.)}$ are the horizontal and vertical ground acceleration, respectively;

a , v , and u are the structural acceleration, velocity, and displacement, respectively;

R_h and R_v are the assembly matrices for the horizontal and vertical ground motion, respectively.

The Newmark Method is chosen because of the stability and accuracy (Chopra 2001). The most common cases of the Newmark method are the average acceleration method and linear acceleration method. The advantage of using the average acceleration method is its unconditional stability, while the linear acceleration method is conditionally stable for time steps less than 0.551 times the period of vibration. Although with a given time step the linear acceleration method provides a more accurate solution than the constant acceleration method, the constant acceleration method is chosen because of the stability reason.

In the numerical analysis, the accuracy of the solution is affected by the time steps in the analysis. Sub-stepping can increase the accuracy. In general, a time step which is 1/2000 of the fundamental period of vibration is sufficient to provide convergence in the response.

Chapter 6 Verification of Multistory Model in NONLIN

6.1 Purpose of Verification

Before proceeding to the analysis of the effect of vertical acceleration on multi-degree of freedom structures, verification has to be done to ensure that the newly developed Multistory Model is giving accurate answers. The program used for the verification is SAP 2000 (CSI 2002).

6.2 SAP Verification

SAP 2000 version 8 is chosen because it is the most popular commercial program for structural analysis. It is widely recognized for its accuracy and the speed of computations. In SAP 2000, the model used for the verifications was constructed of N-Link elements. The N-Link element chosen was “Plastic (Wen)” element. This is chosen because it can match the bilinear hysteretic model used in NONLIN. In the properties selection manual of the “Plastic” N-Link element, there is an item called “Yielding Exponent”. This is used to determine the smoothness of the elastic-inelastic intersections. When the value of “Yielding Exponent” is chosen to be too small, the behavior of the N-Link element cannot behave like a bilinear nonlinear element. Hence, in the verification, it is important to choose a high value of “Yielding Exponent” to ensure the bilinear behavior. In the verification, Yielding Exponent was equal to 2000.

P-delta effects can be included in SAP 2000 version 8. There are two options for P-delta analysis. The first option is called “P-delta”, which updates the geometric stiffness every time step using the undeformed shape. The second option is called “P-delta plus Large Displacement analysis”, which updates the geometric stiffness in every time step using the deformed shape.

The first option in SAP is very similar to the option “Use instantaneous geometric stiffness” in NONLIN. Therefore, the geometric effect in NONLIN can also be verified using SAP 2000.

There are several options to specify the damping in the time history analysis in SAP. In the verifications, Rayleigh proportional damping is used to calculate the natural damping in SAP. In the verification, the “Rayleigh” proportional damping coefficients were first obtained from NONLIN and were used to input the damping properties in SAP. This ensures that both programs have the same model for the verifications.

6.3 Description of Model Used in the Verification

Verification is only performed for the moment frame models in this thesis. The verification can be divided into these categories. They are moment frame with no P-delta effects, moment frame with P-delta effect with respect to the initial structural weight, and moment frame with P-delta effect with respect to the “instantaneous structural weight”. Linear dynamic analysis will be performed in each category, followed by the nonlinear dynamic analysis. The P-delta effect will only be considered in the nonlinear analysis.

The model used in the verification was a 3-story structure. The weight of each story was 500 kips. The lateral stiffness of each story was 500 kips/in. The vertical stiffness was 10000 kips/in. The yield strength of each story was 100 kips. Two sets of analyses were done using two different post-yield stiffness ratios, which were 0.01 and 0.1. The entire story weight was assumed to contribute to the geometric stiffness. The damping ratio was chosen to be two percent of critical for the first and third modes of the model. The mass proportional damping factor and

the stiffness proportional damping factor were 0.281 and 0.000906, respectively. The schematic drawing of the model used in the analysis is shown in Figure 6.1.

In NONLIN, there are three hysteretic models, which are “Linear”, “Bilinear”, and “Multi-linear”. As mentioned in Chapter 3, the “Multi-linear” model was created to simulate the deterioration of structures after yielding. Unfortunately, in SAP 2000, there is no model that can be used to verify this hysteretic behavior. Therefore, the verification was only performed the “Linear” and the “Bilinear” model.

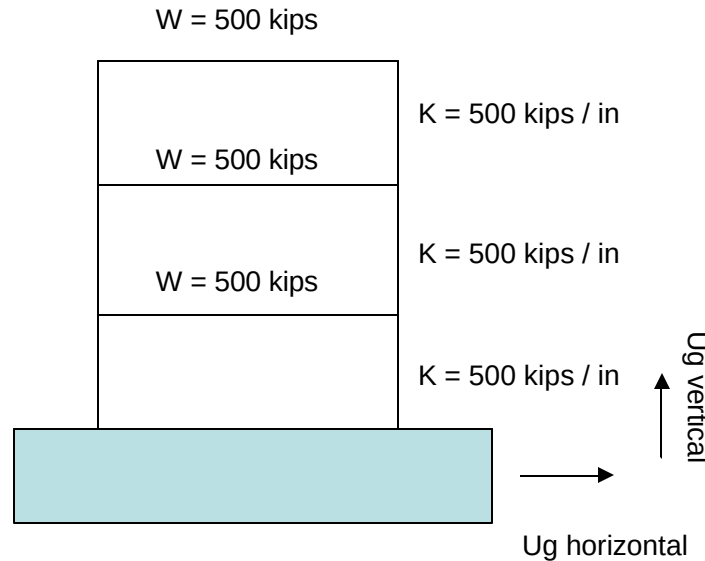


Figure 6.1 Model for Verifications

6.4 Description of Ground Motion Used in the Verification

The ground motions used in each analysis are a sine function and the Loma Prieta Ground Motion. Ground acceleration plots are shown in Figures 6.2 and 6.3, respectively. For the sine loading, the amplitude and period were 0.1 g and 1 sec, respectively. The sine loading was used as both the horizontal and vertical ground motion. For the earthquake ground motions,

the detailed information is shown in Table 6.1. The ground motions were obtained from the Pacific Earthquake Engineering Research Center (PEER) website, and were corrected to filter the instrumental errors. These ground motions were chosen randomly but the accuracy of the verification was ensured.

Table 6.1 Earthquakes Used to Compare NONLIN and SAP 2000

Earthquake	Station	Direction	PGA hor. & vert. (g)
Loma Prieta 10/18/89 00:05	Mission / Fremont, San Jose	000, Up	0.124, 0.08

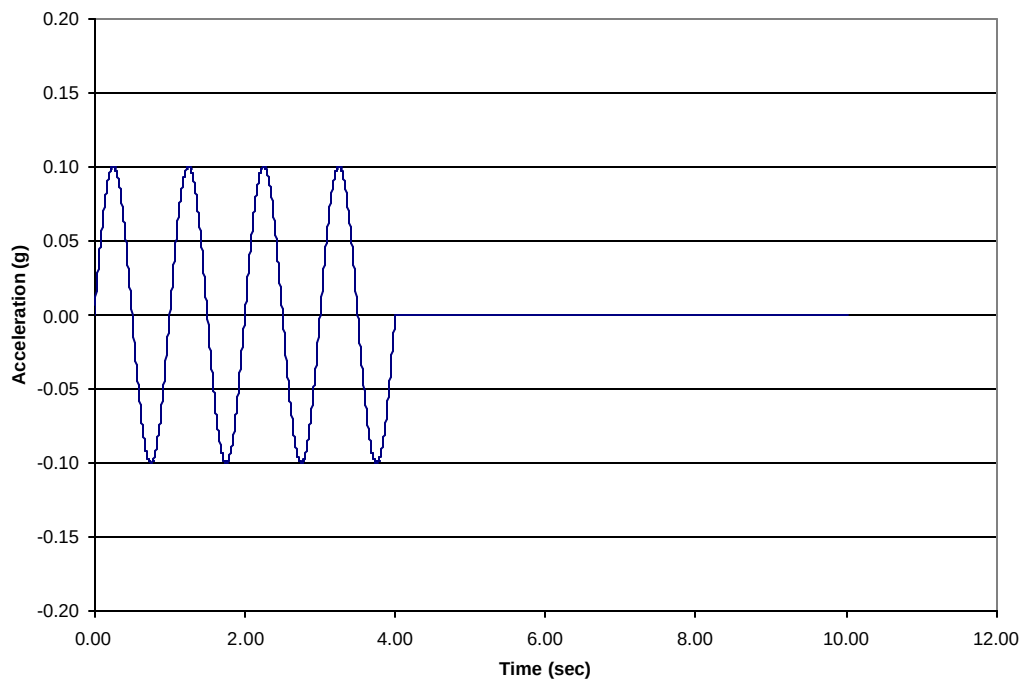


Figure 6.2 Harmonic Ground Motion (Vertical and Horizontal)

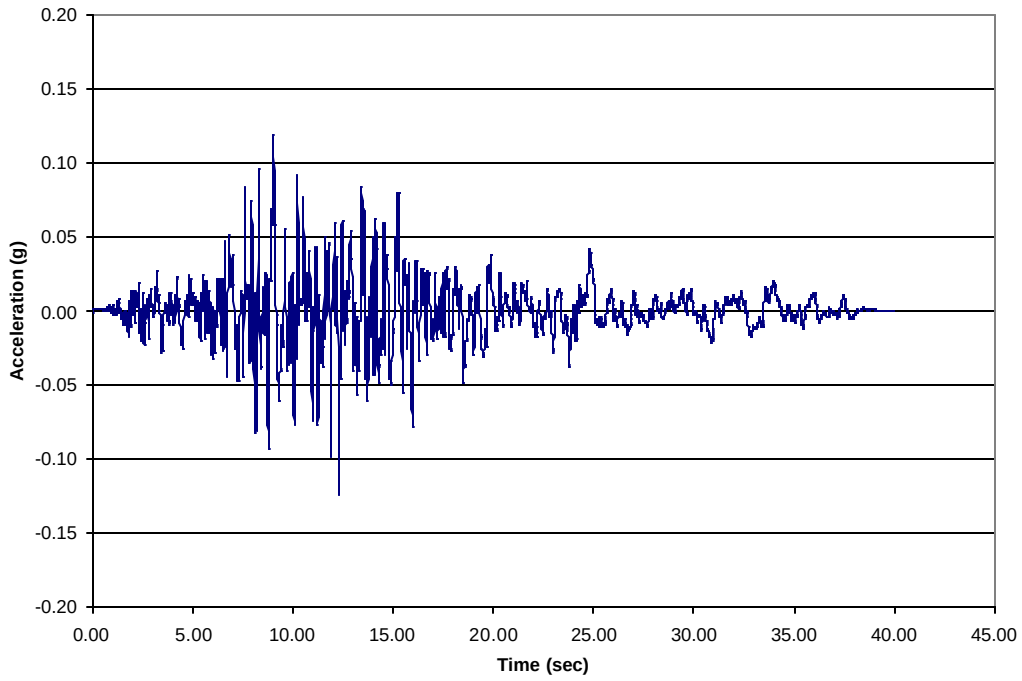


Figure 6.3(a) Loma Prieta Horizontal Acceleration

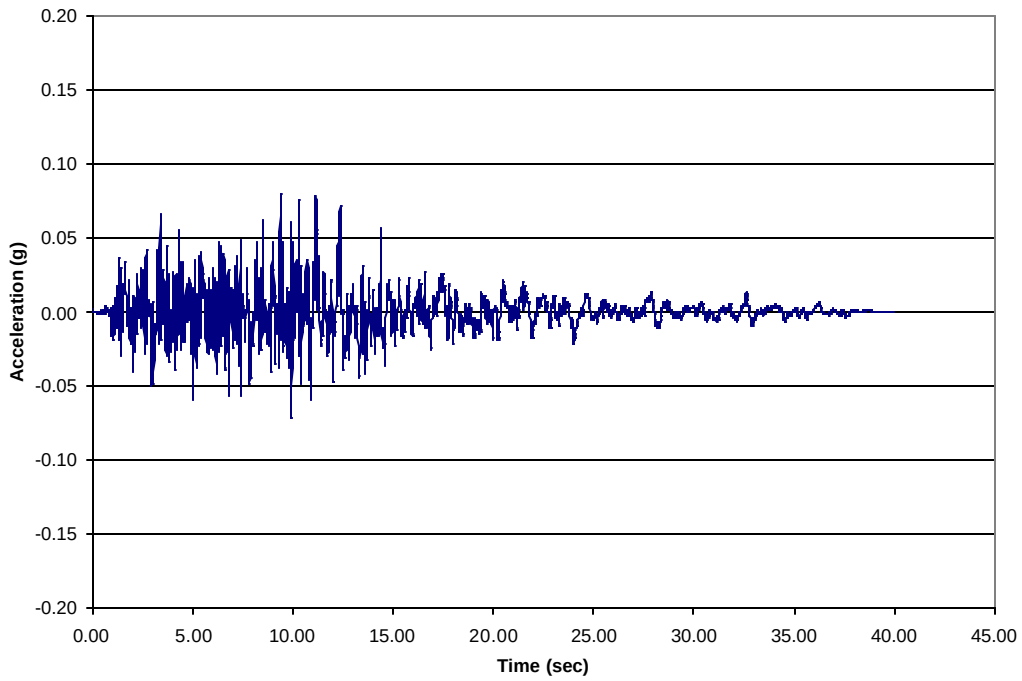


Figure 6.3(b) Loma Prieta Vertical Acceleration

6.5 Verification Plots

The verifications show that SAP and NONLIN produce very similar results. The modal properties, shown in Table 6.2, verified that SAP 2000 and NONLIN analyzed the same type of structures. The fundamental period of vibration was used for the comparison.

Table 6.2 Comparison for the Fundamental Period of Vibration

Mode	SAP 2000	NONLIN
1	0.718	0.718
2	0.256	0.256
3	0.177	0.177
4	0.161	0.161
5	0.0573	0.0573
6	0.0397	0.0397

Figures 6.4 through 6.9 show the response of the top story displacement under ground motions. Figures 6.4 to 6.7 show the responses of the structure subjected to the harmonic ground motion. Figures 6.7 to 6.9 show the structural response when subjected to the Loma Prieta Earthquake. For linear elastic analysis (Figure 6.4 and Figure 6.5), the P-delta effect was not verified because usually the difference in the response between inclusion and exclusion of the P-delta effect is insignificant (Wilson 2002). For cases that have inelastic hysteretic behavior, “a” refers to “No P-delta effect”, “b” refers to “P-delta effect updated at the beginning of the analysis”, “c” refers to “P-delta effect updated at each time step”, and “d” refers to “Vertical response”.

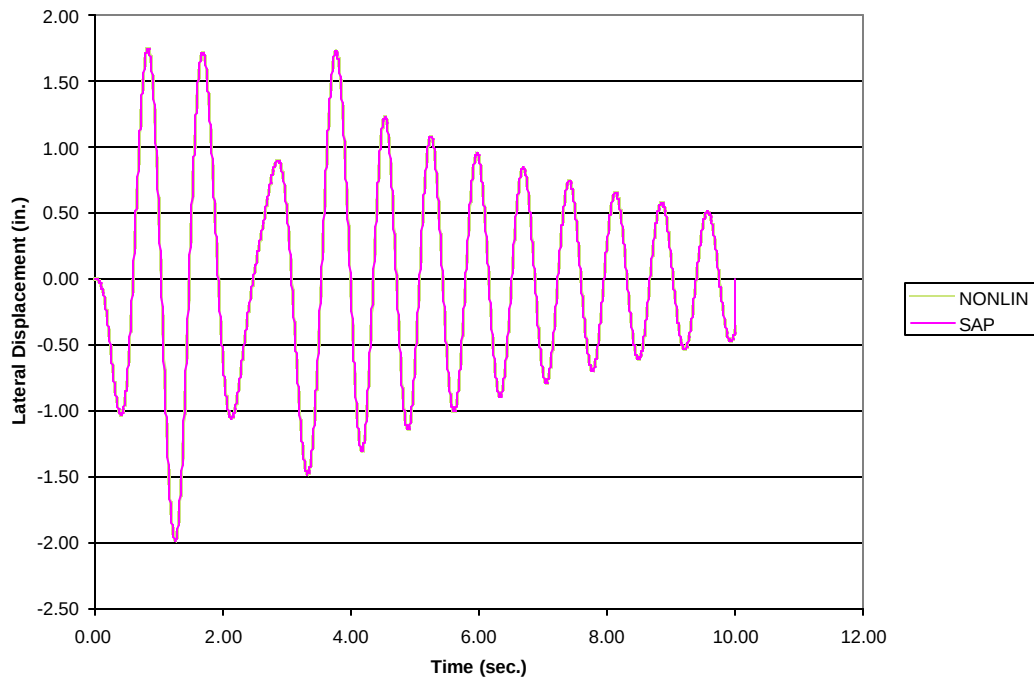


Figure 6.4 Response History of the Third Story Lateral Displacement for Structure under Horizontal Harmonic Ground Acceleration. (Elastic Stiffness, No Geometric Stiffness)

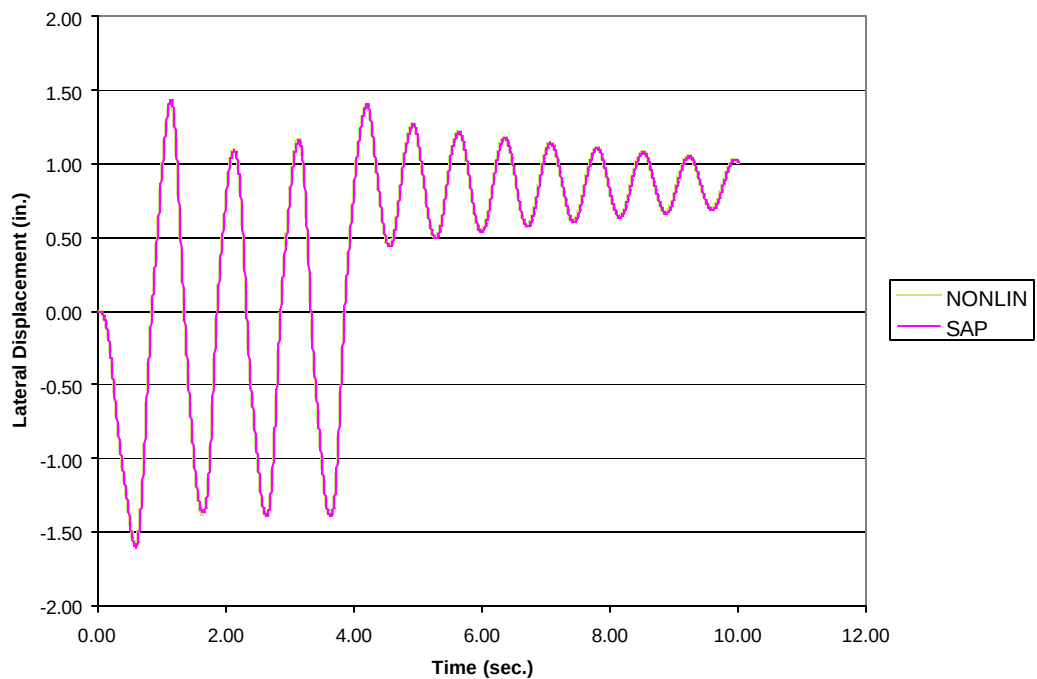


Figure 6.5(a) Response History of the Third Story Lateral Displacement for Structure under Horizontal Harmonic Ground Acceleration. (Yield Stiffness Ratios of 0.01, No Geometric Stiffness)

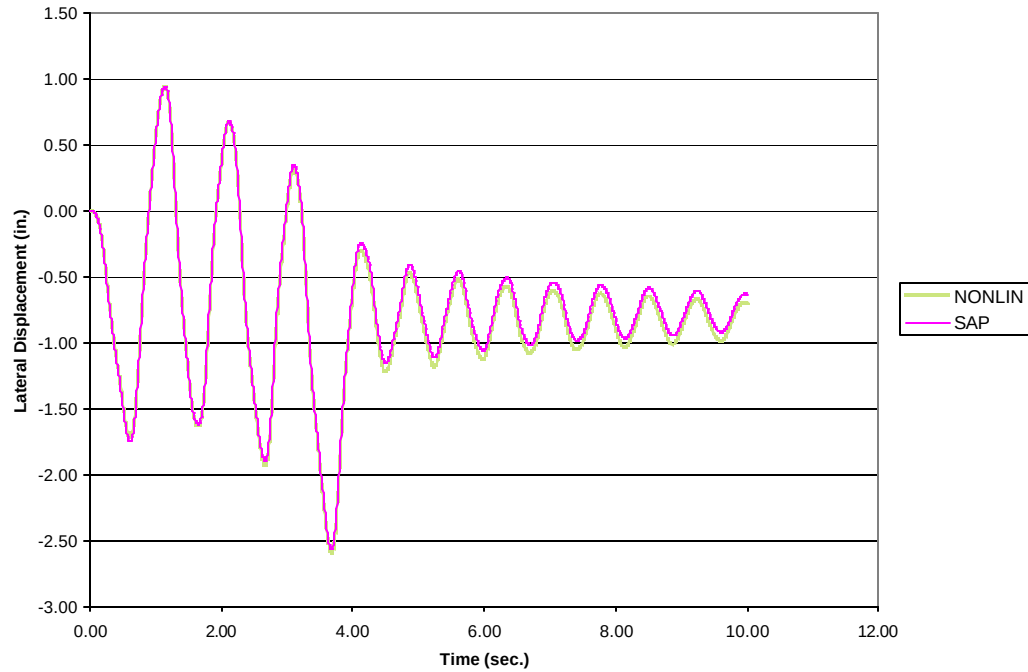


Figure 6.5(b) Response History of the Third Story Lateral Displacement for Structure under Horizontal Harmonic Ground Acceleration. (Yield Stiffness Ratios of 0.01, With Geometric Stiffness Calculated from the Initial Condition)

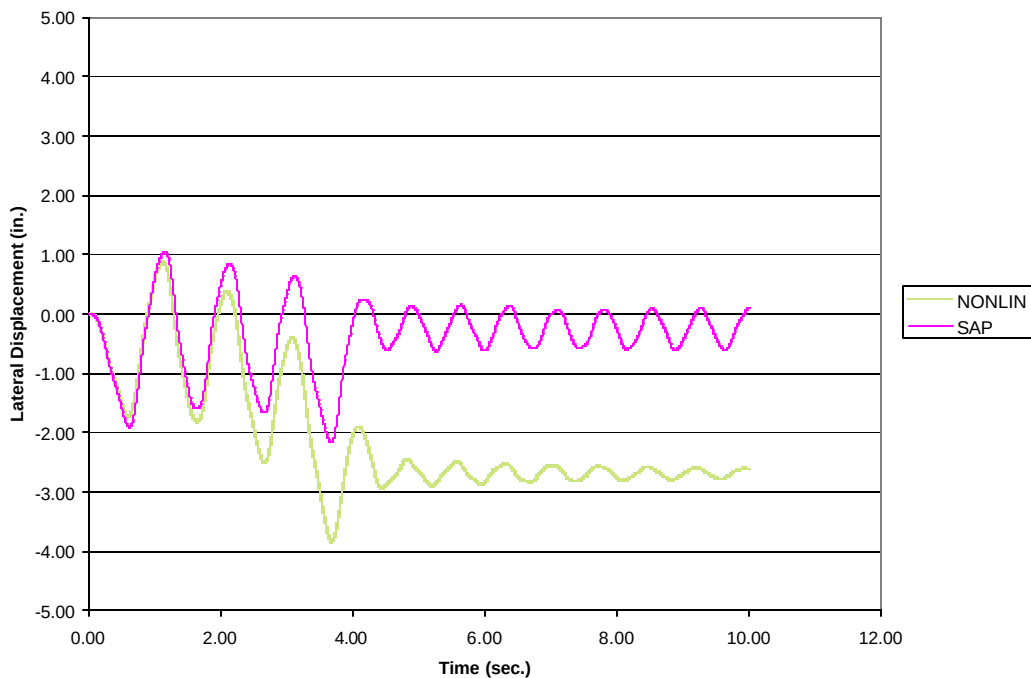


Figure 6.5(c) Response History of the Third Story Lateral Displacement for Structure under Horizontal Harmonic Ground Acceleration. (Yield Stiffness Ratios of 0.01, With Geometric Stiffness Updated in Every Time Step)

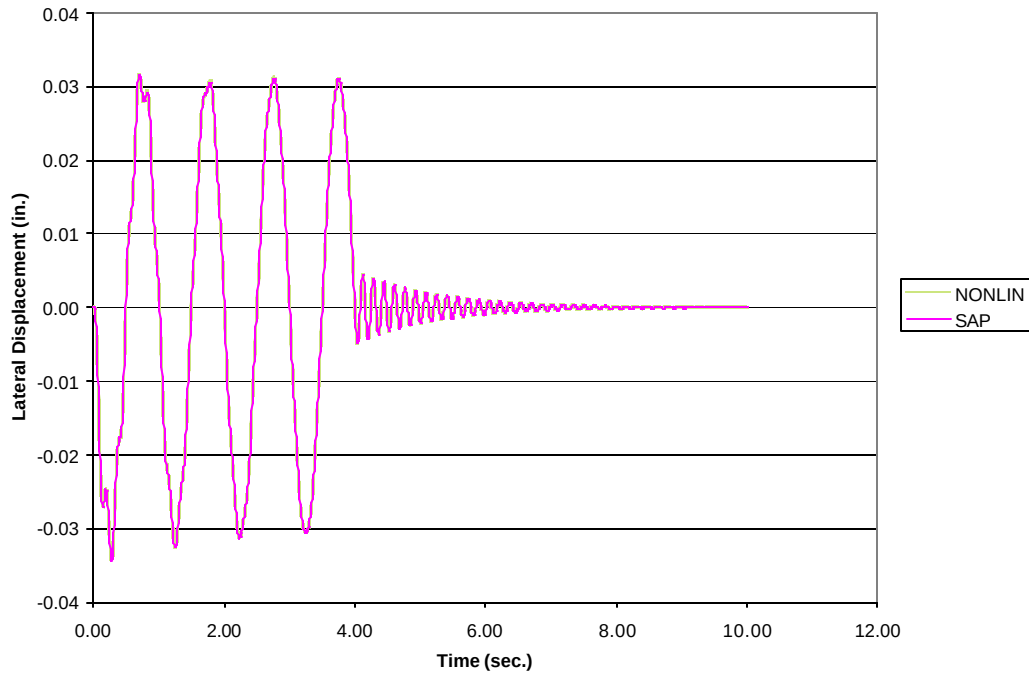


Figure 6.5(d) Response History of the Third Story Vertical Displacement for Structure under Horizontal Harmonic Ground Acceleration.

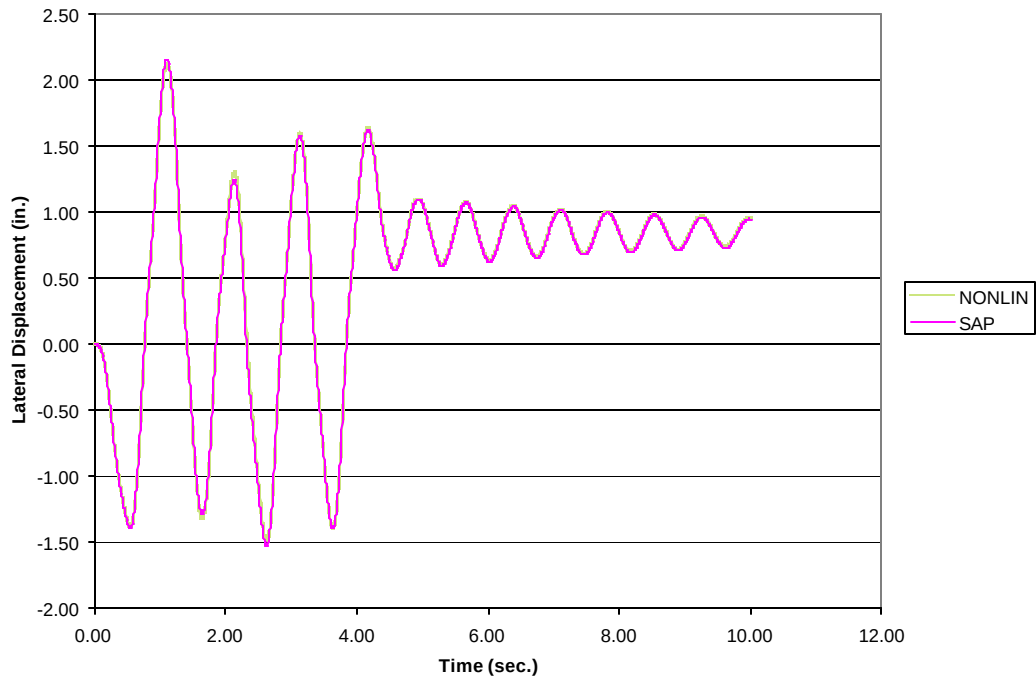


Figure 6.6(a) Response History of the Third Story Lateral Displacement for Structure under Horizontal Harmonic Ground Acceleration. (Yield Stiffness Ratios of 0.1, No Geometric Stiffness)

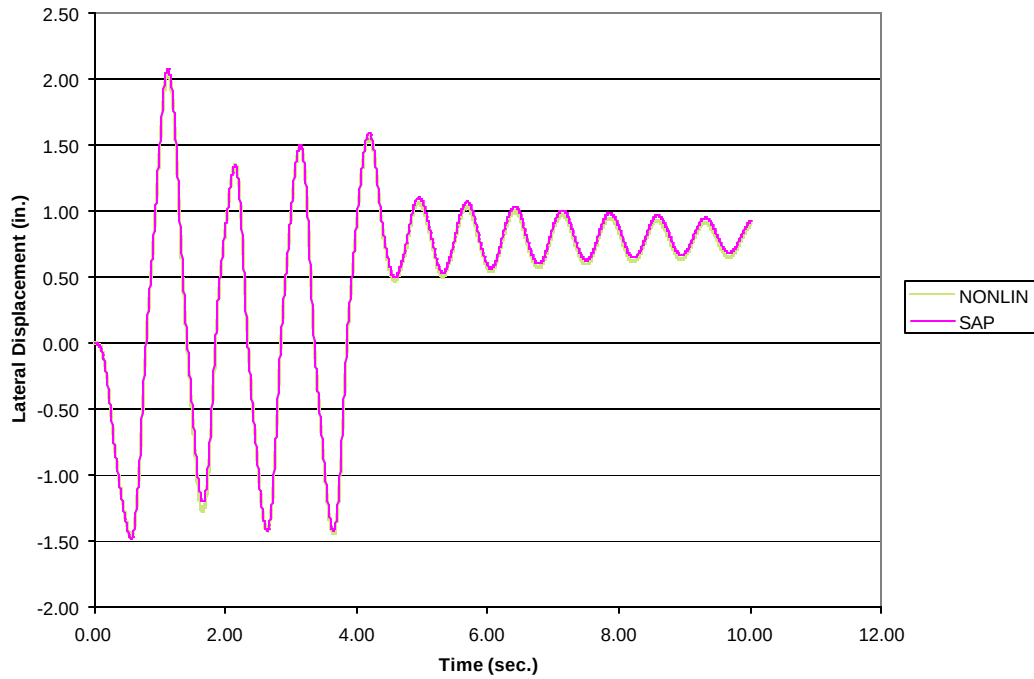


Figure 6.6(b) Response History of the Third Story Lateral Displacement for Structure under Horizontal Harmonic Ground Acceleration. (Yield Stiffness Ratios of 0.1, With Geometric Stiffness Calculated from the Initial Condition)

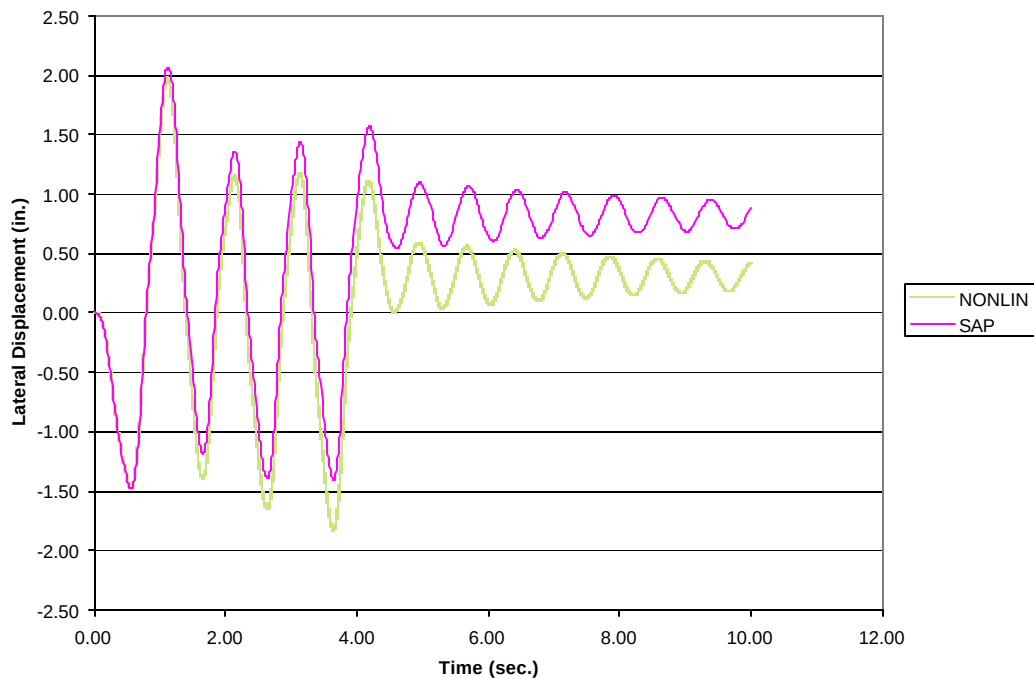


Figure 6.6(c) Response History of the Third Story Lateral Displacement for Structure under Horizontal Harmonic Ground Acceleration. (Yield Stiffness Ratios of 0.1, With Geometric Stiffness Updated in Every Time Step)

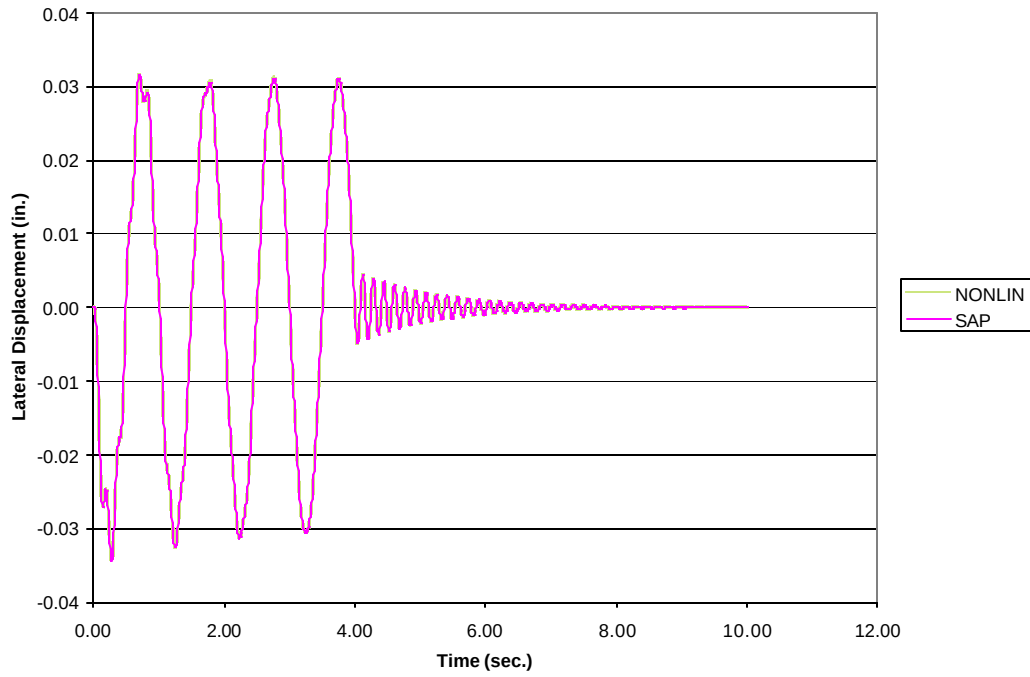


Figure 6.6(d) Response History of the Third Story Vertical Displacement for Structure under Horizontal Harmonic Ground Acceleration.

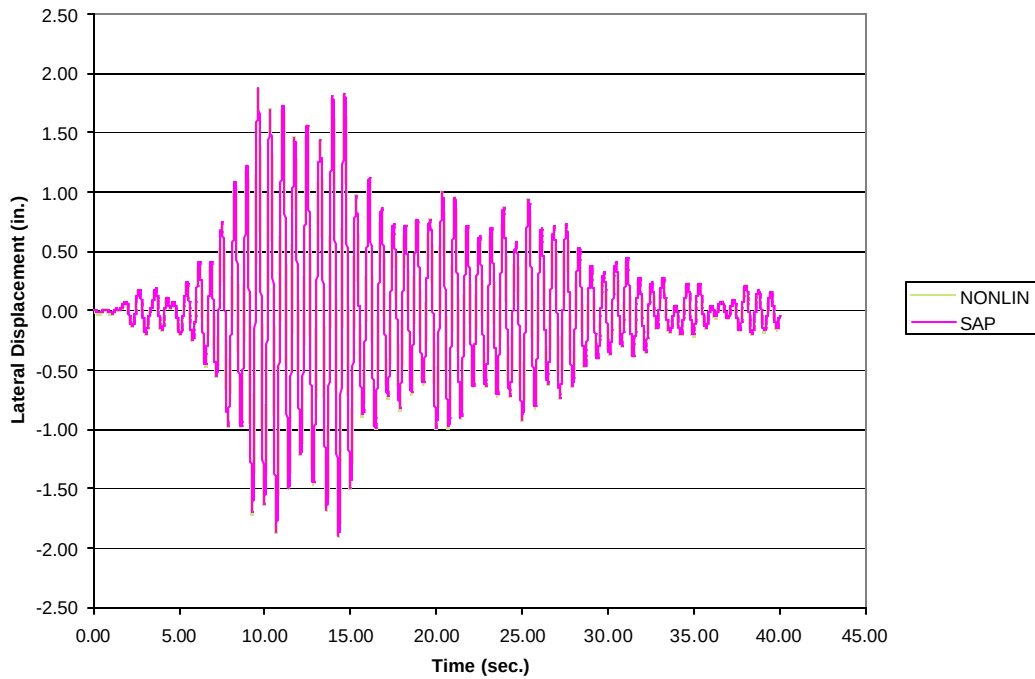


Figure 6.7 Response History of the Third Story Lateral Displacement for Structure under Loma Prieta Ground Acceleration. (Elastic Stiffness, No Geometric Stiffness)

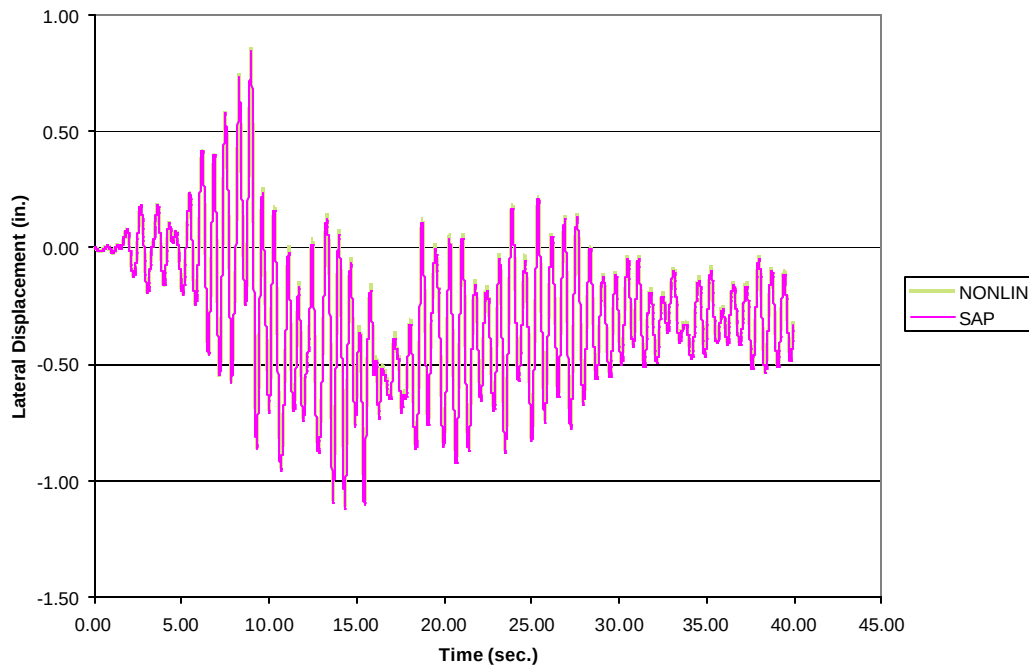


Figure 6.8(a) Response History of the Third Story Lateral Displacement for Structure under Loma Prieta Ground Acceleration. (Yield Stiffness Ratios of 0.01, No Geometric Stiffness)

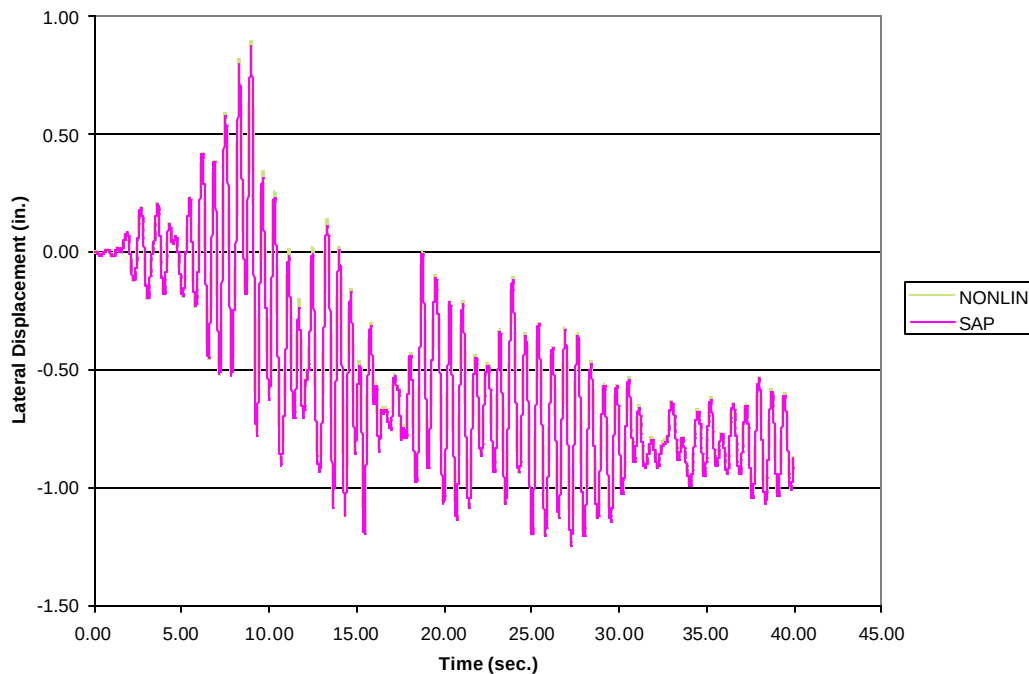


Figure 6.8(b) Response History of the Third Story Lateral Displacement for Structure under Loma Prieta Ground Acceleration. (Yield Stiffness Ratios of 0.01, With Geometric Stiffness Calculated from the Initial Condition)

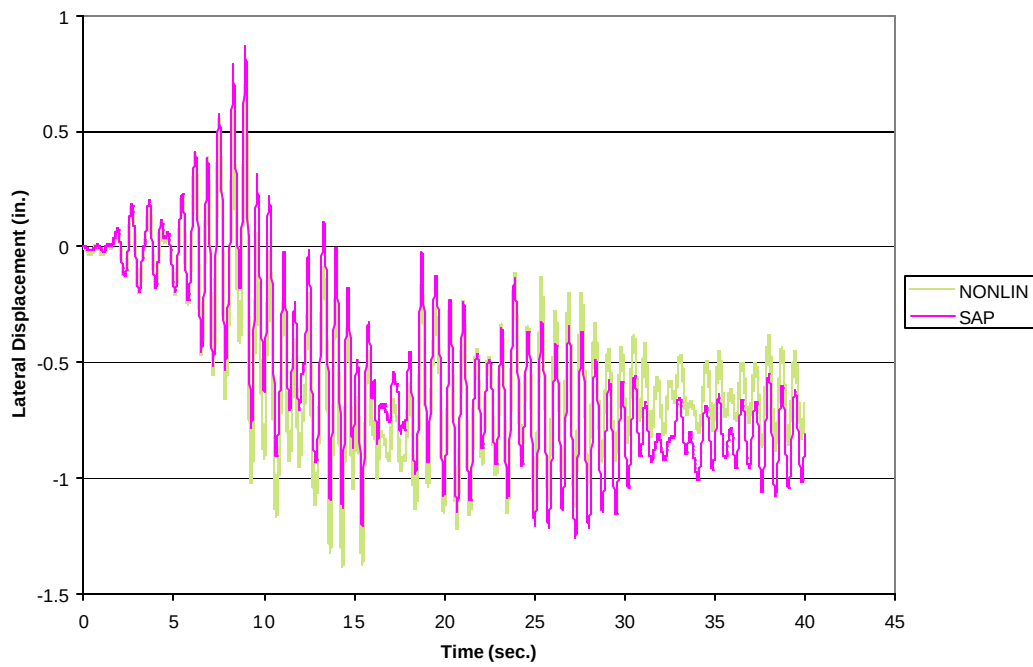


Figure 6.8(c) Response History of the Third Story Lateral Displacement for Structure under Loma Prieta Ground Acceleration. (Yield Stiffness Ratios of 0.01, With Geometric Stiffness Updated in Every Time Step)

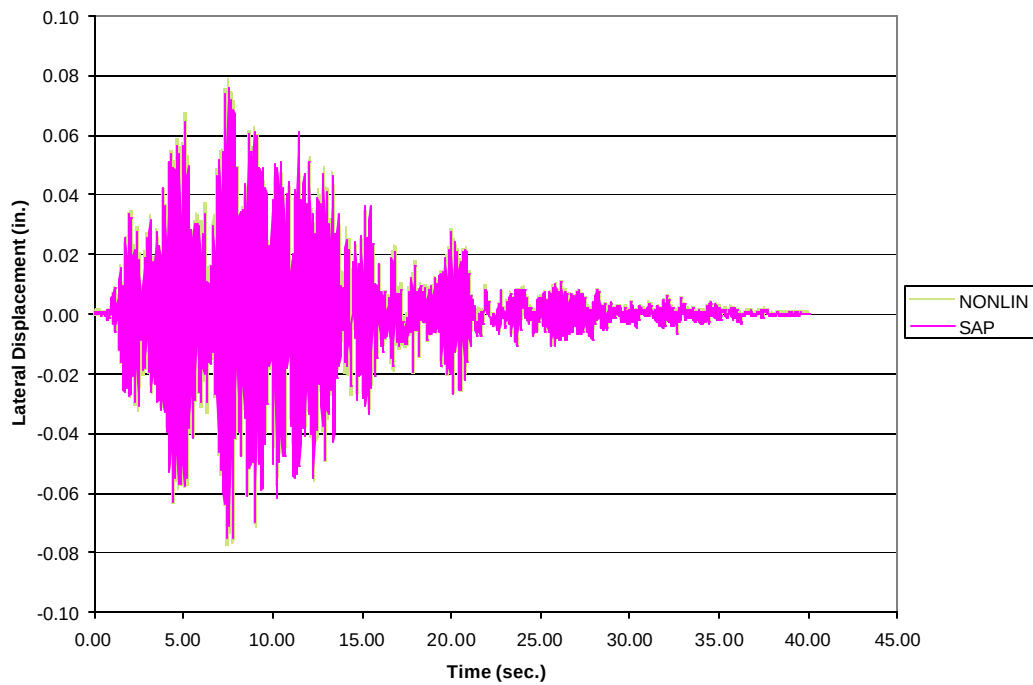


Figure 6.8(d) Response History of the Third Story Vertical Displacement for Structure under Loma Prieta Ground Acceleration.

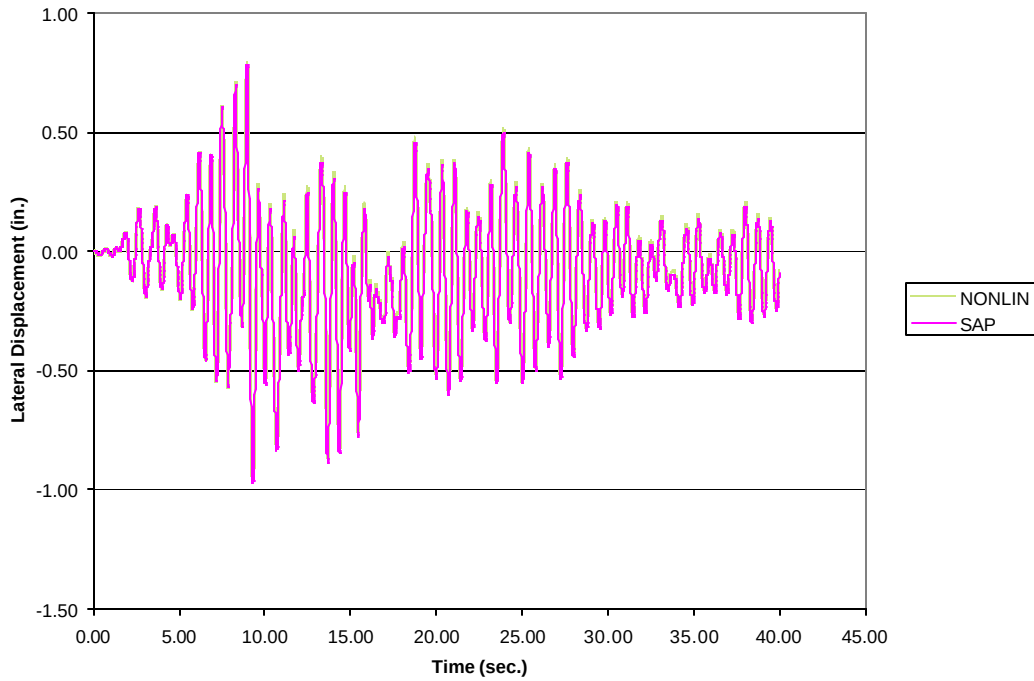


Figure 6.9(a) Response History of the Third Story Lateral Displacement for Structure under Loma Prieta Ground Acceleration. (Yield Stiffness Ratios of 0.1, No Geometric Stiffness)

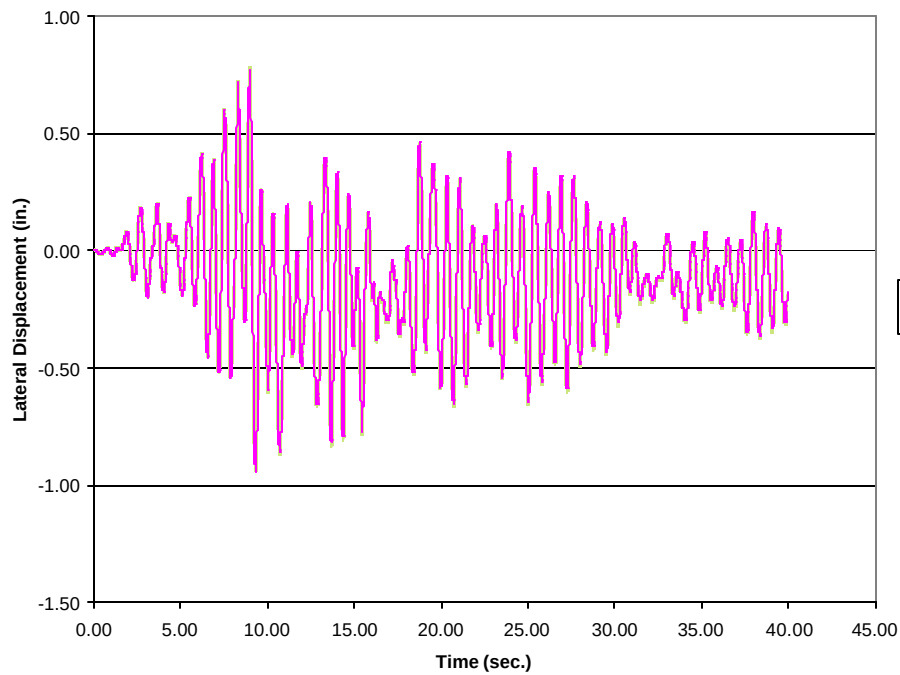


Figure 6.9(b) Response History of the Third Story Lateral Displacement for Structure under Loma Prieta Ground Acceleration. (Yield Stiffness Ratios of 0.1, With Geometric Stiffness Updated in Every Time Step)

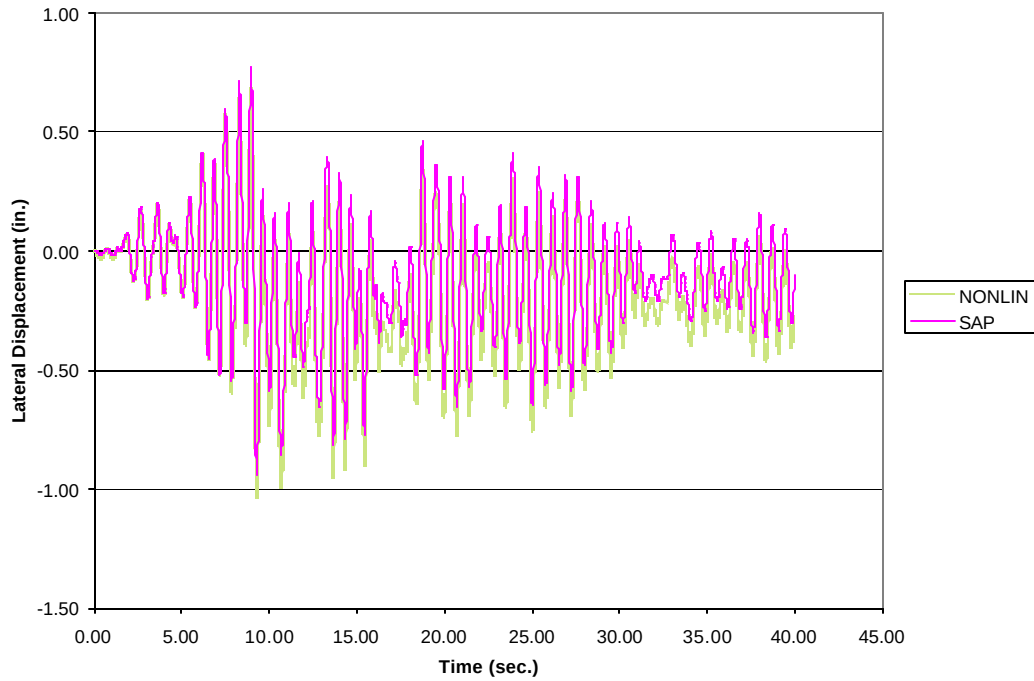


Figure 6.9(c) Response History of the Third Story Lateral Displacement for Structure under Loma Prieta Ground Acceleration. (Yield Stiffness Ratios of 0.1, No Geometric Stiffness)

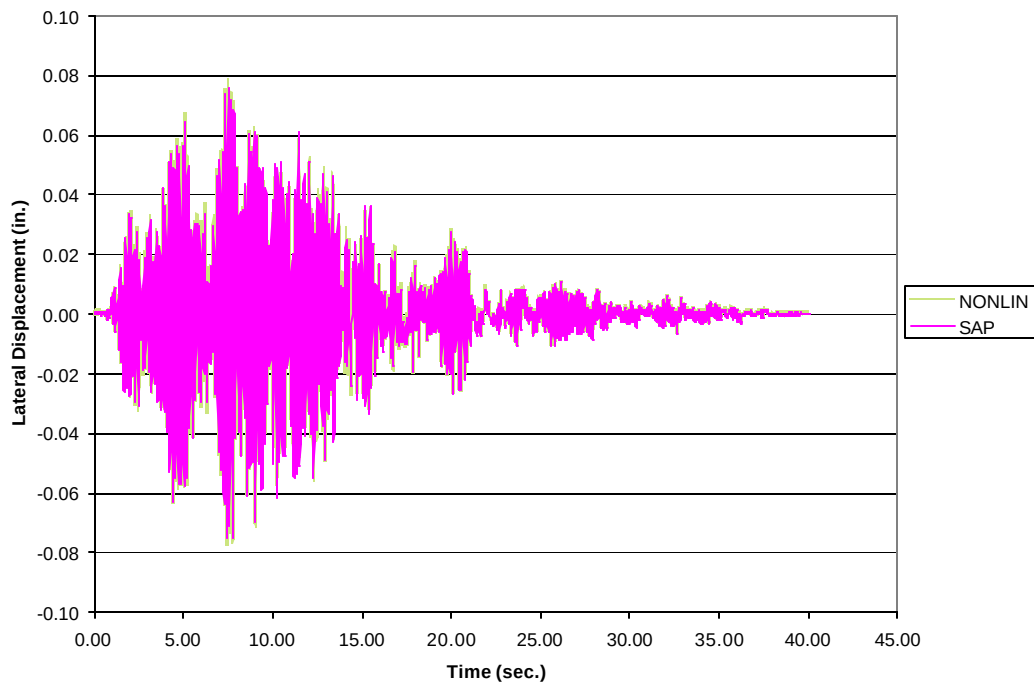


Figure 6.9(d) Response History of the Third Story Vertical Displacement for Structure under Loma Prieta Ground Acceleration.

The verifications prove that NONLIN is accurate in analyzing the structure when the geometric stiffness is not updated in every time step. However, when the geometric stiffness were updated in every time step, discrepancies were found in the lateral displacements between SAP and NONLIN. Due to this difference, SAP was used in the IDA analysis when vertical acceleration was included in the analysis. For the rest of the IDA analysis in the Chapter 7, NONLIN was used as the tool for analysis.

It is also interesting to point out the small difference between NONLIN and SAP 2000 in the response history curves even when the geometric stiffness is not updated. This is because the iterations are performed in SAP analysis; on the contrary, no iterations are performed in NONLIN. However, the accuracy of NONLIN can be improved by increasing the number of time steps, which will produce identical answers between NONLIN and SAP.

Another interesting point is the difference between SAP and NONLIN in the verification of the case “With Initial Geometric Stiffness”. This is because of the method of handling P-delta effects in SAP. In SAP 2000 version 8, the P-delta effects can be included by applying the vertical loads on the compression member. For dynamic time history analysis, the vertical load has to be included in addition to the lateral load. The vertical load has to be applied slowly using a linear (ramp) time series to eliminate the dynamic effects on the structure (Spears 2004). It is believed that the discrepancy between NONLIN and SAP is caused by the dynamic effects of the vertical loads.

In performing nonlinear dynamic analysis, it is not likely to obtain an exact correlation between two programs if the nonlinear properties and solution methods are different. The following are the identified differences between SAP and NONLIN.

1. SAP performs Newton-Raphson iteration on equilibrium while NONLIN does not. The convergence was achieved by using an exceedingly small integration time step. In all of the NONLIN analyses run, convergence was obtained when a sufficiently number of time steps was used.
2. NONLIN uses a sharp yielding model while SAP uses a “WEN” model. Slight differences in these models can cause significant differences in the dynamic responses.
3. SAP updates the tangent damping matrix in every time step, while NONLIN does not. In addition, SAP also includes the geometric stiffness in the stiffness proportional part of the tangent damping matrix, which is theoretically incorrect.

Chapter 7 Incremental Dynamic Analysis

7.1 Assumption for Model Selection

The model used in this study is a 5-story moment frame office building. Three different sets of structural properties were selected and analyzed, where each set represents a different site. The sites are: Berkeley, CA; New York, NY and Charleston, SC. The Berkeley site can be considered to have high seismicity, New York has low seismicity, and Charleston has intermediate seismicity.

There are seven assumptions in the construction of the model being analyzed:

1. The building dimensions are 120 ft (L) x 120 ft (W) x 75 ft (H) (including the basement); each story is 12.5 ft high.
2. The building density is 9 pounds per cubic foot.
3. The stiffnesses of the buildings in Charleston and New York are 75% and 50% respectively of the stiffness of the building in Berkeley.
4. The fundamental period of vibration of the 5-story building will be estimated based on Section 5.4.2 of FEMA 368 Period Determination (FEMA 2000b).
5. The stiffness of each story is increased by approximately 20% per story from the top story of the structure.
6. The selections for the strength and the stiffness of the structure will follow FEMA (2000b). The Equivalent Lateral Force Method will be used in selecting the member properties.

The procedures for model selection are simple. First, the fundamental period of vibration is estimated using the equations detailed in the Provisions. Then the stiffness of each story is estimated by back calculation. After that, the strength of each story is determined by the Equivalent Lateral Force (ELF) method.

7.1.1 Design Response Spectrum

To construct the Design Response Spectrum, one of the essential pieces of information is the framing system. In Table 5.2.2 of the NEHRP *Provisions* many structural systems are listed with various corresponding factors used in design. One of the factors is called the response modification factor, R , which is used to represent the basic ductility of the structure. The higher the R value, the higher the ductility, and the lower the shear force for which the structure must be designed.

In this study, three different types of moment frame were considered. Since only the special ductile moment frame is allowed by the *Provisions* for the building located in Berkeley, CA, it was applied in this study for that region. For the building located in New York, NY, an ordinary moment frame was used. For the building located in Charleston, SC, an intermediate moment frame was used. The special moment frame is the most ductile moment frame, and the value of R is 8. For the ordinary and intermediate moment frame, the response modification factors R are equal to 3.5 and 4.5, respectively.

The other important information required to construct the design response spectrum is the maximum considered earthquake. NEHRP has provided contour maps for determination of the

maximum considered earthquake ground motion. Maps 1 through 24 were developed with a uniform likelihood of occurrence of 2% in 50 years with 5% damping on the structure. The return period of the earthquake is about 2500 years. Two types of acceleration values were provided in the contour maps, which are the maximum considered earthquake spectral response accelerations for short period spectral acceleration (S_s) and 1 second period (S_1). The equations for calculating the design spectral accelerations at short period (0.2 sec) and 1 second period are presented in Equation 7.1 and Equation 7.2, respectively.

$$S_{DS} = \frac{2}{3} F_a S_s \quad (7.1)$$

$$S_{D1} = \frac{2}{3} F_v S_1 \quad (7.2)$$

where F_a is the coefficient for site class and mapped short period maximum considered earthquake spectral acceleration;

F_v is the coefficient for site class and mapped 1 second period maximum considered earthquake spectral acceleration;

S_s is the mapped spectral acceleration of the maximum considered ground motion at short period ;

S_1 is the mapped spectral acceleration of the maximum considered ground motion at one second period.

In Equations 7.1 and 7.2, the factor of 2/3 is based on the fact that buildings have a margin of reserve strength against collapse of about 1.5. The F_a and F_v factors account for site conditions (stiffer or softer soils). The contour maps provided by NEHRP represent ground

shaking for structures built on a class B site (firm rock), for which both F_a and F_v are 1.0 . For different site classes it is necessary to determine the actual site class coefficients.

The last item required to construct the response acceleration spectrum is the importance of occupancy. The definition for each category of seismic use group and their importance factors are outlined in Section 1.4 of the Provisions. The purpose of the importance factor is to serve as an extra safety factor for structures that are more important to public safety. The structure being considered in this study is a 5-story office building which should be in Seismic Use Group I and the corresponding importance factor is 1.

Table 7.1 shows a summary of the parameters used in constructing the seismic response coefficient curve. Appendix B1, B2, and B3 show the design spectral acceleration curve for the sites of Berkeley, CA, New York, NY, and Charleston, SC, respectively.

Table 7.1 Parameters Used in the Design Spectral Acceleration Curve

Location	Berkeley, CA	New York, NY	Charleston, SC
Site	Site B	Site B	Site B
S_s	2.1	0.43	1.66
S_1	0.93	0.095	0.47
F_a	1.000	1.000	1
F_v	1.000	1.000	1
S_{DS}	1.400	0.287	1.107
S_{D1}	0.620	0.063	0.313

7.1.2 Period Determination (Stiffness Parameter)

As mentioned before, the Equivalent Lateral Force Method (ELF) is applied in the determination of the strength and the stiffness of the structure. The fundamental period of

vibration is one of the most important pieces of information required for an ELF design. The first mode period is required to determine the design spectral acceleration (S_a), while the design spectral acceleration is used to determine the Seismic Response Coefficient (C_s). To determine the undamped period of vibration, the mass and stiffness of the structure are required. The mass is calculated based on the assumptions made in the previous section. Unfortunately, the stiffness cannot be found without a detailed design and analysis process. Therefore, a different approach will be used to estimate the stiffness of each story. In the NEHRP Provisions, there are two equations for the approximation of the fundamental period of vibration of the structure. The purpose of these equations is to estimate the period of vibration of structures with minimal information on the building. The estimated period is used to determine the C_s values. Equations 7.3 and 7.4 show the formulas for period determination as published in Section 5.4.2 of the Provisions.

$$T_a = C_r h_n^x \quad (7.3)$$

where T_a = the approximate fundamental period;

C_r and x = parameters dependent on structural type, determined from Table 5.4.2.1 of the NEHRP Provisions;

h_n = height (ft) above the base to the highest level of the structure.

$$T_a = 0.1N \quad (7.4)$$

where N = number of stories.

There are some limitations on Equation 7.4. The structure cannot exceed 12 stories and the story height cannot be less than 10 ft. Equations 7.1 and 7.2 are only based on the general description of the building type and overall dimensions to estimate the vibration period for preliminary design. The estimated period of vibration may be greater than the actual value. The usage of a greater period of vibration is not conservative. Therefore, the 2000 NEHRP Provisions has put an upper limit on calculated period in Table 5.4.2 of the Provisions. The upper limit is equal to the period estimated by Equation 7.1 or Equation 7.2 (T_a), and Coefficient (C_U). The coefficient of C_U accounts for the fact that buildings located in a high seismic zone would likely have greater stiffness than in a low seismic area.

For this study, Equation 7.3 is used for the approximation of the fundamental period of vibration. Based on the dimensions and the structural properties presented in Section 7.1.1 of this chapter, the fundamental period obtained from Equation 7.3 is 0.765 second. The coefficient for upper limit from Table 5.4.2 of the Provision for Berkeley and Charleston is 1.4. For New York, the upper limit is 1.7. Hence, the upper limit for calculated periods for Berkeley and Charleston is 1.071 second, and for New York, the upper limit is 1.301 seconds.

Once the estimated period of vibration was found, the stiffness of the structure was obtained by using an iteration method. As stated in the previous section, the lateral stiffness of a story is approximately 20% higher than the story adjacently above. Table 7.2 shows the stiffness and the weight for the model located in Berkeley, CA. The calculated fundamental period of vibration for the “Berkeley” building is 0.787 second.

Table 7.2 Lateral Stiffness and Weight of Each Story for Model in Berkeley, CA

Story	Lateral Stiffness (kips/in.)	Story Weight (kips)
5	2000	1620
4	2500	1620
3	3000	1620
2	3500	1620
1	4000	1620

Another assumption was that the structures located in the Eastern United States are more flexible than the structures in the Western United States. Based on this assumption, the stiffnesses for the New York and Charleston models have to be modified. The stiffnesses and story weights for the New York and Charleston models are shown in Table 7.3 and Table 7.4, respectively. The calculated periods of vibration for structures in New York and Charleston are 1.291 and 0.909 seconds. In all cases, the period of vibration is smaller than the upper limit in the Provision.

Table 7.3 Lateral Stiffness and Weight of Each Story for Model in New York, NY

Story	Lateral Stiffness (kips/in.)	Story Weight (kips)
5	1000	1620
4	1250	1620
3	1500	1620
2	1750	1620
1	2000	1620

Table 7.4 Lateral Stiffness and Weight of Each Story for Model in Charleston, SC

Story	Lateral Stiffness (kips/in.)	Story Weight (kips)
5	1500	1620
4	1875	1620
3	2250	1620
2	2625	1620
1	3000	1620

7.1.3 Strength Determinations

Strength is one of the important structural properties for nonlinear analysis. The Equivalent Lateral Force method is used to determine the base shear and the story shear of the structure. Before the base shear is calculated, it is required to obtain the response coefficient curve first. The description of the procedures to obtain the response coefficient curve can be found in the NEHRP Provisions. The response coefficient curve can be found in Appendices B4 through B6. Once the response coefficient curve is obtained, the base shear of the structure can be found using Equation 7.5 as published in section 5.4 of the Provisions. The base shears (V) obtained for Berkeley, New York, and Charleston are shown in Table 7.5.

$$V = C_s W \quad (7.5)$$

where C_s = the response coefficient obtained from the response coefficient curve;

W = the total weight of the structure.

Table 7.5 Seismic Coefficient and Base Shear Requirement for Models Located in Berkeley, CA, New York, NY, and Charleston, SC

Location	Berkeley, CA	New York City, NY	Charleston, SC
C_s	0.0985	0.0140	0.0766
Base Shear (Kips)	798	114	620

The lateral force induced at any level of the structure and the vertical distribution factor (C_{vx}) can be found by using Equation 7.6 and Equation 7.7, respectively.

$$F_x = C_{vx} \cdot V \quad (7.6)$$

where C_{vx} = vertical distribution factor;

V = total design lateral force or shear at the base of the structure .

$$C_{vx} = \frac{w_x \cdot h_x^k}{\sum_{i=1}^n w_i \cdot h_i^k} \quad (7.7)$$

where w_i and w_x = the portion of the total gravity load of the structure, W ,
located or assigned Level i or x

h_i and h_x = height (ft) above the base to the highest level of the
structure

k = an exponent related to the structural period as
follows:

For structures having a period of 0.5 seconds or less, $k = 1$

For structures having a period of 2.5 seconds or more, $k = 2$

For structures having a period between 0.5 and 2.5 seconds,
 k shall be 2 or shall be determined by linear interpolation
between 1 and 2

The minimum required strength was determined by using the ELF method. The round-off design strengths are shown in Table 7.6 for the buildings located in Berkeley, New York, and Charleston.

Table 7.6 Story Strength in Berkeley, CA, New York, NY, and Charleston, SC

	Strength (kips)		
Story	Berkeley, CA	New York City, NY	Charleston, SC
5	270	40	210
4	480	70	380
3	640	90	500
2	750	110	580
1	800	120	620

7.1.4 Post Yield Stiffness

The post-yield stiffness is the other important parameter for nonlinear analysis. The post-yield stiffness depends on the detailing of the frame. In this study, the post-yield stiffness is part of the parameter study. When the secondary stiffness is not a variable parameter, it will be set to be 10%.

7.1.5 Vertical Stiffness

The vertical stiffness is assumed to be 100 times the lateral stiffness. This assumption is based on story height being approximately 150 inches. It is also assumed that the vertical stiffness is approximately the same throughout the country. Therefore, the vertical stiffness of the “Berkeley” building is used for all systems in this study.

7.1.6 Natural damping

Damping has a big influence on the structural response. In this study, Rayleigh Proportional damping was used. As mentioned in Chapter 4, the damping ratios are usually assumed to be 2% to 7% for linear analysis. For analysis beyond the yield stress, the damping ratios are assumed to be 5% to 20% (Chopra 2001). In this study, a 5% damping ratio is used for the 1st and 3rd modes for the calculation of the mass and stiffness coefficient factor. The 5%

damping ratio is used because the lower the damping ratio is, the larger the response on the structure is expected; therefore, 5% is chosen based on conservative reasons (De 2004).

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7.2 Ground Motion

Selection of ground motion is very important because different earthquakes can have very different effects on different structures. The goal is to analyze as many earthquakes as possible in this study. In this study, each ground motion was used many times in different parameter studies; therefore, only two ground motions were selected. The earthquakes along with applicable information are listed in Table 7.7. The unscaled ground motions are shown in Appendices A2 and A3.

Table 7.7 Earthquakes Used to IDA

Earthquake	Station	Direction	PGA hor. & vert. (g)
Loma Prieta 10/18/89 00:05	Mission / Fremont, San Jose	000, Up	0.124, 0.08
North Ridge 01/17/94 12:312	Sylymar – Olive View Med	090, Up	0.604, 0.535

7.2.1 Scaling of Horizontal Ground Motion

Structures are designed to withstand a certain level of ground motion. It is obvious that a building located in Berkeley is designed to withstand a higher level of ground motion than in New York. Therefore, it is essential to scale the ground motion to a certain intensity before it is in the IDA analysis.

In this study, the ground motions are scaled so that the pseudo accelerations on the structure when behaving elastically are equal to the design spectral accelerations for the

corresponding period of vibration of the structure. The Design Response Spectrums for Berkeley, New York, and Charleston are shown in Appendices 7.1, 7.2, and 7.3, respectively. Table 7.8 and Table 7.9 show the design spectrum accelerations (S_a) and the horizontal scale factor of the two ground motions for each location, respectively.

Table 7.8 Earthquakes Used in IDA

Location	Berkeley, CA	New York, NY	Charleston, SC
Spectral Acceleration, S_a (g)	0.6821	0.0805	0.3447

Table 7.9 Horizontal Scale Factor for Each Location

Location	Berkeley, CA	New York, NY	Charleston, SC
Scale Factor For Loma Prieta Ground Motion	2.197	0.552	1.725
Scale Factor For Northridge Ground Motion	0.624	0.150	0.341

7.2.2 Scaling of Vertical Ground Motion

Usually, Peak Ground Accelerations (PGA) of the horizontal and the vertical ground motions are typically in the same range (Spear 2004). Therefore, it is reasonable to use the same scaling factor that is used for the horizontal ground motion for the vertical ground motion. However, one of the objectives of this study is to analyze the effect of vertical acceleration on structural response. Therefore, it is prudent to try a greater factor to amplify the effect of vertical acceleration. A scale factor for the vertical ground motion is 1.5 times of that for the horizontal ground motion. The vertical scale factor is shown in Table 7.10.

Table 7.10 Vertical Scale Factor for Each Location

Location	Berkeley, CA	New York, NY	Charleston, SC
Scale Factor For Loma Prieta Ground Motion	3.295	0.828	2.588
Scale Factor For Northridge Ground Motion	0.936	0.225	0.512

7.3 Incremental Dynamic Analysis

The Incremental Dynamic Analysis study is based on single record IDA. In each IDA curve, only one systemic parameter is varied at a time. Two ground motion records were used. Studies of three different parameters were conducted, and they are post-yield stiffness, degrading stiffness, and degrading strength. The objective of this IDA analysis is to determine the general relationship between the structural response and the variation in each parameter. In the IDA analysis, the interstory drift was used as the damage measure because of the ease of implementation. The intensity measure is the percentage of scaled Peak Ground Acceleration (PGA), and gradually increases from 20% of the scaled PGA to 200% PGA in 20% increments. At the end of each parameter study, the IDA curves were used to identify any trends. In some cases, trends were observed, while in other cases, no trend was observed.

7.3.1 Variation of Post-yield Stiffness

Three different models, which represent different structures located in Berkeley, New York, and Charleston, were used in the analysis of the effect of variation of the post-yield stiffness. The details of each structural property can be found in Section 7.1.

The different post-yield stiffnesses used were 0%, 2%, 4%, 6%, 8%, and 10% of the elastic stiffness. In addition, three different methods of handling the geometric stiffness were used, which were neglected geometric stiffness, initial geometric stiffness, and updated geometric stiffness.

Figure 7.1 and Figure 7.2 show the peak interstory drift for the “Berkeley” building when subjected to Loma Prieta and Northridge ground motions, respectively. Figure 7.3 and Figure 7.4 show the peak interstory drift for the “New York” building when subjected to Loma Prieta and Northridge ground motions, respectively. Figure 7.5 and Figure 7.6 show the peak interstory drift for the “Charleston” building when subjected to Loma Prieta and Northridge ground motions, respectively. The letters “(a)”, “(b)” and “(c)” represent neglected geometric stiffness, included initial geometric stiffness, and updated geometric stiffness, respectively.

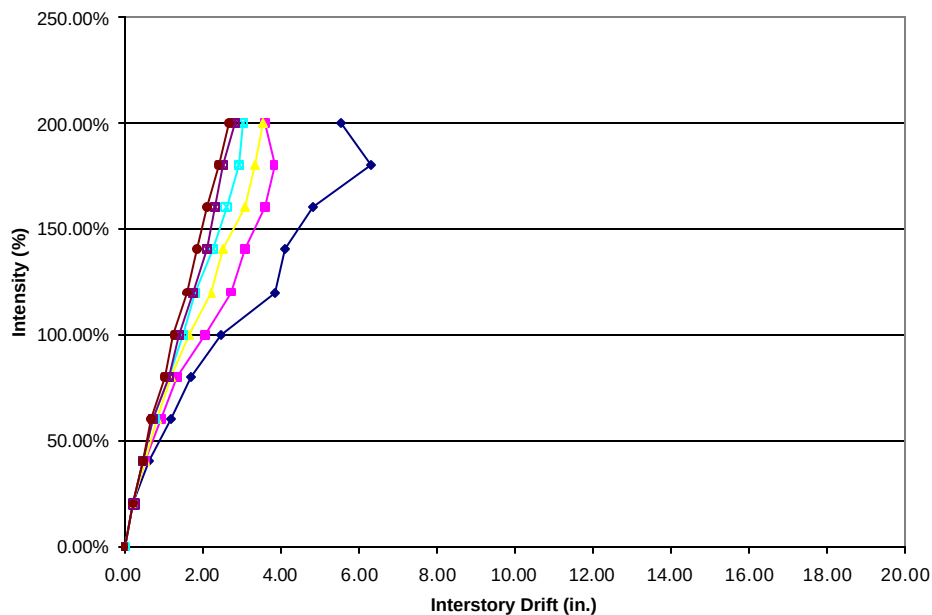


Figure 7.1(a) IDA Plot of Interstory Drift for the Berkeley Building under Loma Prieta Ground Motion for variable secondary stiffness without considering geometric stiffness

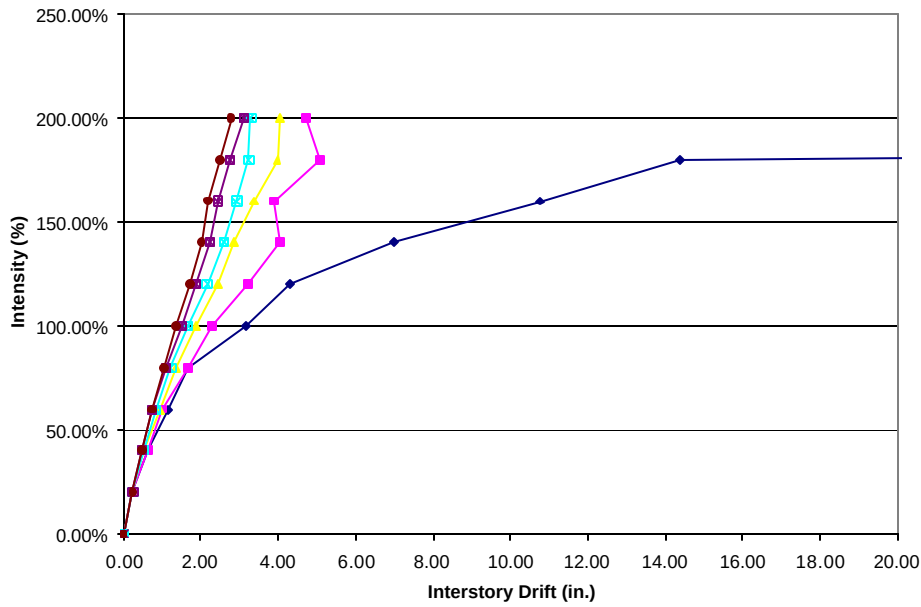


Figure 7.1(b) IDA Plot of Interstory Drift for the Berkeley Building under Loma Prieta Ground Motion for variable secondary stiffness with initial geometric stiffness

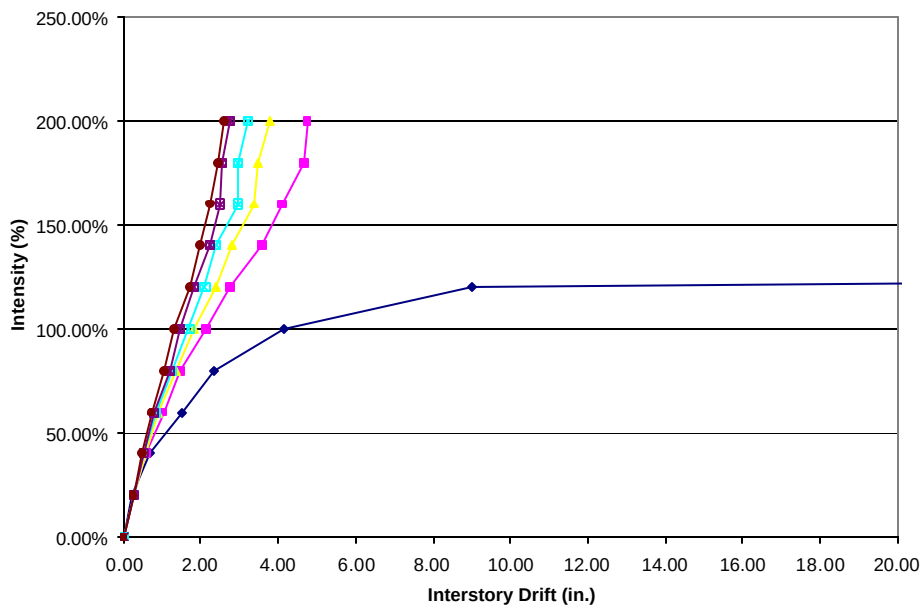


Figure 7.1(c) IDA Plot of Interstory Drift for the Berkeley Building under Loma Prieta Ground Motion for variable secondary stiffness with updated geometric stiffness

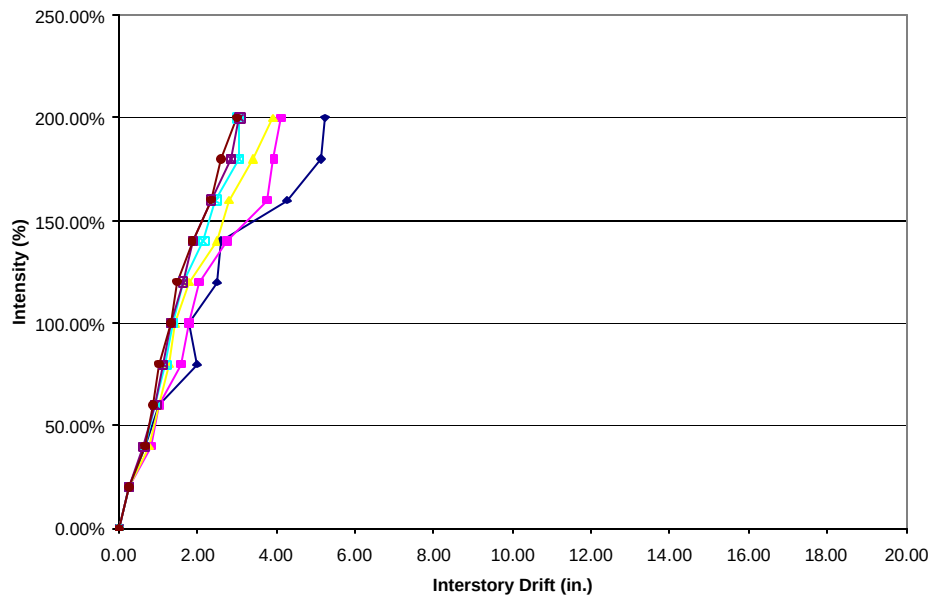


Figure 7.2(a) IDA Plot of Interstory Drift for the Berkeley Building under Northridge Ground Motion for variable secondary stiffness without considering geometric stiffness

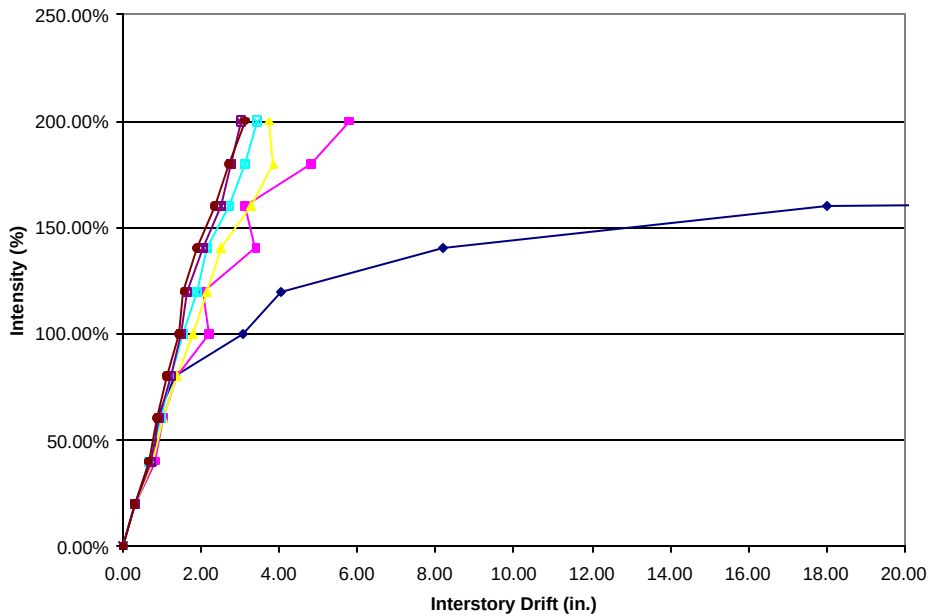


Figure 7.2(b) IDA Plot of Interstory Drift for the Berkeley Building under Northridge Ground Motion for variable secondary stiffness with initial geometric stiffness

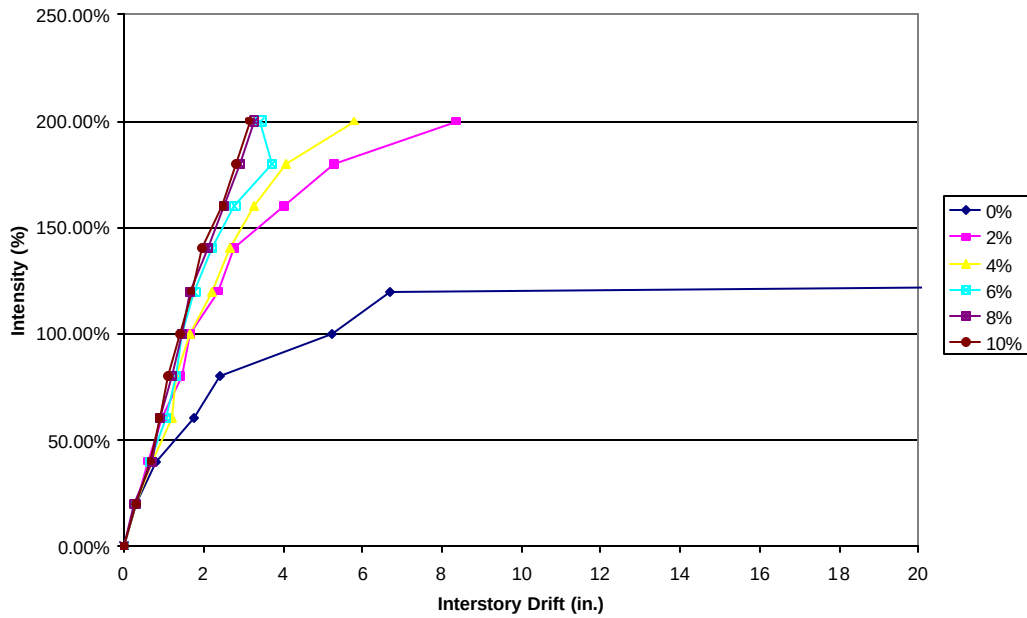


Figure 7.2(c) IDA Plot of Interstory Drift for the Berkeley Building under Northridge Ground Motion for variable secondary stiffness with updated geometric stiffness

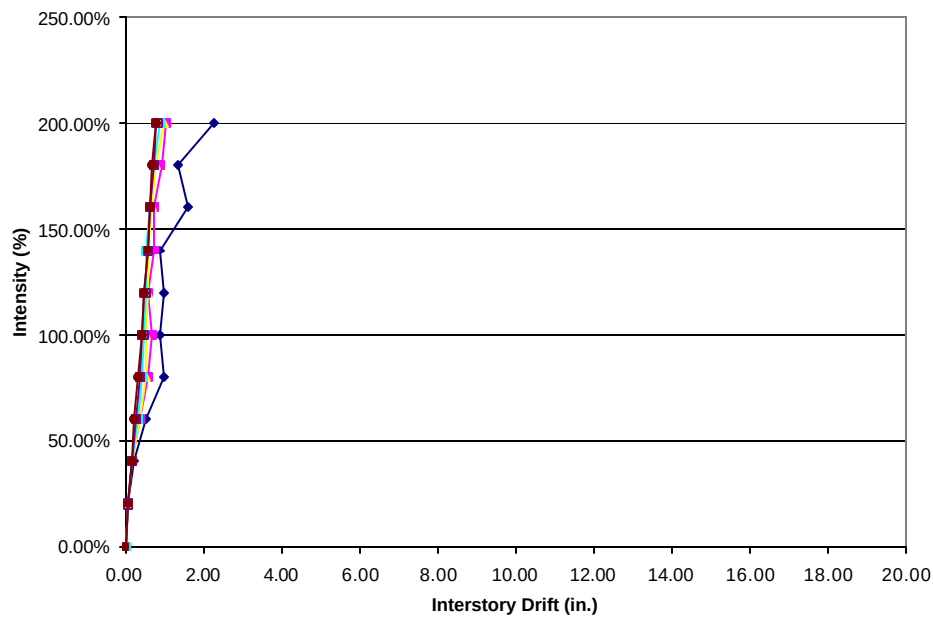


Figure 7.3(a) IDA Plot of Interstory Drift for the New York Building under Loma Prieta Ground Motion for variable secondary stiffness without considering geometric stiffness

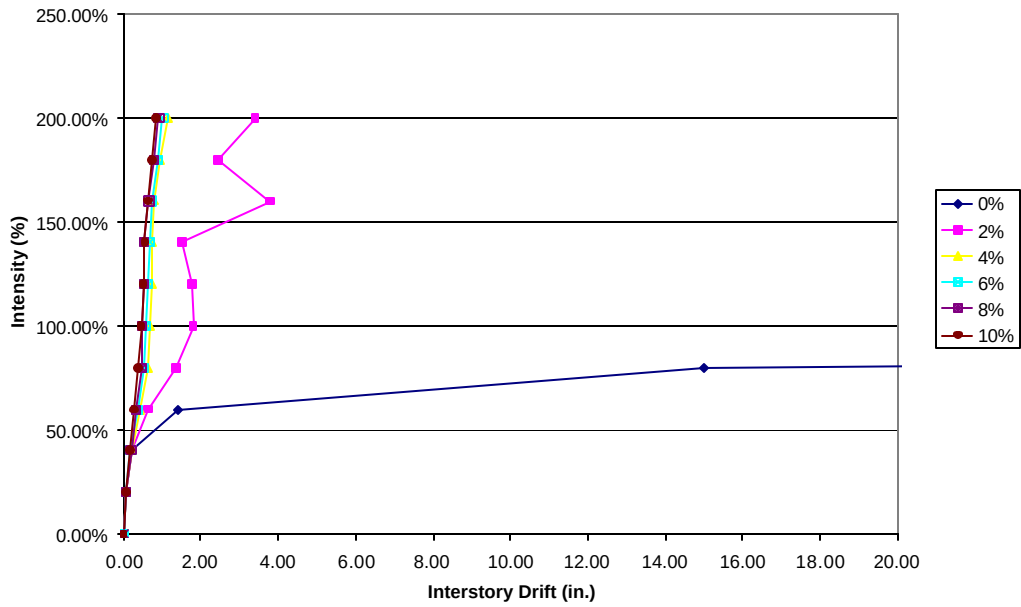


Figure 7.3(b) IDA Plot of Interstory Drift for the New York Building under Loma Prieta Ground Motion for variable secondary stiffness with initial geometric stiffness

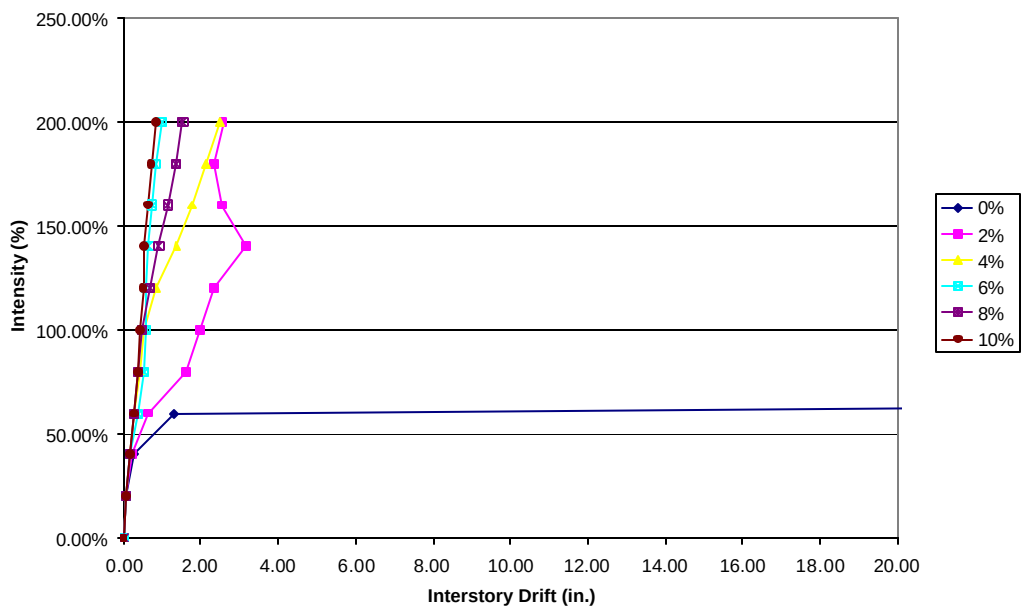


Figure 7.3(c) IDA Plot of Interstory Drift for the New York Building under Loma Prieta Ground Motion for variable secondary stiffness with updated geometric stiffness

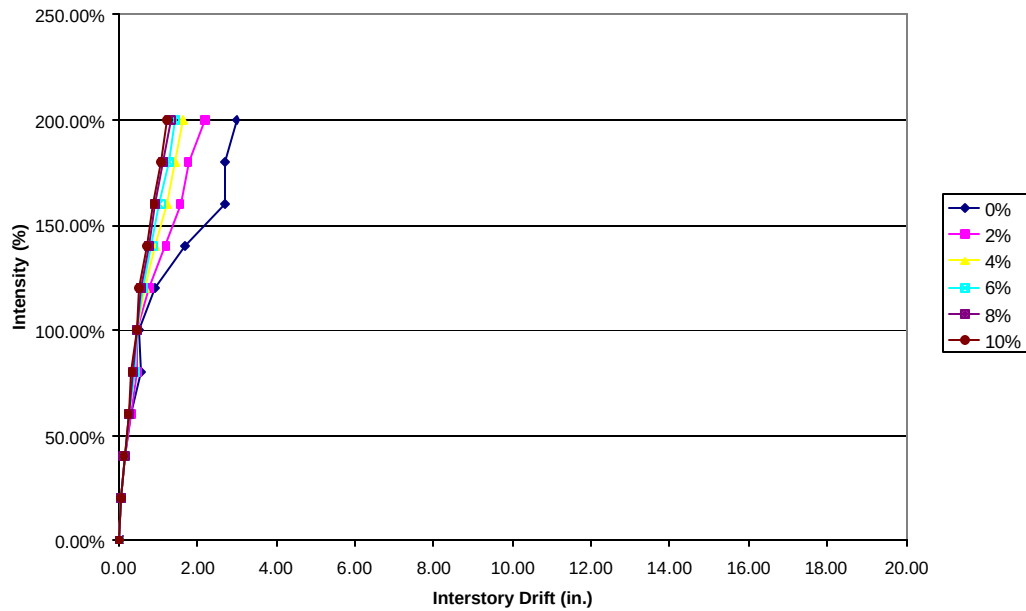


Figure 7.4(a) IDA Plot of Interstory Drift for the New York Building under Northridge Ground Motion for variable secondary stiffness without considering geometric stiffness

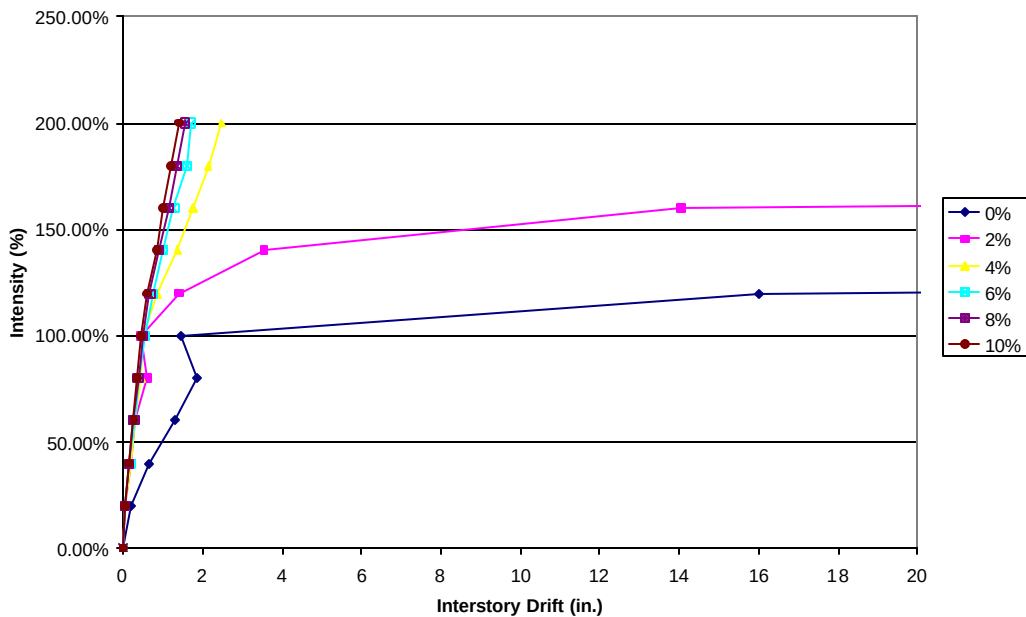


Figure 7.4(b) IDA Plot of Interstory Drift for the New York Building under Northridge Ground Motion for variable secondary stiffness with initial geometric stiffness

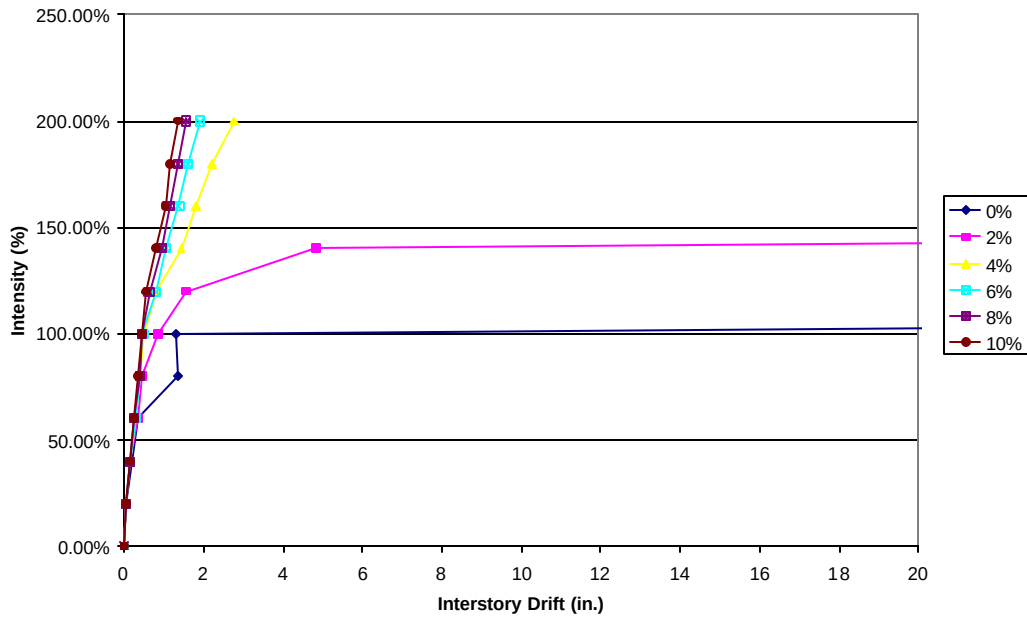


Figure 7.4(c) IDA Plot of Interstory Drift for the New York Building under Northridge Ground Motion for variable secondary stiffness with updated geometric stiffness

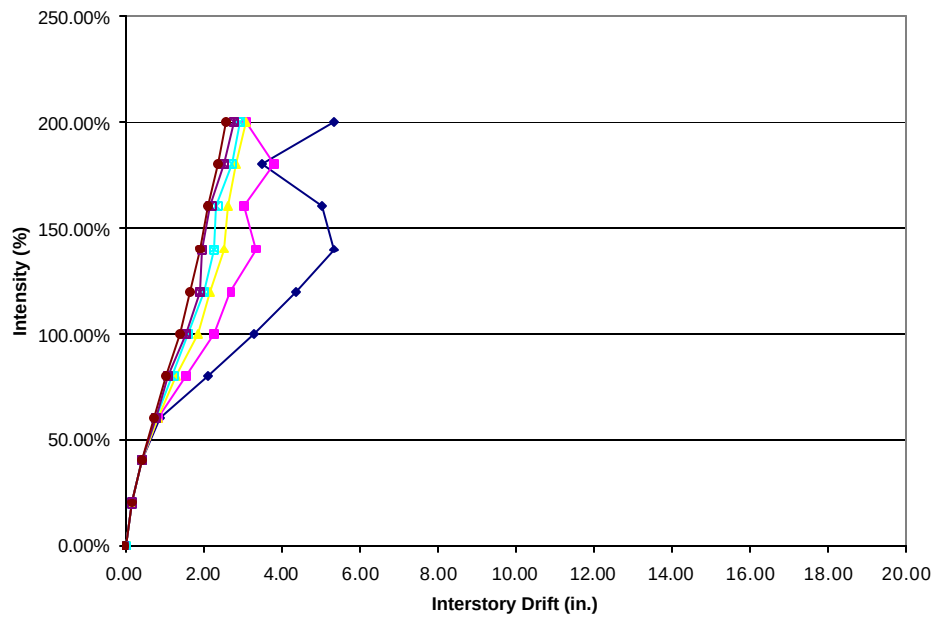


Figure 7.5(a) IDA Plot of Interstory Drift for the Charleston Building under Loma Prieta Ground Motion for variable secondary stiffness without considering geometric stiffness

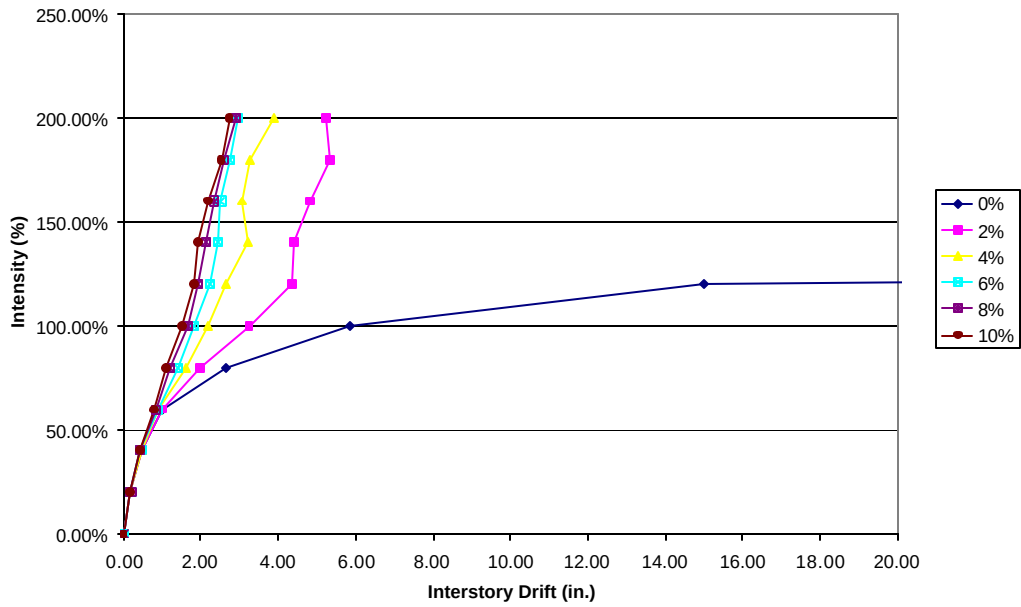


Figure 7.5(b) IDA Plot of Interstory Drift for the Charleston Building under Loma Prieta Ground Motion for variable secondary stiffness with initial geometric stiffness

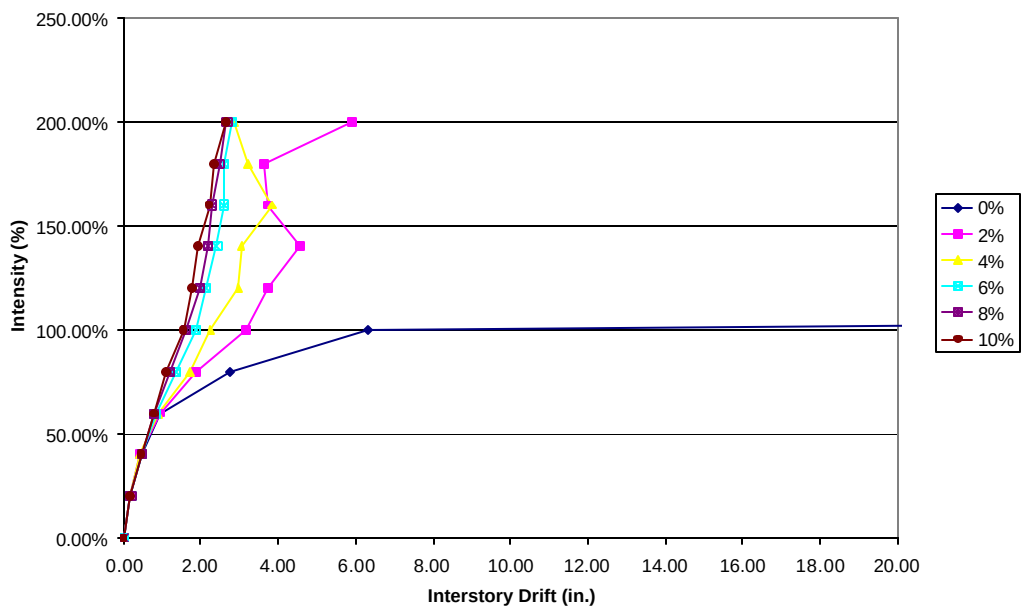


Figure 7.5(c) IDA Plot of Interstory Drift for the Charleston Building under Loma Prieta Ground Motion for variable secondary stiffness with updated geometric stiffness

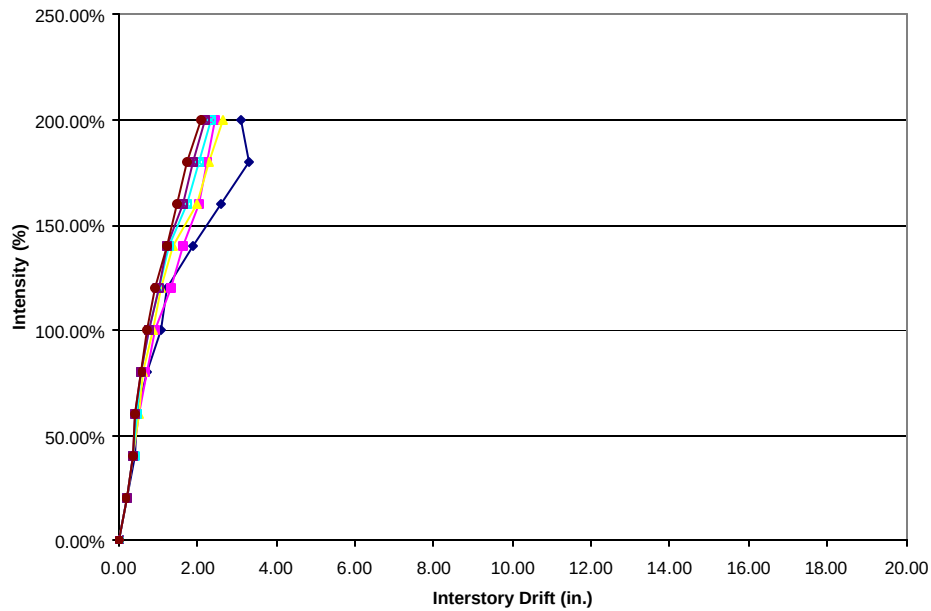


Figure 7.6(a) IDA Plot of Interstory Drift for the Charleston Building under Northridge Ground Motion for variable secondary stiffness without considering geometric stiffness

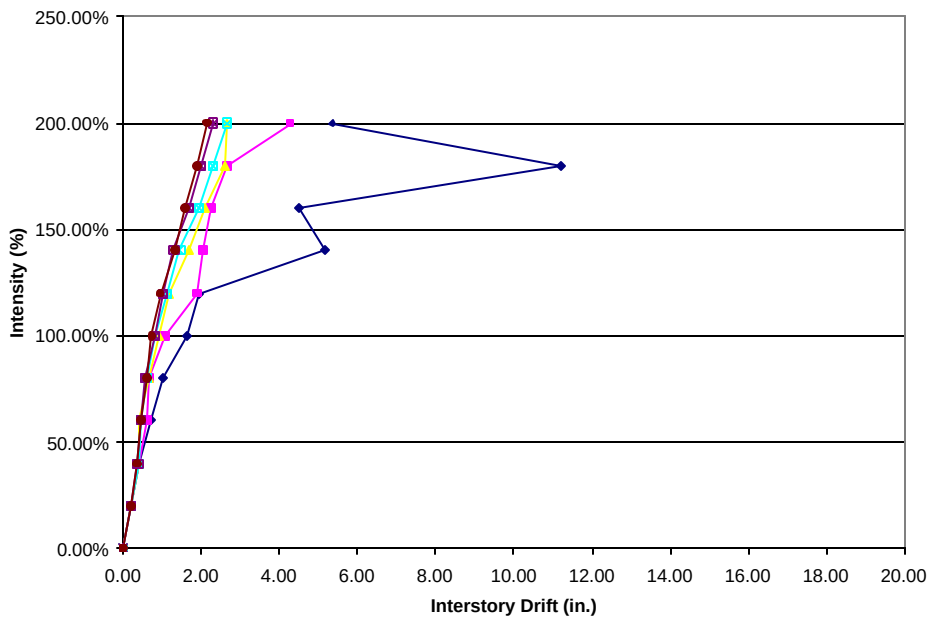


Figure 7.6(b) IDA Plot of Interstory Drift for the Charleston Building under Northridge Ground Motion for variable secondary stiffness with initial geometric stiffness

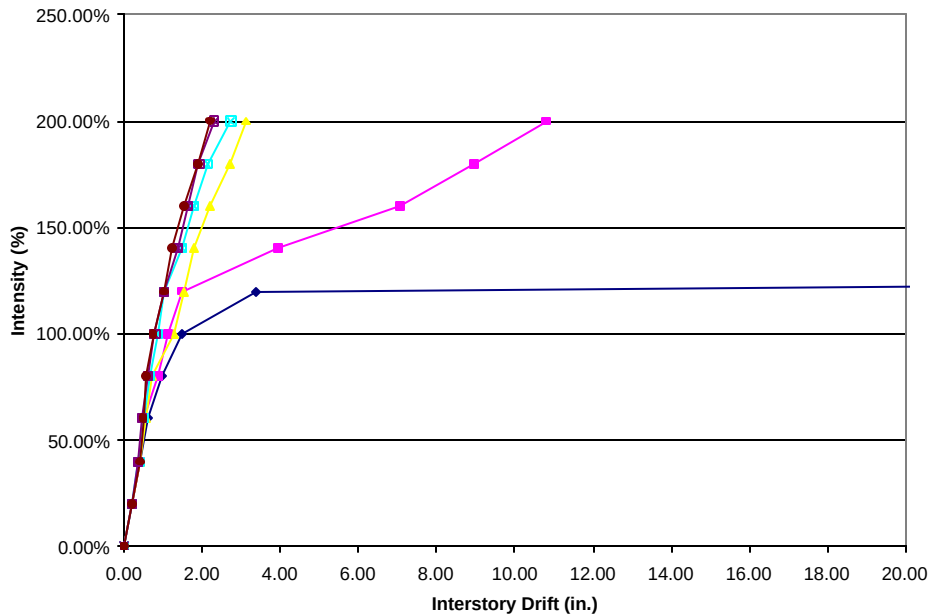


Figure 7.6(c) IDA Plot of Interstory Drift for the Charleston Building under Northridge Ground Motion for variable secondary stiffness with updated geometric stiffness

Based on the IDA curves, several trends were observed. When the geometric stiffness is neglected from the analysis, all structures remain dynamically stable. However, when the geometric stiffness is included, structures with a low post-yield stiffness are likely to become dynamically unstable. This is because when the P-delta effect is included, the geometric stiffness may be higher than the post-yield stiffness, which lowers the total stiffness of the structures and dramatically increases the drift and causes a large interstory drift. When the geometric stiffness is updated, most structures become dynamically unstable with a lower intensity of ground motion when compared with a structure that only includes the initial geometric stiffness.

For the “Berkeley” building, when the ground motion intensity was below approximately 1.5 times the target ground acceleration, the building did not experience any dynamic instability.

For the ‘Charleston’ building, when the ground motion intensity was below the target ground acceleration, the building did not experience any dynamic instability. However, for the ‘New York’ building, the building experienced dynamic instability when the intensity is approximately 50% of the target ground acceleration. This leads to the conclusion that the geometric stiffness is more likely to cause structural collapse in a lower seismic zone.

7.3.2 Variation of Degradation Properties

The degradation rules were discussed in Chapter 3. However, the real tested model is required to input realistic parameters for both stiffness and strength degradation. Reinhorn et al. (1999) presented the range of each parameter for modeling deteriorating inelastic structures that had moderate to severe degradations. The values table for the degradation parameters are shown in Table 7.8 (Sivaselvan and Reinhorn, 1999).

Degradation	Mild	Moderate	Severe
α	15	10	4
β_1	0.01	0.3	0.6
β_2	0.01	0.15	0.3

Table 7.11 Range of Parameters (Sivaselvan and Reinhorn, 1999)

In the study of the influence of the degradation properties, the parameters would be similar to the range of parameters given in Table 7.11. Similar to the study of the post-yield stiffness, three models were used to represent structures located in Berkeley, New York, and Charleston.

7.3.2.1 Stiffness Degradation

As discussed in Chapter 3, the stiffness degradation is controlled by “ α ”, the stiffness degradation parameter. In the study of the stiffness degradation, four different values of “ α ” were used. The four different values were 20, 10, 5 and 2. Additionally, the simple bilinear model with the same structural properties was used.

Figure 7.7 and Figure 7.8 show the peak interstory drift for the “Berkeley” building when subjected to Loma Prieta and Northridge ground motions, respectively. Figure 7.9 and Figure 7.10 show the peak interstory drift for the “New York” building when subjected to Loma Prieta and Northridge ground motion, respectively. Figure 7.11 and Figure 7.12 show the peak interstory drift for the “Charleston” building when subjected to Loma Prieta and Northridge ground motion, respectively.

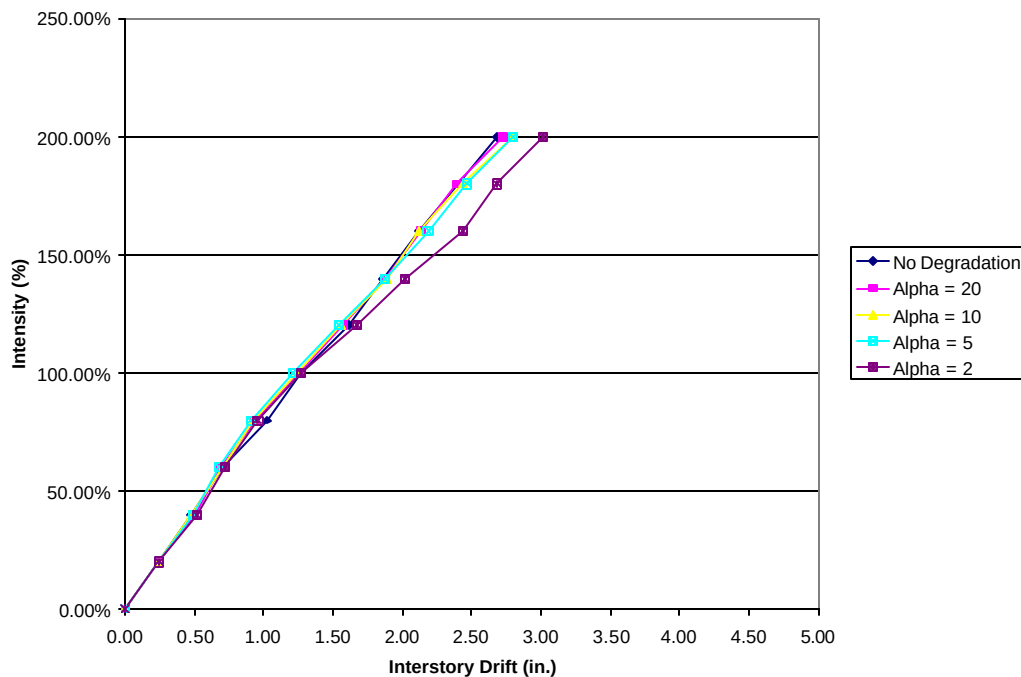


Figure 7.7 IDA Plot of Interstory Drift for the Berkeley Building under Loma Prieta Ground Motion for variable degrading stiffness

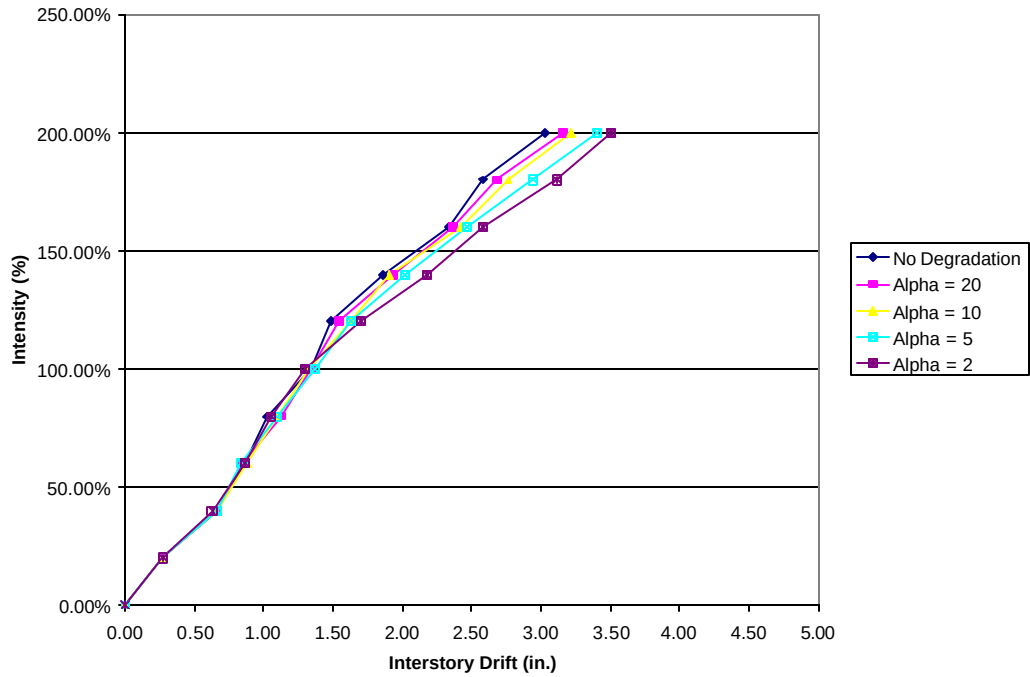


Figure 7.8 IDA Plot of Interstory Drift for the Berkeley Building under Northridge Ground Motion for variable degrading stiffness

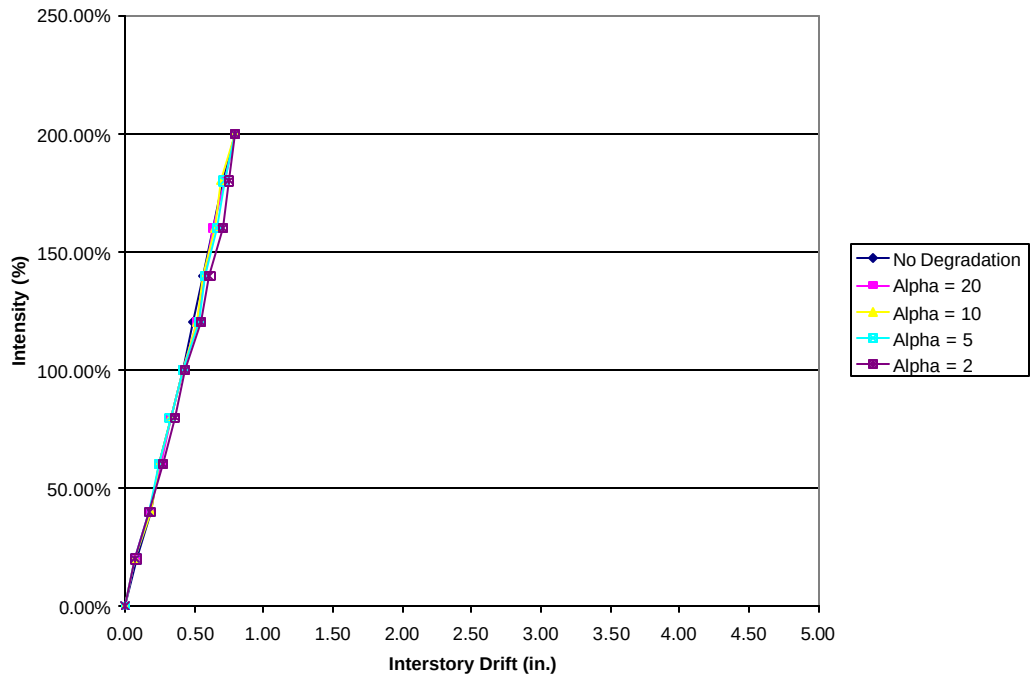


Figure 7.9 IDA Plot of Interstory Drift for the New York Building under Loma Prieta Ground Motion for variable degrading stiffness

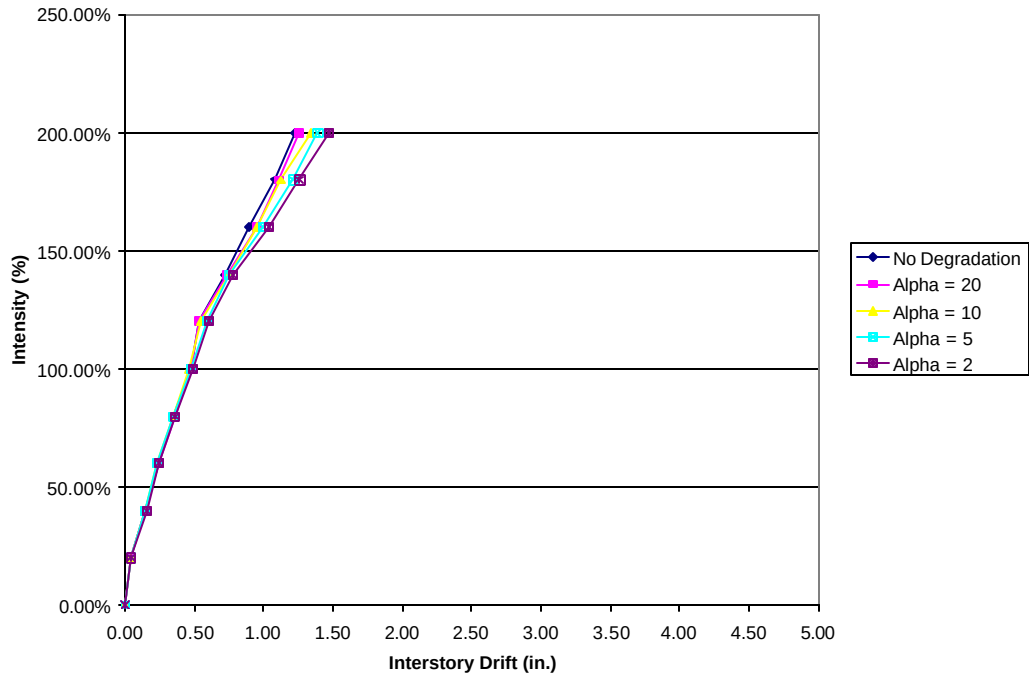


Figure 7.10 IDA Plot of Interstory Drift for the New York Building under Northridge Ground Motion for variable degrading stiffness

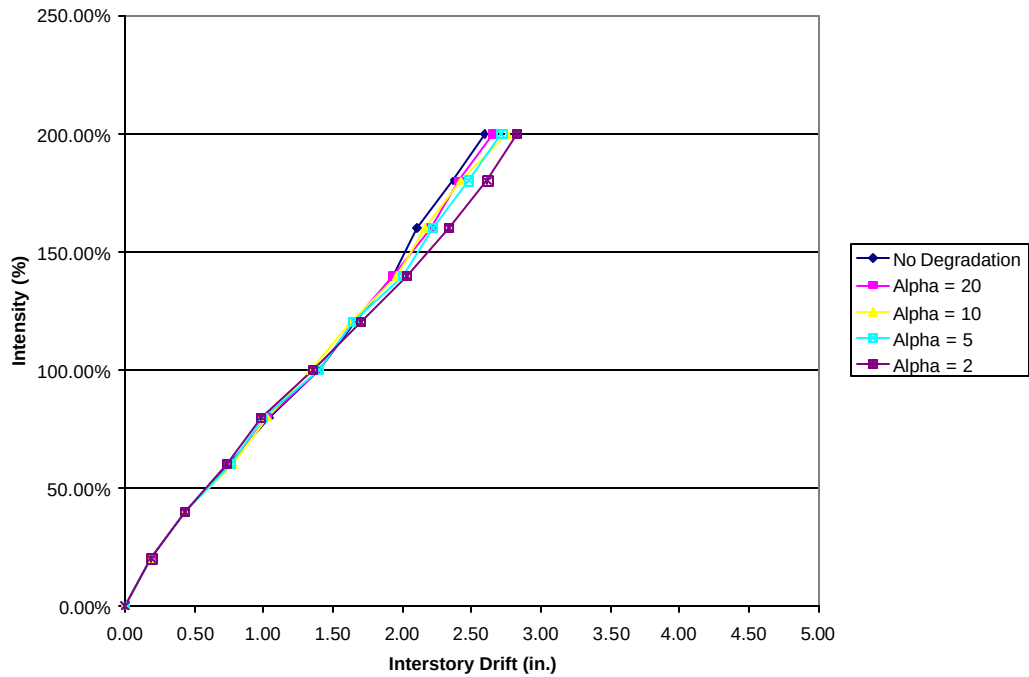


Figure 7.11 IDA Plot of Interstory Drift for the Charleston Building under Loma Prieta Ground Motion for variable degrading stiffness

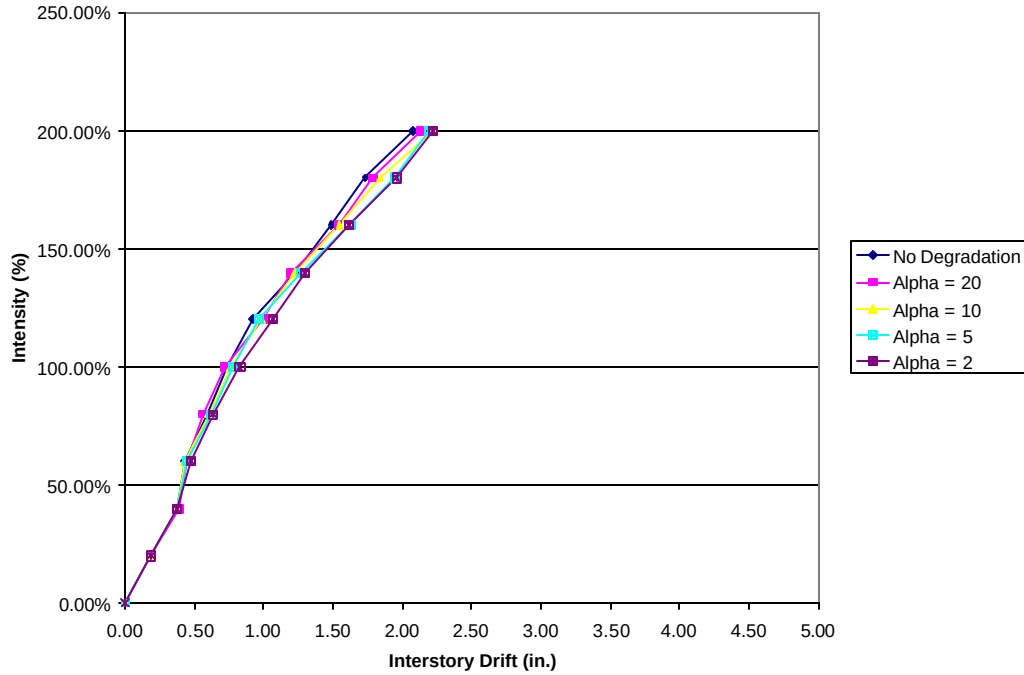


Figure 7.12 IDA Plot of Interstory Drift for the Charleston Building under Northridge Ground Motion for variable degrading stiffness

From the IDA curves, a general trend is observed. The interstory drift increases with the stiffness degradation. The range of increase in the interstory drift is approximately 10% to 20% of the original drift. The effect of including the stiffness degradation is not significant. This is because stiffness degradation only changes the primary stiffness of the elements, when the primary stiffness is usually high.

7.3.2.2 Strength Degradation

Three parameters are used to model the strength degradation behavior. The parameters are “ b_1 ”, “ b_2 ”, and the ultimate ductility. “ b_1 ” and “ b_2 ” are the ductility-based strength degradation parameter and energy-based strength degradation parameter, respectively. To simplify the analysis, the same values will be used for “ b_1 ” and “ b_2 ”. Four different values used

for both “ b_1 ” and “ b_2 ” were 0.01, 0.2, 0.4 and 0.6 that represent low, mild, moderate, and severe strength degradation, respectively. In addition, the simple bilinear model with the same structural properties, which represents no strength degradation, was used.

Figure 7.13 and Figure 7.14 show the peak interstory drift for the “Berkeley” building when subjected to Loma Prieta and Northridge ground motions, respectively. Figure 7.9 and Figure 7.10 show the peak interstory drift for the “New York” building when subjected to Loma Prieta and Northridge ground motions, respectively. Figure 7.11 and Figure 7.12 show the peak interstory drift for the “Charleston” building when subjected to Loma Prieta and Northridge ground motions, respectively.

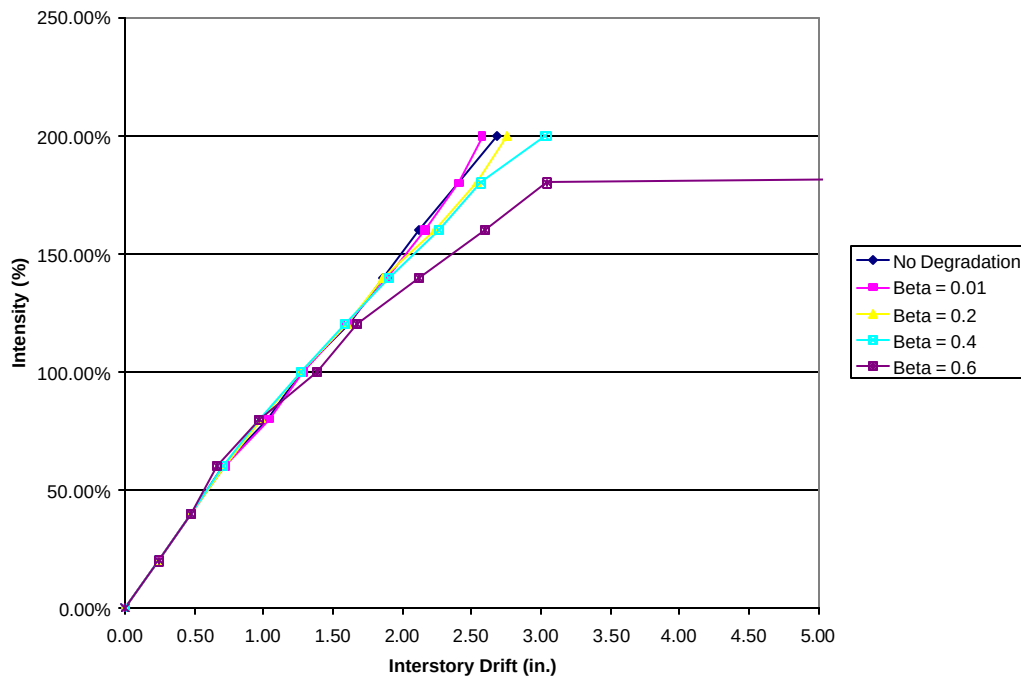


Figure 7.13 IDA Plot of Interstory Drift for the Berkeley Building under Loma Prieta Ground Motion for variable degrading strength

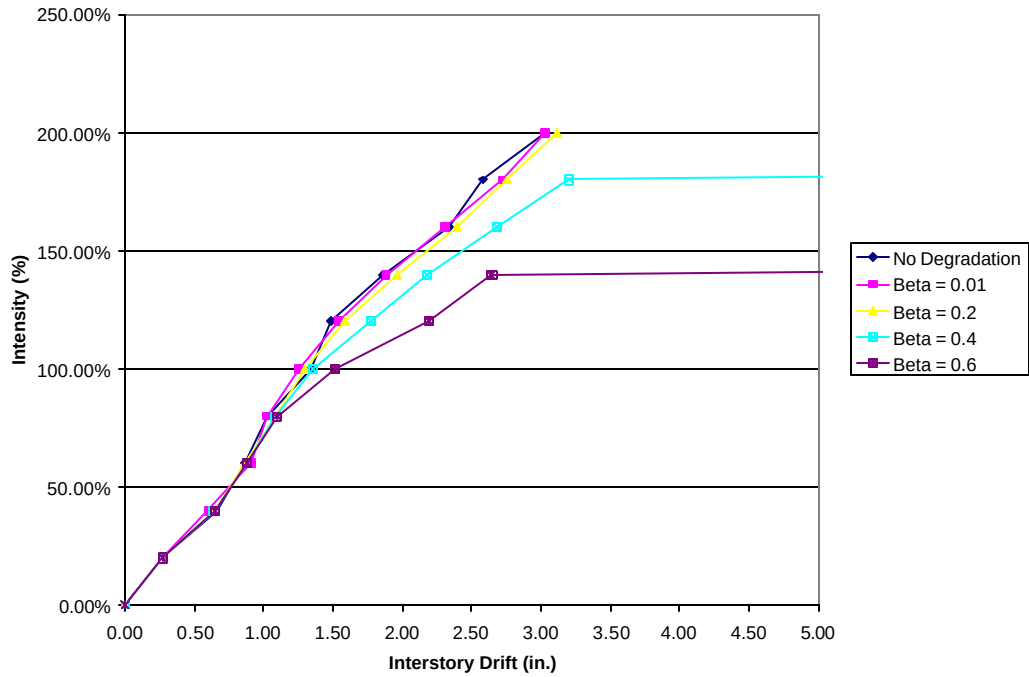


Figure 7.14 IDA Plot of Interstory Drift for the Berkeley Building under Northridge Ground Motion for variable degrading strength

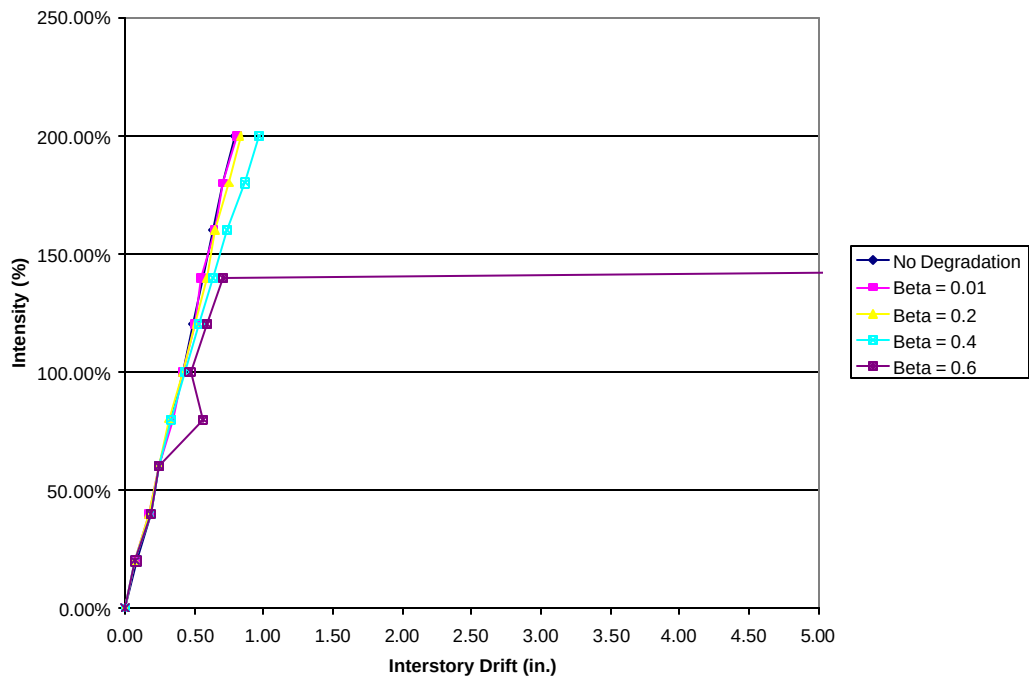


Figure 7.15 IDA Plot of Interstory Drift for the New York Building under Loma Prieta Ground Motion for variable degrading strength

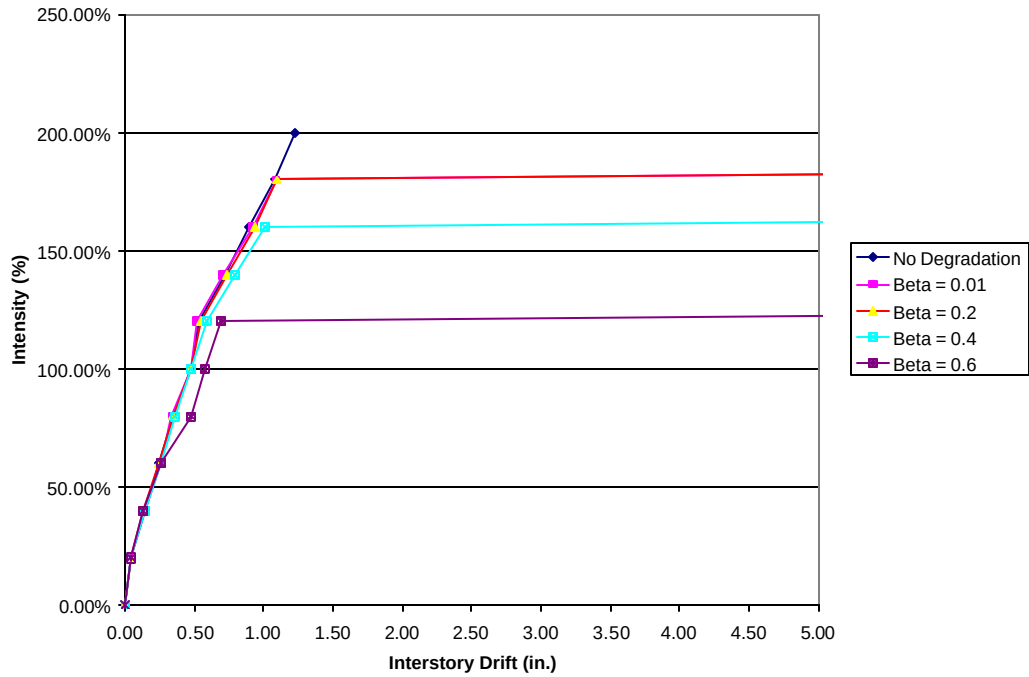


Figure 7.16 IDA Plot of Interstory Drift for the New York Building under Northridge Ground Motion for variable degrading strength

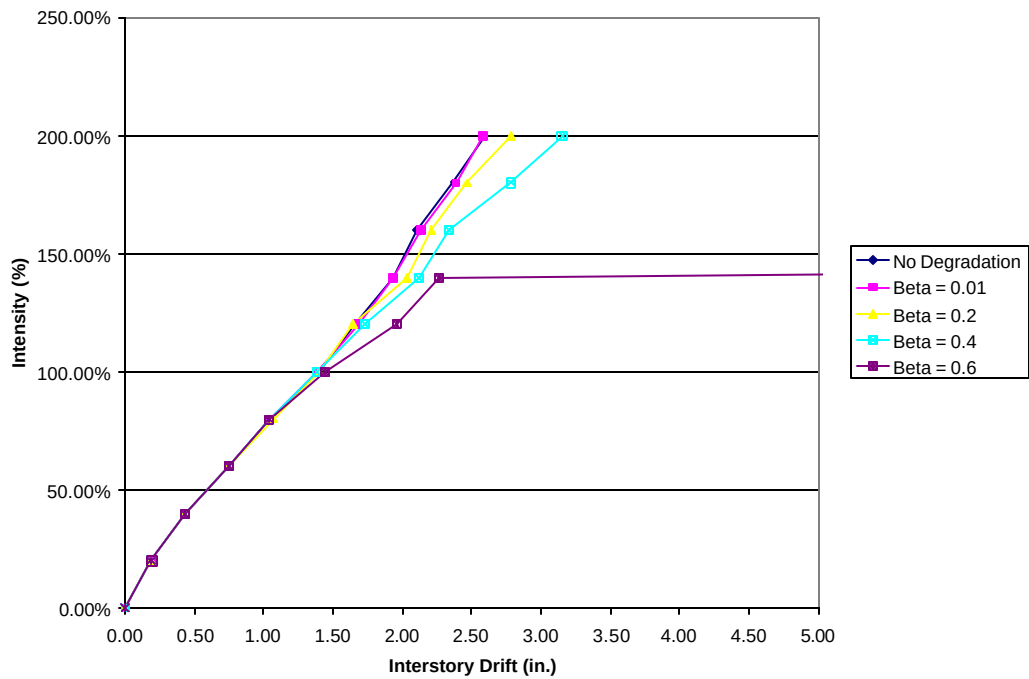


Figure 7.17 IDA Plot of Interstory Drift for the Charleston Building under Loma Prieta Ground Motion for variable degrading strength

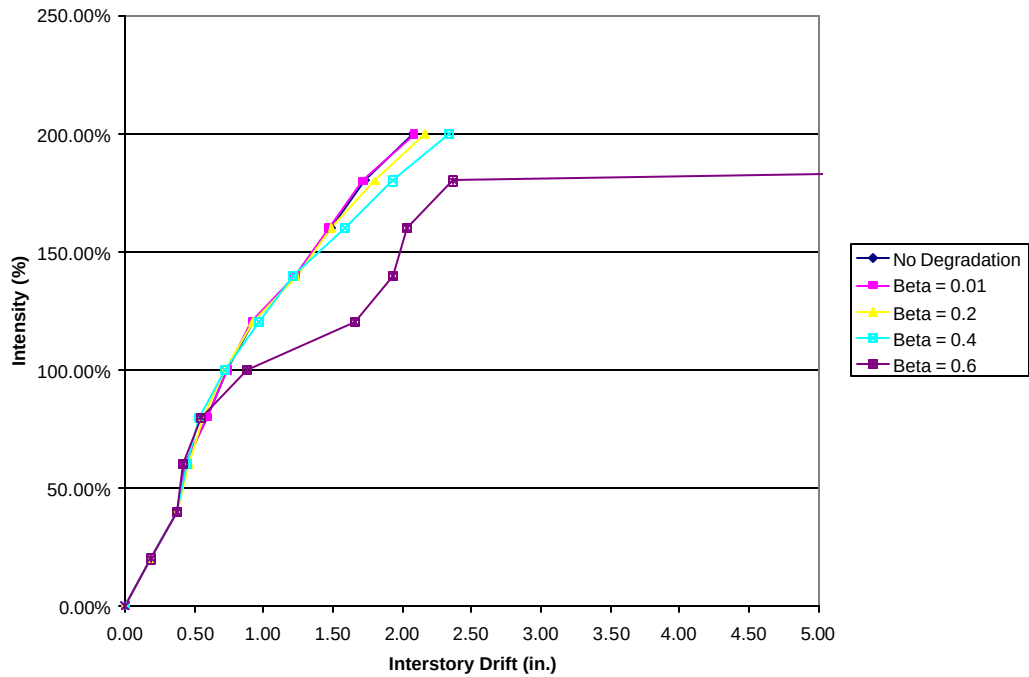


Figure 7.18 IDA Plot of Interstory Drift for the Charleston Building under Northridge Ground Motion for variable degrading strength

In general, the interstory drift increases with the strength degradation. For severe strength degradation parameters ($b = 0.6$), all structures experienced dynamic instability at 200% of the target acceleration. The “New York” building experienced dynamic instability for low to severe strength degradation. This is because the design strength for the “New York” building is relatively low, compared with the “Berkeley” and “Charleston” buildings.

Chapter 8 Conclusions

8.1 Description of the procedures

This study was set up to determine the effects of geometric stiffness and the degradation properties on the structural response of three buildings located in Berkeley, CA, New York, NY, and Charleston, SC. In this study, incremental dynamic analysis was chosen because it can provide a good evaluation of structural damage under scaled ground motions. Ground motions were scaled so that the pseudo acceleration for an elastic SDOF structure, which had the same period of vibration at the fundamental period of the multistory building located in the particular site, was equal to the design spectral acceleration. Two different programs, NONLIN Version 8 and SAP 2000, were used for performing the analysis. The results obtained from the software were exported to Microsoft Excel so that graphical results could be shown.

The system that was used was a five-story moment frame office building. Special Moment Frame, Intermediate Moment Frame, and Ordinary Moment Frame were used for the “Berkeley”, “New York”, and “Charleston” buildings, respectively. The structural properties were determined using the NEHRP Provisions (FEMA 2000b).

8.2 Results

8.2.1 Variation in post-yield stiffness

In the study of the effect of variation of the post-yield stiffness, three different methods of handling geometric stiffness were used. One method neglected geometric stiffness, one included the initial geometric stiffness, and one updated the geometric stiffness at every time

step. For the first two methods, NONLIN was used for the analysis. For the last method, SAP 2000 was used. The post yield stiffness ratios used for this study were 0%, 2%, 4%, 6%, 8% and 10% of the primary stiffness. Single-record IDA curves were obtained for two different ground motions. Through these analyses, the following conclusions were reached:

1. Systems with greater post-yield stiffness always show smaller response in the inelastic region.
2. When the initial geometric stiffness is included, systems with 0% post-yield stiffness tend to become dynamically unstable. However, for systems that have greater secondary stiffness, the inclusion of initial geometric stiffness sometimes reduces the response.
3. When the geometric stiffness is updated in every time step, systems that experience dynamic instability with initial geometric stiffness always become dynamically unstable at lower intensities.
4. Buildings in a high seismic zone are less likely to experience dynamic instability than buildings in a low seismic zone under the design spectral acceleration of the specific location.

8.2.2 Variation in degradation properties

The effects of degradation in stiffness and strength were investigated separately. Since an actual model was required to provide accurate parameters for the degradation properties, empirical parameters provided by Reinhorn et al. (1999) were used as the range for determining the degradation parameters. The post-yield stiffness used in the study of the degradation properties was 10% of the primary stiffness.

8.2.2.1 Degradation in stiffness

Five degradation models for stiffness, which represented no degradation, low, mild, moderate, and severe, were used. The following conclusions were drawn from the analysis:

1. The interstory drift increases with the degradation level. The range of the increase in the interstory drift is approximately 10% to 20% of the drift with no degradation.
2. In some situations, the interstory drifts were reduced with increased level of stiffness degradation.
3. The stiffness degradation does not have a big influence on the structural response.

8.2.2.2 Degradation in strength

Similar to the degradation in stiffness, five models were used again to represent no, low, mild, moderate, and severe degradations. Several observations were noted:

1. In general, the interstory drift increases with the level of strength degradation.
2. All structures become dynamically unstable when there is a severe strength degradation at two times the target accelerations.
3. Buildings located in New York are likely to experience dynamic instability at a lower level of strength degradation.

8.3 Summary

Based on the results from the analysis, the following conclusions can be drawn for the multistory system subjected to incremental dynamic analysis:

1. The inclusion of geometric stiffness increases the structural response.

2. When the vertical acceleration is included in the analysis, buildings in a low seismic zone are more likely to experience dynamic instability than buildings in a higher seismic zone.
3. Stiffness degradation does not have a big effect on structural response. This is the least sensitive factor in the parameter study.
4. Strength degradation increases the interstory drift. For buildings located in a low seismic zone, strength degradation often caused dynamic instability.

8.4 Limitations

The limitations of this study include:

1. Only two ground motion records were used.
2. The range of variability of a particular parameter was limited. Only secondary stiffness, degradation strength, and degradation stiffness were used as the variable parameters.
3. The base shear and the spring force were not recorded.
4. Mass is lumped at the beam-column joints. In reality, mass is spread along the members, therefore the model may not truly represent a real structure. This is particularly important when vertical acceleration is considered.
5. Only a five story model was used.

8.5 Recommendation for future research

In this study, only two ground motions were used. Ideally, for incremental dynamic analysis, a greater number of ground motions would increase the confidence of the trend observed. In addition, two vertical ground motions are not enough to see the influence of vertical

acceleration on the structural response. Therefore, it is essential to extend this study with more ground motion records.

The study should also be extended to analysis of a variety of number of degrees of freedom. The response of the structure relies heavily on the fundamental period of the structure. Therefore, it is important to have a model that can represent more variety of structures.

In the study of the variation in degradation properties, the parameters used in the study do not have a relationship to the reality performance of structures. Therefore, it is important to obtain some real test data to find out the actual values of parameters for different kinds of structural systems.

Finally, it is important to determine why NONLIN and SAP give different answers when vertical accelerations are included.

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Appendix A – Ground Accelerations

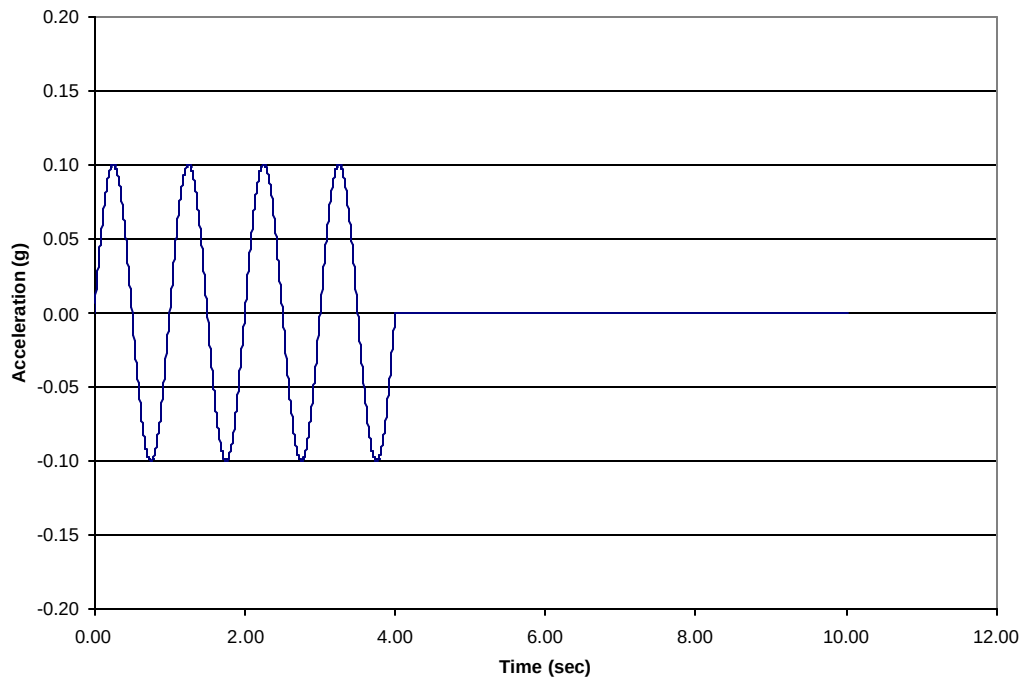


Figure A1 Harmonic Ground Motion (Vertical and Horizontal)

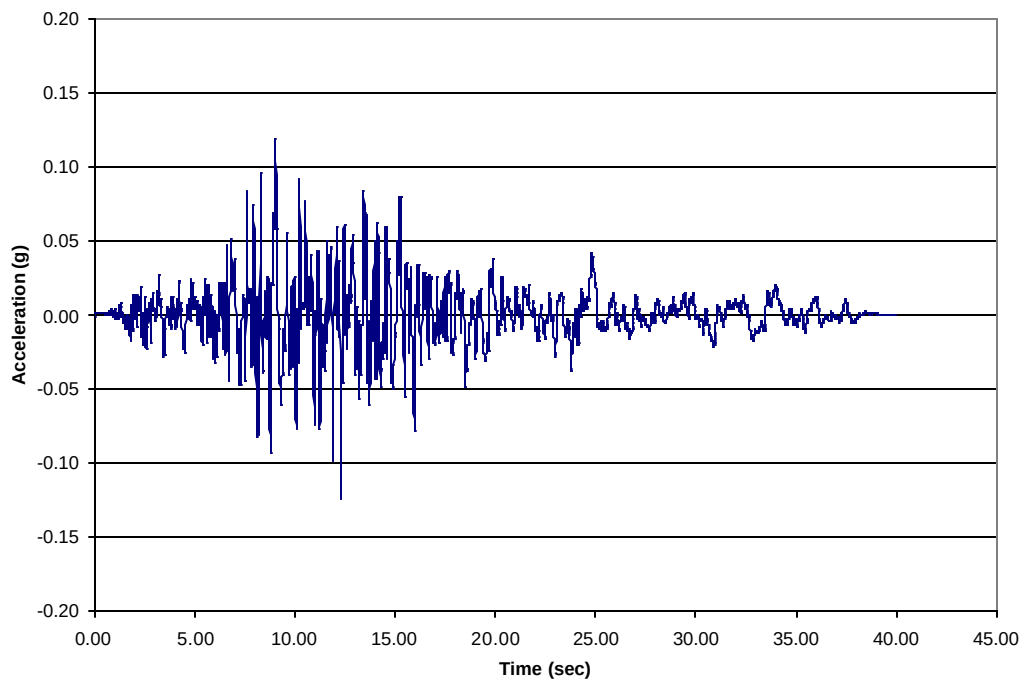


Figure A2(a) Loma Prieta Horizontal Acceleration

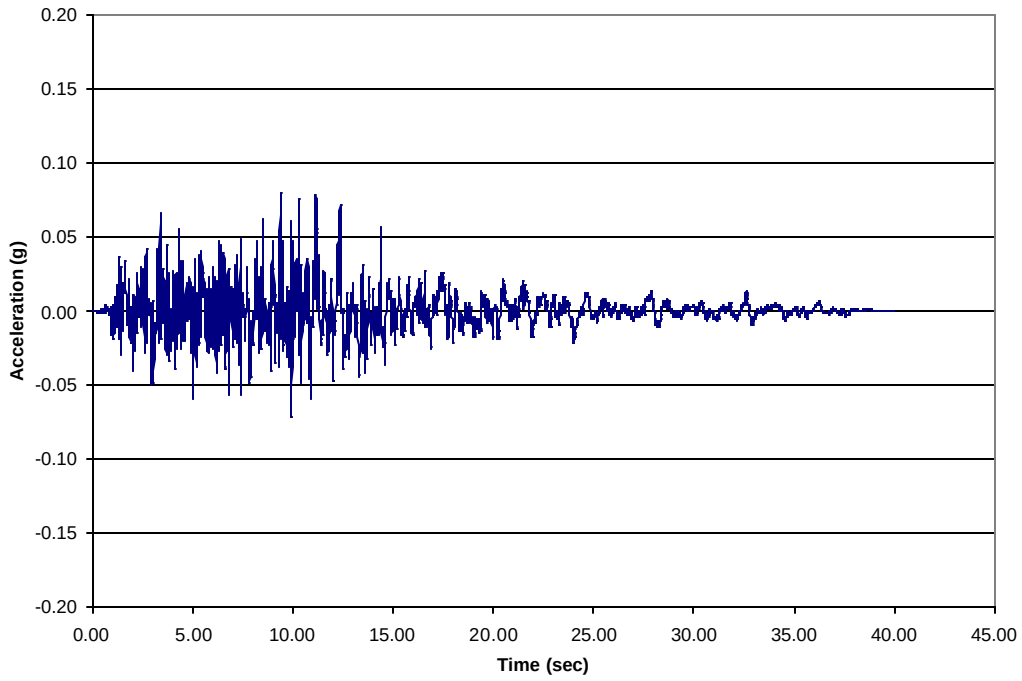


Figure A2(b) Loma Prieta Horizontal Acceleration

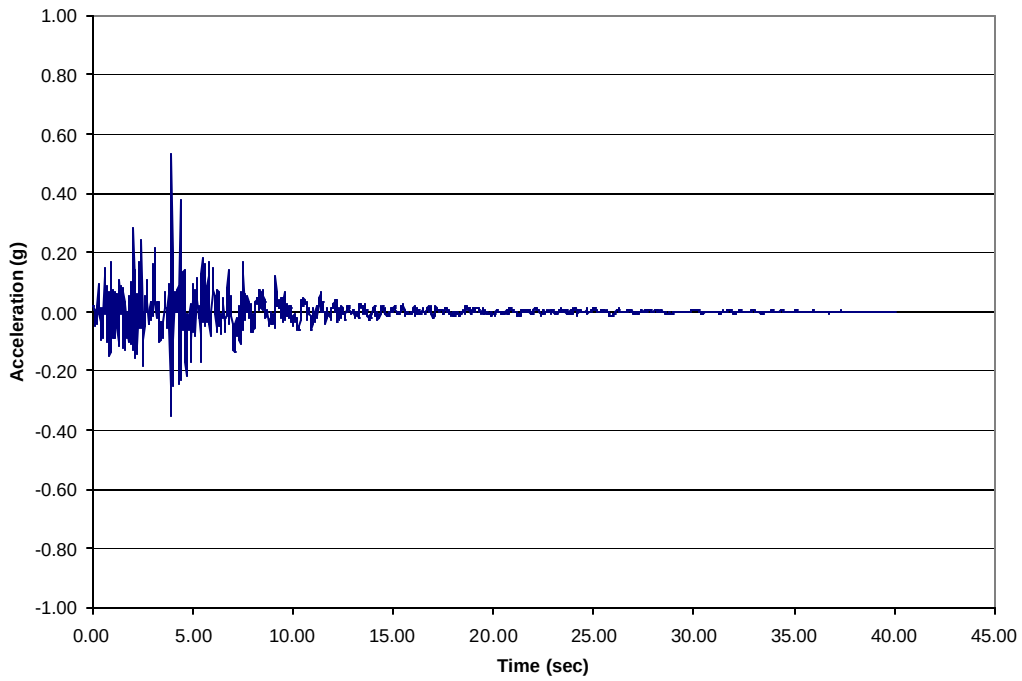


Figure A3(a) Northridge Horizontal Acceleration

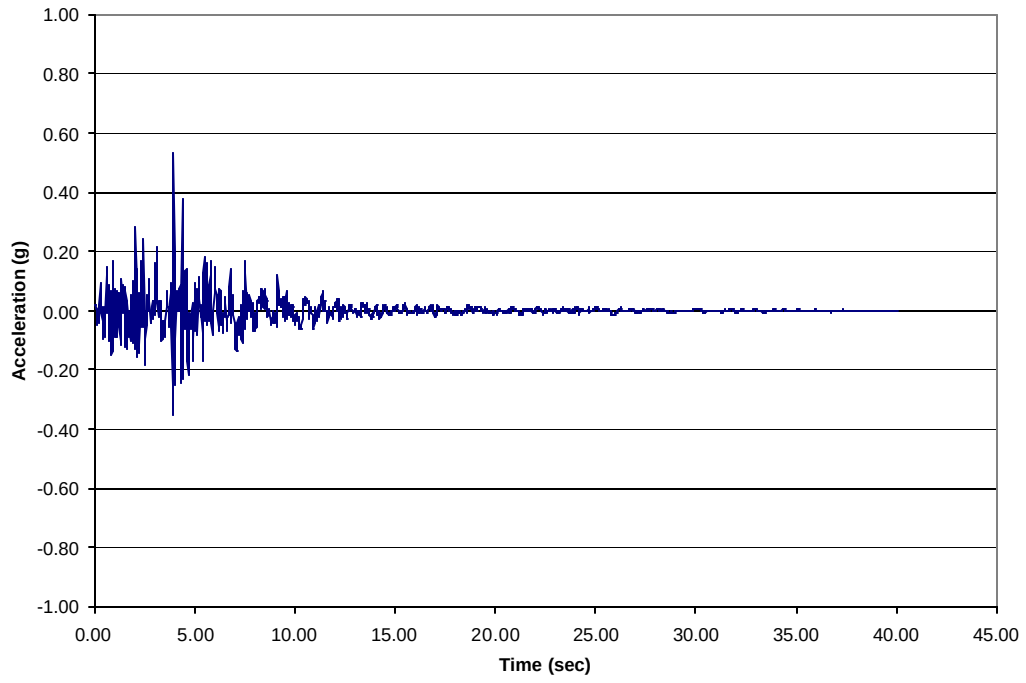


Figure A3(b) Northridge Horizontal Acceleration

Appendix B – Seismic Coefficients and Design Spectral Accelerations

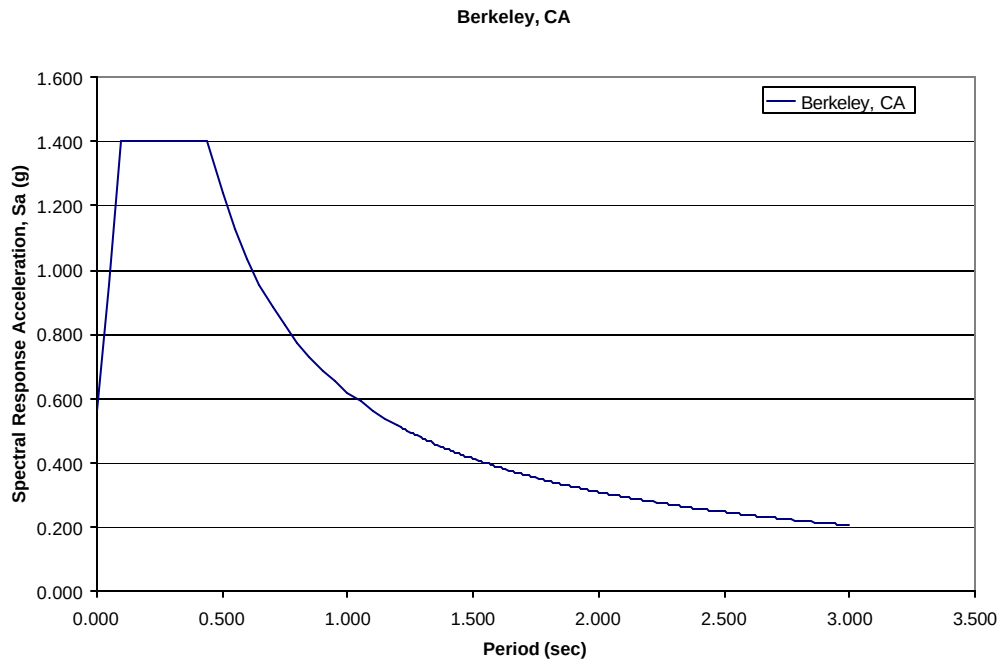


Figure B1 Spectral Response Acceleration for Berkeley, California

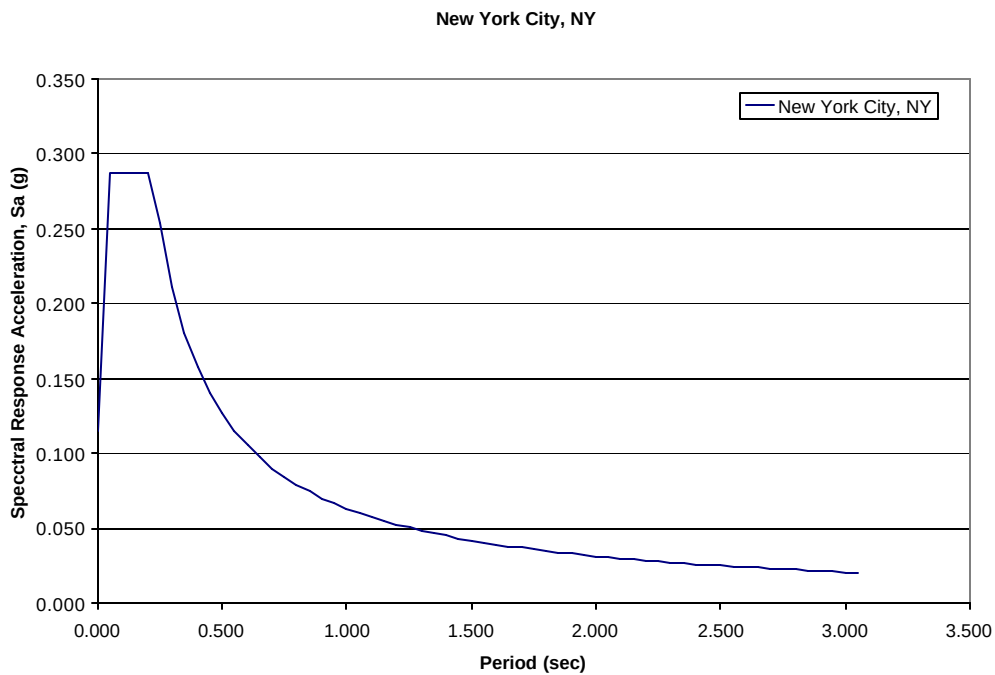


Figure B2 Spectral Response Acceleration for New York, New York

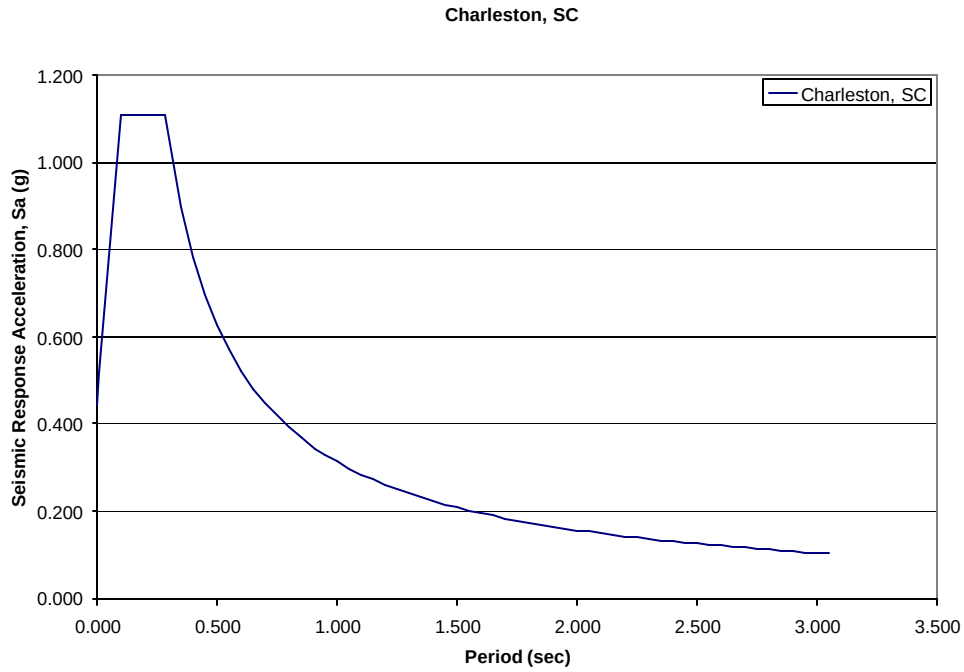


Figure B3 Spectral Response Acceleration for Charleston, South Carolina

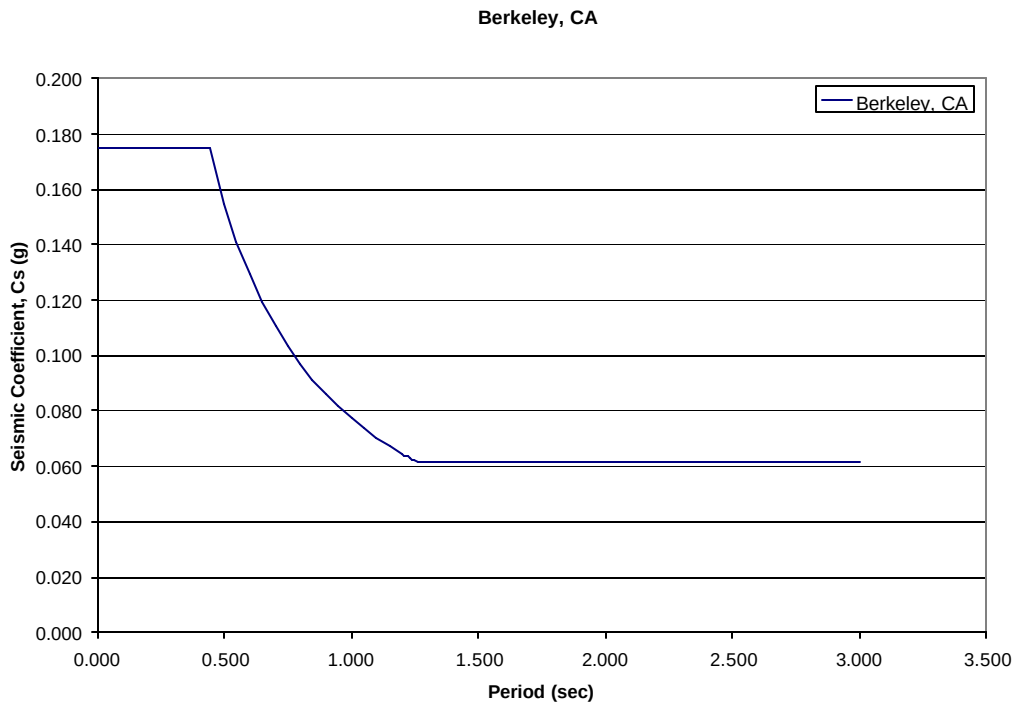


Figure B4 Seismic Coefficient for Berkeley, California

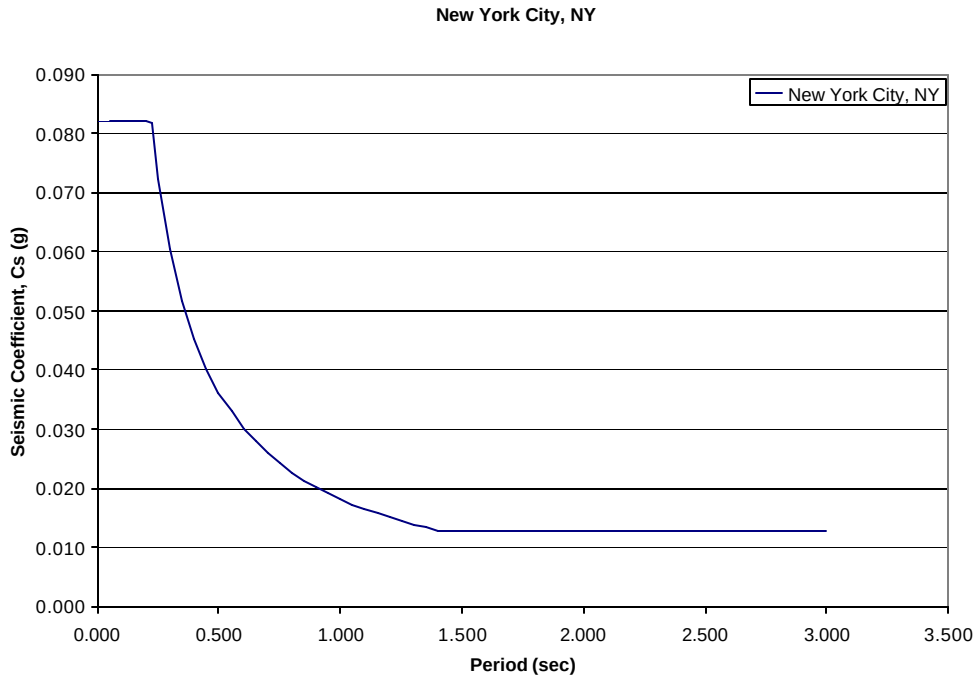


Figure B5 Seismic Coefficient for New York, New York

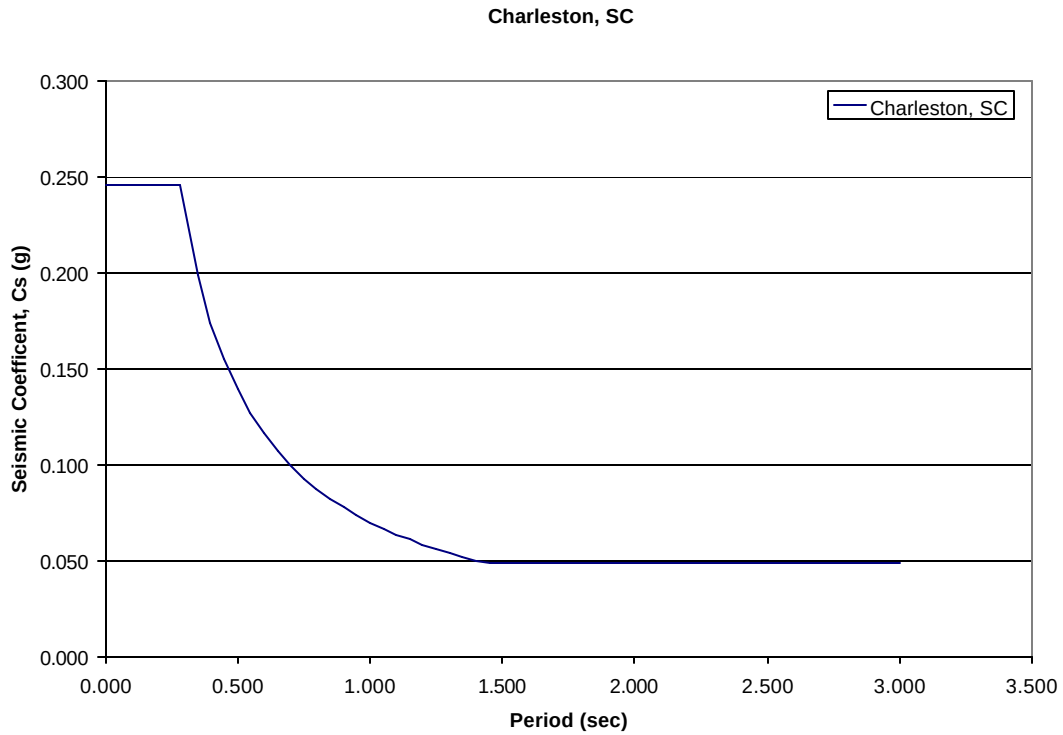


Figure B6 Seismic Coefficient for Charleston, South Carolina

VITA

(February 2005)

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