Performance based design of degrading structures

Mouchir Chenouda

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Performance Based Design of Degrading Structures

by

Mouchir Chenouda

A dissertation submitted in partial fulfillment
of the requirements for the degree of
Doctor of Philosophy
Department of Civil and Environmental Engineering
College of Engineering
University of South Florida

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DEDICATION

To all my dear family members specially my wife Mirey and my son Mark.
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Seismic code provisions are now adopting performance-based methodologies, where structures are designed to satisfy multiple performance objectives. Most codes rely on approximate methods to predict the desired seismic demand parameters. Most of these methods are based on simple SDOF models, and do not take into account neither MDOF nor degradation effects, which are major factors influencing structural behavior under earthquake excitations. More importantly, most of these models can not predict collapse explicitly under severe seismic loads. This research presents a newly developed model that incorporates degradation effects into seismic analysis of structures. A new energy-based approach is used to define several types of degradation effects. The research presents also an evaluation of the collapse potential of degrading SDOF and MDOF structures. Collapse under severe seismic excitations, which is typically due to the formation of structures mechanisms amplified by P-Delta effects, was modeled in this work through the degrading hysteretic structural behavior along with P-Delta effects due to gravity loads. The model was used to conduct extensive statistical dynamic analysis of different structural systems subjected to a large set of recent earthquake records. To perform this task, finite element models of a series of generic SDOF and MDOF
structures were developed. The degrading hysteretic structural behavior along with P-Delta effects due to gravity loads proved to successfully replicate explicit collapse. For each structure, collapse was investigated and inelastic displacement ratios curves were developed in case collapse doesn’t occur. Furthermore, seismic fragility curves for a collapse criterion were also developed. In general, seismic fragility of a system describes the probability of the system to reach or exceed different degrees of damage. Earlier work focused on developing seismic fragility curves of systems for several values of a calibrated damage index. This research work focuses on developing seismic fragility curves for a collapse criterion, in an explicit form. The newly developed fragility curves represent a major advancement over damage index-based fragility curves in assessing the collapse potential of structures subject to severe seismic excitations. The research findings provide necessary information for the design evaluation phase of a performance-based earthquake design process.
PREFACE

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CHAPTER 1
INTRODUCTION

1.1 Introduction

The seismic design provisions of building codes in the United States are moving towards adopting the general concept of performance based design. A Performance Based Earthquake Engineering (PBEE) design process is a demand/capacity procedure that incorporates multiple performance objectives. The procedure consists of four main steps. In the first step, performance objectives of a structural system at different hazard levels are defined. In the second step, a conceptual design of the structure is performed in order to meet the objectives defined in step 1. A design evaluation phase is then needed in order to evaluate the conceptual design developed in step 2. Finally a socio-economic study is needed to finalize the process. In the design evaluation phase, seismic demands of the structure need to be evaluated as accurately as possible at different hazard levels for demand/capacity comparison. Most codes rely on approximate methods that predict the desired demand parameters; the most common two are the method of coefficients and the capacity spectrum method. Most of these methods are based on simple SDOF models, and do not take into account neither MDOF nor degradation effects, which are major factors influencing structural behavior under earthquake excitations. More importantly, most of these models can not predict collapse explicitly under severe seismic
loads. This research proposes a newly developed model that incorporates degradation effects into seismic analysis of structures. This degrading structural behavior is essential for accurate investigation of structural behavior and for collapse assessment of structures subject to severe seismic excitations. A new energy-based approach is used to define several types of degradation effects for different material models. Collapse under severe seismic excitations, which is typically due to the formation of structures mechanisms amplified by P-Delta effects, was modeled in this work through the degrading hysteretic structural behavior along with P-Delta effects due to gravity loads. The new degrading model was used to conduct extensive statistical dynamic analysis of different structural systems subjected to a large set of recent earthquake records with the goal of predicting their maximum inelastic deformations and investigating their potential for collapse. To perform this task, finite element models of a series of generic SDOF and MDOF structures were developed. The structures covered a wide range of periods, yield values, and levels of degradation. The degrading hysteretic structural behavior of the structural elements along with P-Delta effects due to gravity loads proved to successfully replicate explicit collapse. Inelastic displacement ratios and seismic fragility curves for a collapse criterion were developed. The research findings proved to provide necessary information for the design evaluation phase of a performance-based earthquake design process.

1.2 Research Objectives

The main goal of the research study is to investigate the behavior of degrading structural systems and their potential for collapse under seismic excitations. The study is essential for the design evaluation phase of a performance-based earthquake design
process, particularly for collapse prevention limit states. To accomplish the research objectives, it is necessary to develop a new numerical procedure for predicting maximum inelastic displacements, and for estimating collapse of degrading structures under seismic excitations. Several constitutive material models including both static and dynamic degradation effects were developed. The models include a strength softening branch to model static degradation under monotonic loads. In addition, the models incorporate four types of cyclic degradation: strength degradation, unloading stiffness degradation, accelerated stiffness degradation, and cap degradation. The models were added to the element library of the non-linear frame analysis program DRAIN-2DX. The degradation parameters were calibrated versus experimentally tested specimens of concrete, steel and timber structures. The work consisted of conducting statistical analytical studies on a large ensemble of degrading structural systems, and using a large suite of earthquake records representing recent events. Both SDOF and MDOF systems were investigated. In addition, several other parameters were investigated such as yield forces, material model types, and levels of degradation. The results were used to predict maximum inelastic displacements of degrading structures, and to investigate the collapse probability of structures under earthquake excitations through seismic fragility analysis. The findings proved essential in providing the necessary background for evaluation and modification of current seismic design codes to reflect the effect of degradation and potential for collapse.
1.3 Thesis Organization

The report is organized as follows:

*Chapter 1* presents an introduction and brief summary of the research objectives and scope of work.

*Chapter 2* presents a discussion of previous research work on seismic analysis procedures, methods for displacement estimates, and damage evaluation of structures subject to earthquake excitations. A review of current seismic design guidelines is also introduced.

*Chapter 3* presents a detailed description of the new degrading material models. Formulation of the energy criterion used to account for the hysteretic degrading behavior is presented. Calibration of the degradation parameters versus experimentally tested specimens is conducted. Evaluation of the effect of degradation on the inelastic behavior of SDOF systems is performed. The chapter also presents the database of earthquake records used to conduct the analytical studies. A discussion on the scaling effect of records is also presented.

*Chapter 4* presents the statistical analytical studies conducted on the degrading SDOF systems. The dynamic properties of the SDOF systems and the different variables evaluated in the statistical studies are defined. Inelastic displacement ratio curves are developed for degrading systems and compared to non-degraded ones. Incremental dynamic analysis is performed and ductility capacities are estimated. Seismic fragility curves for a collapse criterion are also developed, and conclusions regarding the collapse potential of the systems are drawn.
Chapter 5 presents the statistical analytical studies conducted on the degrading MDOF systems. A detailed description of the MDOF systems and their dynamic characteristics is presented. Description of the inelastic model assumptions is also presented. The effect of higher modes and P-Delta on the degrading behavior of MDOF systems is presented through plots of MDOF displacement ratios. Seismic fragility analysis for a collapse criterion is also conducted for buildings with different number of stories. Conclusions regarding the degrading behavior of MDOF systems and their collapse potential are drawn.

Finally, Chapter 6 presents a summary of the work conducted, and a discussion of the main conclusions drawn. The chapter also offers recommendations for future research work in the field.
CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

This chapter summarizes previous research work in seismic analysis and design of building structures. The extensive research in this field, which started through the pioneering work of Biot (1933), Housner (1941), and Veletsos and Newmark (1960) has led to the development of several seismic design codes for buildings that are currently used throughout the world. A brief description of the most widely used codes of practice is presented first.

2.2 Strength-Based Building Design Codes

The seismic design section in most of the current building codes necessitates that structures has to be designed using the equivalent static load concept. These static forces assigned at each floor level are function of structure’s properties and seismic zone. The outcome of the analysis is usually shear forces and overturning moments used to design against seismic loads. Recently, modern codes allows for seismic analysis using linear dynamic procedures such as response spectrum analysis and response history analysis. Linear dynamic analysis is required for some special cases, for instance for buildings with long periods or for irregular buildings. Chopra (2005) mentioned that current
California Building Code requires dynamic analysis of hospital structures. The seismic static design provisions in 3 building codes are presented herein after (Chopra 2001 & 2005).

2.2.1  **International Building Code (United States)**

The base shear \( V_b \) in the 2003 edition of the International Building Code (IBC) is specified as:

\[
V_b = \frac{IC}{R} W
\]

(2.1)

Where:

- \( I \) = Importance factor,
- \( C \) = Period-dependant coefficient based on structure location and site class,
- \( R \) = Strength reduction factor or Elastic seismic coefficient when \( R = 1.0 \), and
- \( W \) = Total dead load of the structure.

The lateral forces at each floor are distributed over the structure height using the base shear. The equation used for the \( j^{th} \) floor is the following:

\[
F_j = V_b \frac{w_j h_j^k}{\sum_{i=1}^{N} w_i h_i^k}
\]

(2.2)

Where:

- \( w_i \) = Weight of the \( i^{th} \) floor,
- \( h_i \) = Height of the \( i^{th} \) floor above the base,
- \( k \) = Coefficient depending on the vibration period, and
- \( N \) = Total number of floors.
The design forces of floors and elements are calculated by subjecting the structure to the lateral forces determined from the preceding equations.

### 2.2.2 National Building Code of Canada

The 1995 edition of the National Building Code of Canada specifies the base shear \((V_b)\) as:

\[
V_b = \frac{\nu SIFU}{R} W \tag{2.3}
\]

Where:

\(\nu = \) Zonal velocity ratio varying between 0 and 0.4,

\(S = \) Seismic response factor depending on fundamental natural vibration period and seismic zone,

\(I = \) Seismic importance factor,

\(F = \) Foundation factor depending on the soil category defined in the code,

\(U = \) Overstrength factor,

\(R = \) Force modification factor reflecting design and construction experience, and

\(W = \) Total dead load of the structure.

Similar to the UBC (2003), the lateral forces at each floor are distributed over the structure height using the base shear.

The design forces of story shears are calculated by subjecting the structure to the lateral forces. The overturning moments are multiplied by reduction factor at the structure’s base and each floor.
2.2.3 Mexico Federal District Code

The base shear ($V_b$) in the 1987 edition of the Mexico Federal District Code is calculated as follows:

$$V_b = \frac{C_e W}{Q'}$$  \hspace{1cm} (2.4)

Where:

$C_e$ = Elastic seismic coefficient depending on fundamental period and seismic zone,

$Q'$ = Seismic behavior factor depending on several factors including the structural system and structural materials, and

$W$ = Total dead load of the structure.

The lateral forces at each floor are distributed over the structure height using the base shear. The basic equation used for the $j^{th}$ floor is similar to the previous codes mentioned:

$$F_j = V_b \frac{w_j h_j^k}{\sum_{i=1}^{N} w_i h_i^k}$$ \hspace{1cm} (2.5)

Where:

$w_i$ = Weight of the $i^{th}$ floor,

$h_i$ = Height of the $i^{th}$ floor above the base,

$k$ = Coefficient depending on the vibration period, and

$N$ = Total number of floors.

This equation is modified and separated into two parts depending on the value of the period at the end of the constant pseudo-acceleration region of the design spectrum.

The design forces of floors and elements are calculated by subjecting the structure to the lateral forces previously determined. The overturning moments are multiplied by
reduction factor at structure’s base and each floor to obtain the design values. However, the reduced moments at any floor should not be less than the product of the story shear at that elevation and the distance to the center of gravity of the building portion above the floor elevation considered.

Chopra (2005) concluded from the comparison of the three codes that the base shear is overestimated. The reduction factor used is intended to account for several factors such as the difference between design strength and yield strength, and the performance of different structural systems and materials during precedent earthquakes.

2.3 Performance-Based Building Design Codes

The seismic design provisions of building codes in the United States are moving towards adopting the general concept of performance based design. A Performance Based Earthquake Engineering (PBEE) design process is a demand/capacity procedure that incorporates multiple performance objectives. The procedure consists of four main steps. In the first step, performance objectives of a structural system at different hazard levels are defined:

- Immediate occupancy,
- Life safety, and
- Collapse prevention.

In the second step, a conceptual design of the structure is performed in order to meet the objectives defined in step 1. The third step is a design evaluation phase needed in order to evaluate the conceptual design previously developed in step 2. Finally a socio-economic study is required in the fourth step to finalize the process. In the design
evaluation phase, seismic demands of the structure need to be evaluated as accurately as possible at different hazard levels for demand/capacity comparison.

Moehle (1992) and Priestley (1996) have shown that present criteria for the seismic design of new structures and for the seismic evaluation of existing structures can be significantly improved if they are based on the explicit consideration of lateral deformations demands as the key design parameter rather than based on lateral forces. However, implementation of displacement-based seismic design criteria into structural engineering practice requires simplified analysis procedures to estimate displacement demands imposed on structures by earthquake ground motions (Miranda, 2001).

Miranda (1999) found that recently there has been a growing interest in displacement-based design procedures in which lateral displacement demands are used rather than lateral force demands (Moehle 1992). During preliminary design stages of new buildings, or for a quick seismic evaluation of existing buildings, there is a need for estimating the maximum lateral displacements that can take place in the building subjected to the design earthquake ground motion. The estimation of the maximum roof displacement and maximum interstory drift ratio (IDR) (defined as the ratio of the maximum interstory drift to the interstory height) is helpful in recognizing the required capacities, particularly the required lateral stiffness, in order to reach the desired performance level of the building.

Whittaker et al., (1998) pointed out that even though the basic objective of performance-based earthquake engineering is to construct structures that respond in more reliable behavior during earthquake excitation, many engineers relate performance-based earthquake engineering with overall enhanced performance (i.e., damage control). The
study revealed the fact that damage of structural elements in a building frame can be limited if lateral displacements are controlled to predetermined values for the specific intensity of earthquake excitation. The conclusion drawn out from this fact was that methods to calculate dependable estimates of lateral displacements are needed since the damage control is the essence of performance-based earthquake engineering.

Several researchers developed procedures for estimating maximum inelastic displacements. In most of these studies, the material models used followed simple hysteretic non-degrading rules. Few of these studies considered degradation, but still followed very simple rules. In addition, degradation effects were not based on physical reasoning. Furthermore, none of these studies considered collapse prediction of the structures. A brief summary of earlier studies in this field is given below.

The first research work in this field is the one by Veletsos and Newmark (1960) who analyzed SDOF systems using 3 earthquake records. The models were assumed elasto-plastic. They concluded that in the regions of low frequency, the maximum inelastic deformation is equal to the maximum elastic deformation, which is known as the equal displacement rule. They also concluded that this rule doesn’t hold true for regions of high frequency, where the inelastic displacement considerably exceeds the elastic one.

Shimazaki and Sozen (1984) conducted a similar numerical study on a SDOF system using five different hysteretic models. The models used were either bilinear or of Clough type (1966), and only El Centro earthquake record was used for the analysis. No degradation was considered in their study. In their work, they developed a relation between maximum inelastic displacements and corresponding maximum elastic displacements for different values of strength and period ratios. The conclusion of their
work is that for periods higher than the characteristic period, defined as the transition period between the constant acceleration and constant velocity regions of the response spectra, the maximum inelastic displacement equals approximately the maximum elastic displacement regardless of the hysteresis type used, confirming the equal displacement rule. For periods less than the characteristic period, the maximum inelastic displacement exceeds that of the elastic displacement and the amount vary depending on the type of hysteretic model and on the lateral strength of the structure relative to the elastic strength. Their conclusion was confirmed later by Qi and Moehle (1991).

Miranda (1991, 1993a and 1993b) analyzed over 30,000 SDOF systems using a large ensemble of 124 earthquake ground motions recorded on different soil types. He developed ratios of maximum inelastic to elastic displacements for 3 types of soil conditions. He also studied the limiting period value where the equal displacement rule applies. The material model used in his study is also elasto-plastic. Lately, Miranda and Ruiz-Garcia (2002) evaluated six different methods for predicting maximum inelastic displacements. Four methods are based on equivalent linearization techniques, while two are based on multiplying maximum elastic displacements by modification factors. In all methods, cyclic degradation effects were not considered. Krawinkler and his co-workers (1991, 1993 and 1997) conducted similar studies to the one by Miranda. The material models used were either bilinear, Clough or of pinching type. Degradation effects were included, but in the form of strength degradation only, or stiffness degradation only. Gupta and Kunnath (1998) conducted a similar study on SDOF systems subjected to 15 ground motions. They included degradation effects using a 3 parameters model.
More recently, Whittaker et al. (1998) conducted a numerical study on SDOF systems using 20 earthquake records. They used the Bouc-Wen model (1976) in their analysis and neglected degradation effects. They developed mean and mean + 1 sigma ratio plots of maximum inelastic to elastic displacements for different strength values. Miranda (2000) extended his earlier work, and developed displacement ratio plots for different earthquake magnitudes, epicenter distance, and soil conditions. His study was also on non-degrading SDOF systems. Most recently, Miranda (2001) showed that maximum inelastic displacements could be related to maximum elastic displacements either through inelastic displacement ratios or through strength reduction factors. He also showed that the second method is a first order approximation of the first, and that both methods yield similar results in the absence of variability.

Several studies were also conducted on MDOF systems [e.g. Ayoub and Filippou (1999a, 1999b and 2000), Saiidi and Sozen (1981), Freeman (1978), Fajfar and Fischinger (1988), Qi and Moehle (1991), and Krawinkler (1991 and 1997)]. Most researchers concluded that the demand of MDOF systems could be estimated by appropriate modification of the response of the first mode SDOF of the system. Two methods were established in that sense, the capacity spectrum method developed originally by Freeman (1978) and adopted by the Applied Technology Council ATC-40 (1996), and the method of coefficients developed by Krawinkler (1991) and used by the Federal Emergency Management Agency FEMA-356 (2000). Both methods are similar in the sense that they are based on a nonlinear static push-over of the structure. They are different, however, in the way they estimate the maximum “target” inelastic displacement.
2.3.1 Capacity Spectrum Method

The capacity spectrum method is adopted by ATC-40 and is based primarily on superimposing capacity diagram plots on demand diagram plots, and estimating the target displacement with an iterative procedure using elastic dynamic analyses. The procedure consists of the following:

1. Conducting a push-over analysis to construct a relationship between base shear and roof displacement

2. Converting the push-over curve into a capacity diagram. The capacity diagram represents a relationship between the first mode spectral displacement and spectral acceleration. The first mode spectral displacement could be easily calculated as a function of the roof displacement evaluated in 1 using modal analysis, and the first mode spectral acceleration is a function of the base shear also evaluated in 1.

3. Establishing the elastic response spectrum of the earthquake record of interest, and converting it from the standard period-spectral acceleration form into a spectral displacement-spectral acceleration form. The resulting diagram is referred to as a demand diagram.

4. Superimposing the demand diagram evaluated in 3 on the capacity diagram evaluated in 2. An iterative procedure using dynamic analyses of equivalent linear systems is performed to determine the displacement demand point. The displacement demand point represents the inelastic spectral displacement of the system subject to the record of interest.
5. The demand point is converted back into a target roof displacement value. The target displacement represents the maximum roof displacement due to the earthquake record of interest. The push-over diagram is then repeated up to the specified target displacement in order to estimate all seismic demand parameters.

Several modified versions were introduced to improve the originally developed method. Paret et al. (1996) and Bracci et al. (1997) modified the proposed procedure to account for higher mode effects. WJE (1996), Reinhorn (1997), Fajfar (1999), and Chopra and Goel (1999) further improved the procedure by using inelastic design spectra as defined by Newmark and Hall (1982) rather than elastic spectra. In these later versions, inelastic dynamic analyses are performed but using simple bilinear non-degrading material models.

### 2.3.2 Method of Coefficients

In the method of coefficients adopted by FEMA-356, the target displacement at a specific hazard level is calculated by multiplying the maximum corresponding elastic displacement by a series of coefficients that account for inelastic behavior, higher mode effects, and dynamic second order effects. A static pushover analysis is then conducted for the structure up to the specified maximum displacement in order to estimate the different seismic demand parameters.

Specifically, the target displacement \( \delta_t \) is calculated as follow:

\[
\delta_t = c_0 \times c_1 \times c_2 \times c_3 \times S_o \times \frac{T_e^2}{4\pi^2}
\]  
(2.6)
Where:

\( c_0 = \) Modification factor that accounts for MDOF effects, and is equal to the first mode participation factor at the roof,

\( c_1 = \) Modification factor that accounts for the expected ratio of maximum inelastic to maximum elastic displacements. It is taken as 1.5 for periods less than 0.1 sec. and 1 for periods larger than the characteristic period defined as the period associated with the transition from the constant acceleration segment to the constant velocity segment of the spectrum,

\( c_2 = \) Modification factor that accounts for degradation effects, and is equal to 1.2 for periods larger than the characteristic periods,

\( c_3 = \) Modification factor that accounts for dynamic second-order effects, and is equal to 1 for systems with hardening ratios greater than 5%,

\( S_u = \) The design spectral acceleration, and

\( T_e = \) Effective fundamental period of the structure.

The period and damping-dependent coefficients \( c_0, c_1, c_2, \) and \( c_3 \) were evaluated using statistical studies on representative inelastic structural systems, by comparing their behavior to the corresponding SDOF first mode elastic structure. The selected coefficient values were based on the average values obtained from an ensemble of earthquake records whose average acceleration response spectrum matches the ATC response spectrum for soil type 1.

The factor \( c_2 \) was derived by considering models that degrade only in strength or in stiffness. It also does not account for strength softening behavior.
2.3.3 Drawback of Current Methods

The main drawback of both methods, the capacity spectrum method and the coefficients method, is their inability to accurately estimate maximum inelastic displacements, and to predict failure of individual components of the structure, which might affect the overall response and possibly failure of the entire structure. The reason is that both models use simple numerical procedures in estimating the maximum expected displacement during a specific earthquake excitation. In the capacity spectrum method, only static analysis is performed for non-degrading systems. It is known that any material degrades in strength after reaching its full capacity under static loadings, also known as strength softening, which subsequently causes failure. Also, any material degrades in strength and stiffness under repeated cyclic loadings, which might cause complete loss of strength and possibly dynamic material failure. Since the capacity spectrum method considers only non-degrading systems and neglects dynamic effects, it fails to predict failure accurately. The coefficients method also is mainly based on static analysis, but dynamic effects are introduced by a series of approximate factors determined from extensive statistical parameter studies of simple hysteresis material models. These models also do not account for strength softening, usually the main cause of failure, and consider only strength degradation under repeated dynamic loading. The method therefore also does not predict failure of a component accurately.
2.4 Seismic Analysis Techniques

Both strength-based codes and performance-based design codes require specific analysis techniques in order to evaluate the desired seismic demands.

The different seismic analysis techniques are shown in figure 2.1, which is a simple schematic diagram showing the different methods of analysis for seismic design.

The linear static method is commonly used in design codes. It assumes the structure is linear elastic. The method therefore doesn’t take into account ductility effects, and can not predict collapse accurately. Furthermore dynamic effects are accounted for.
The linear dynamic method is based on either time history or modal analysis. Like the linear static method, it doesn’t account for ductility and can not therefore predict failure.

Presently, the guidelines for buildings evaluation allow the use of non-linear static and dynamic methods. The non-linear static procedure is based primarily on pushover analysis using monotonic loads up to the target displacement point. The procedure is shown in figure 2.2, and is adopted in both the capacity spectrum and coefficient methods, as described earlier. Results are accurate, however, only if higher modes effects are negligible.

![Non-Linear Static Analysis Technique – FEMA 356](image)

**Figure 2.2**  Non-Linear Static Analysis Technique – FEMA 356

An attempt to introduce direct dynamic effects in the non-linear analysis and design of building structures was proposed by Cornell and his co-workers (2002). The
process is named Incremental Dynamic Analysis (IDA) or dynamic pushover analysis. In this process, a dynamic load-deformation plot is determined by subjecting the structure to a specific earthquake history, and then scaling the earthquake record up several times and repeating the analysis. The process has been used by several researchers (e.g. Mehanny and Deierlein, 2001, Yun and Foutch, 2000, and Lee and Foutch, 2001), and is described in figure 2.3 which shows the relationship between the selected force parameter (Spectral Acceleration), and deformation parameter (maximum inter-story drift IDR).

![Figure 2.3 Non-Linear Dynamic Analysis Technique](image)

Although dynamic effects were included in the incremental dynamic analysis method, failure prediction was not possible since the material models used by most researchers followed very simple rules.
2.5 Damage Evaluation of Building Structures

Assessment of the state of a building structure after being subject to an earthquake excitation is an important tool that researchers use to evaluate the accuracy of a design process. Most of the researchers applied the concept of damage index as a mean of assessment of the damage of structural systems subject to seismic shaking. The probability of the system to reach or exceed different degrees of damage, including possible collapse is a concept known as seismic fragility. Earlier work [e.g. Singhal A., and Kiremidjian A. S. (1998), Shinozuka M. et al. (2000), Sasani M., and Kiureghian A. D. (2001)] focused on developing seismic fragility curves of systems for several values of a calibrated damage index. A damage index is a factor that represents the degree of damage of the structure, and typically ranges from 0 to 1, with the value of 1 representing complete collapse. Collapse was therefore expressed implicitly as the state of the structure when its damage index approaches a value of 1. A brief summary of current damage indices is presented next.

2.5.1 Damage Indices

In a study about seismic damage indices for concrete structures, Williams and Sexsmith (1995) tried to summarize most of the known methods for calculating damage indices. They noted that indices may be calculated from the results of a non-linear dynamic analysis, from measured response of a structure during an earthquake, or from a comparison of the physical properties of the structure before and after the earthquake. The result of their research is summarized herein below.
2.5.2 Classification of Damage

Park, Ang and Wen (1987) used a simple classification based on visual signs of damage to correlate damage indices with observed damage. This classification is as follows:

- None: localized minor cracking at worst.
- Minor: minor cracking throughout.
- Moderate: severe cracking and localized spalling.
- Severe: crushing of concrete and exposure of reinforcing bars.
- Collapse: collapse.

Although this classification is considered very simple to apply, it still needs more explanation on the interpretation of the words. For example the word “severe” does not define clearly the magnitude of cracking. Therefore, differences in levels of damage interpreted are expected.

Another different classification related to the ability to repair the building after being exposed to an earthquake, was proposed by Bracci et al (1989) and Stone and Taylor (1993):

- Undamaged or minor damage.
- Repairable.
- Irrepairable.
- Collapsed.

This classification may be harder to apply practically but it serves as a decision making tool for post-earthquake evaluation and planning of building retrofitting. The evaluation
presented by this method is essentially related to repair costs leaving out other consequences that may have occurred and caused for example economic damage due to loss of the structure.

The Earthquake Engineering Research Institute (1994) implements a different scale that encounters non-structural damage, approximate duration of loss of function and risk of fatalities to building tenants:

- **None**
- **Slight** – minor damage to non-structural elements; building reopened in less than one week.
- **Moderate** – mainly non-structural damage, little or no structural damage; building closed for up to 3 months; minor risk of loss of life.
- **Extensive** – widespread structural damage; long term closure and possibly demolition required; high risk of loss of life.
- **Complete** – collapse or very extensive, irreparable damage; very high risk of loss of life.

Williams and Sexsmith (1995) concluded that this classification has a greater correspondence with broader consequences. The main disadvantage is that correlation of this classification with the damage indices discussed afterward is somewhat poor.

### 2.5.3 Categorization of Damage

The numerous damage indices proposed could be categorized between local indices and global indices. The local indices deal with the damage level in individual members or joints. While global indices deal with the overall structure or a large part of
it. In most cases, all these indices are dimensionless parameters ranging between 0 for an undamaged structure and 1 for a collapsed structure. The intermediate values between those two numbers tend to indicate the level of damage. The damage of an overall structure will be best obtained by a global index which in this case will be a grouping of local indices in different parts of the structure or by taking into consideration structural modal parameters.

Williams and Sexsmith (1995) summarized their research in concluding that relatively few attempts were made to adjust the local indices against observed damage but they are still far from being complete. Another limitation was that, in most of those indices, focusing on damage due to bending was the principal factor leaving damage caused by shear to be doubtful. The global indices, which are originated directly from local indices, are usually acquired by using a suitable combination procedure. One can argue that a more flexible, particularly relevant approach would be more appropriate since those indices consist of a prearranged, weighted average of local indices. Even though the global softening indices have the capacity to describe the general damage condition of a structure, they still provide very little information on damage distribution across the structure.

The global indices could serve as a measure of the performance-based design. In this research, more focus is placed on the collapse prevention level of the performance-based. The following section summarizes previous work done on global collapse assessment.
2.6 Previous Work on Collapse Assessment

In a recent study on collapse assessment of frame structures under seismic excitations, Ibarra and Krawinkler (2005) reviewed the previous research on global collapse. They divided the efforts put forth in this subject into $P − \Delta$ effects and degrading hysteretic models. Analytical and experimental collapse investigations were also presented.

2.6.1 $P − \Delta$ Effects

Study of global collapse began by introducing $P − \Delta$ effects to structures under seismic excitations. Under large $P − \Delta$ values, the stiffness became negative leading to collapse of the system. Jennings and Husid (1968) developed a one-story frame with springs at columns bases. Height of the structure, ratio of the earthquake intensity to yield level of the system, and the second slope of the bilinear model were found out to be the most critical parameters affecting collapse. They also stated that duration of ground motion highly affected collapse. This finding was based on the likelihood of collapse increasing when load path stays longer rather than the consideration of degradation behavior.

Gravity effect on the dynamic behavior of an SDOF system and its effect on the change of the system’s period were studied by Sun et al. (1973). They showed that depending upon a suitable coefficient and the yield displacement, the structure can endure maximum displacement without failure. Bernal (1987) focused more on this coefficient and recommended amplification factors based on the ratio of spectral acceleration generated with and without $P − \Delta$ effects. Bernal studied elasto-plastic
SDOF systems and used same stability coefficient for all periods of interest. His conclusion was that amplification factors and natural period were not considerably correlated. In 1994, McRae expanded Bernal’s work by adding structures with more complex hysteretic response when studying $P - \Delta$ effects.

Bernal (1992 and 1998) studied two-dimensional moment-resisting frames and concluded that the system’s failure mechanism is very critical to the base shear capacity needed to resist failure. He used an equivalent elasto-plastic SDOF system comprising $P - \Delta$ effects.

### 2.6.2 Degradation Hysteretic Models

Numerous experimental studies proved that structural parameters influencing deformation and energy-dissipation characteristics affect the hysteretic behavior. Many models were generated in this aspect. Sivaselvan and Reinhorn (2000) developed a smooth hysteretic degrading model including rules for stiffness and strength degradation but excluding negative stiffness. Song and Pincheira (2000) presented cyclic strength and stiffness degradation in their model based on dissipated hysteretic energy.

Ibarra and Krawinkler (2005) used the degrading models developed by Ayoub et al. (2004) for basic bilinear, Clough and pinching hysteretic model. Degradation is based on energy dissipation following the rules proposed by Rahnama and Krawinkler (1993). The same concept was used in this research. Characteristics of these material models are presented in depth in chapter 3.
2.6.3 Analytical Collapse Investigations

Takizawa and Jennings (1980) developed a structural model equivalent to a SDOF system characterized by strength degradation. This model was used to study the maximum capacity of an RC frame under earthquake excitations and was among the first attempts to consider $P-\Delta$ effects and material degradation in collapse assessment. Aschheim and Moehle (1992) focused on the effects of prior seismic damage on peak displacement response of SDOF systems. Prior damage was modeled by reducing the initial stiffness assuming that residual displacements are negligible.

Mehanny and Deierlein (2000) examined collapse of composite structures. They calculated damage indices for a given structure and ground motion intensity record using a second-order inelastic time history analysis. They reanalyzed the damaged structure throughout a second order inelastic static analysis taking into consideration the residual displacements and gravity loads. They presumed global collapse would take place once the applied gravity loads exceeded the maximum vertical loads the system can endure.

Lee and Foutch (2001) analytical models included a fracturing element implemented by Shi (1997) in DRAIN-2DX program. The Incremental Dynamic Analysis (IDA) concept developed by Vamvatsikos and Cornell (2002) was used for evaluating the global drift capacity. Global dynamic instability was defined once the local slope of the IDA curve decreased to less than 20% of the initial slope in the elastic region. The (IDA) approach was also utilized by Jalayer (2003) to estimate global dynamic instability of regular RC structure.

Williamson (2003) evaluated the response of SDOF subjected to seismic excitations taking into consideration $P-\Delta$ effects and material degradation based on
modifying Park and Ang (1985) damage model. Miranda and Sinan (2003) investigated the lateral strengths required to present failure of bilinear SDOF systems with negative post-yield stiffness. They concluded that dispersion of lateral strengths increased as negative post-yield stiffness decreased and fundamental periods increased. Adam and Krawinkler (2003) evaluated the difference in response of non-linear systems under different analytical formulations. They concluded that large displacements formulation generates almost same responses as small displacement formulations including cases when collapse is close.

### 2.6.4 Experimental Collapse Investigations

Numerous experiments were performed to relate collapse with shear and axial failure in columns. Yoshimura and Yamanaka (2000) carried out several tests of RC columns subjected to low axial load. They noticed that loading procedure enforced on each specimen determining the lateral and axial deformation as well as the input energy at failure. In addition, ratio of vertical deformation increment to lateral deformation increment at failure was not affected by changing the loading path. They concluded that failure takes place once the lateral load decrease to less than 10% of the maximum load.

Vian and Bruneau (2001) tested a series of shake-table experiments for a SDOF steel frame structure subjected to gradually increasing earthquakes intensity. The experiments were used till collapse takes place due to geometric nonlinearities which is a form of \( P-\Delta \) effects. The conclusion was that the stability coefficient was the key factor affecting the structure’s behavior near collapse. Vian and Bruneau’s work was extended by Kanvinde (2003). He tested more SDOF systems concluding that current procedures
of non-linear dynamic analysis are reliable for failure prediction in case $P - \Delta$ effects governs the commencement of failure.

Full scale shear-critical RC concrete building columns under cyclic lateral loading was tested by Sezen (2002). The test was carried out till the column could no longer sustain the applied axial load. The tests proved that loss of axial load does not necessarily follow instantaneously after loss of lateral load capacity. Similarly, Elwood (2002) concluded that shear failure does not have to be the cause of failure of the system. They discovered that for columns having lower axial loads, axial load failure takes place at fairly large drifts, despite of whether shear failure had just occurred or occurred at much smaller drift ratios. Columns having larger axial loads experience failure usually at smaller drift ratios and may take place almost right after loss of lateral load capacity.

Ibarra and Krawinkler (2005) studied the dispersion of the collapse capacity due to record variability and uncertainty of system parameters. They concluded that softening of the post-yield stiffness and the displacement at which this softening commences are the two system parameters that control the collapse capacity of a system. Cyclic deterioration was found to be an important but not dominant factor for collapse evaluation.

Despite the large number of research studies on collapse, the previous literature review reveals the need of developing a more comprehensive procedure to assess collapse criterion in an explicit form. A new model that incorporates degradation effects into seismic analysis of MDOF structures is required and is described in the next chapter.
3.1 Material Models

Three material models were used in this research. The models considered were:

- Bilinear model to represent steel elements,
- Modified Clough model as per Clough, R. and Johnson, S. (1966) to represent concrete elements, and
- Pinching model to represent wood elements.

The main skeleton for bilinear, modified Clough, and pinching models is shown in figures 3.1, 3.2, and 3.3 respectively. All models consist of an elastic branch, a strain hardening branch, and a softening branch. A residual strength is assumed in all models. However, the loading-reloading rules under cyclic loading differ from a model to another. For the bilinear model, the initial unloading is parallel to the initial slope. The reloading curve is then bounded by the positive and negative strain hardening branches. As shown in figure 3.1, these branches form two main asymptotes for the model. For the modified Clough model, the initial unloading is parallel as well to the initial slope. As shown in figure 3.2, the behavior under cyclic loading is characterized by targeting the maximum previous displacement point. The pinching model behavior is similar to the modified Clough, except that reloading consists of two branches. The first reloading branch is
directed towards a point defined by a reduced target force. Then, the second branch is directed towards the previous maximum peak point as shown in figure 3.3.

Figure 3.1 Bilinear Model

Figure 3.2 Modified-Clough Model
3.1.1 Degradation

It is well known from experimental verification that all materials deteriorate as a function of the loading history. Each inelastic excursion causes damage and the damage accumulates as the number of excursions increases. Therefore, it is essential to include degradation effects in modeling hysteretic behavior.

There are three common methods to calculate degradation. The first method is based on ductility. The limitation of this method is that for cases when ductility is constant, there is no change between the cycles and therefore degradation does not appear in the system. Second method is a combination of ductility and energy. The main disadvantage of this method lies in its complexity to apply since too many parameters are required for calculating degradation. The third method is derived from energy only. The method has a physical interpretation since it is related to the system capacity and hence gives advantage over the previous methods.
All these models consist of a strength softening branch, refer to as a cap, to model strength degradation under monotonic loads. An 8 parameters energy-based criterion model was developed by Rahnama and Krawinkler (1993) to model four special types of cyclic degradation:

- Yield (Strength) degradation
- Unloading stiffness degradation
- Accelerated stiffness degradation
- Cap degradation

3.1.1.1 Yield (Strength) Degradation

Yield degradation refers to the decrease of the yield strength value as a function of the loading history. The strength degradation parameter is energy dependent and is derived through the following equation:

\[ F_y^i = F_y^{i-1} (1 - \beta_{str}^i) \]  \hspace{1cm} (3.1)

Where:

- \( F_y^i \) = Yield strength at the current excursion \( i \),
- \( F_y^{i-1} \) = Yield strength at the previous excursion \( i-1 \), and
- \( \beta_{str}^i \) = Scalar parameter, ranging from 0 to 1, that accounts for degradation effects at the current excursion \( i \).

The parameter \( \beta_{str}^i \) itself can be defined through the following equation:
\[ \beta_{str}^i = \left( \frac{E_i}{E_{capacity} - \sum_{j=1}^{i} E_j} \right)^{C_{str}} \]  

(3.2)

Where:

\( E_i \) = Hysteretic energy dissipated in the current excursion \( i \);

\[ \sum_{j=1}^{i} E_j \] = Total hysteretic energy dissipated in all excursions up to the current one; and

\( E_{capacity} \) = Energy dissipation capacity of the element under consideration.

\( C_{str} \) = Exponent defining the rate of deterioration.

The term \( E_{capacity} \) represents the resistance of the material to cyclic degradation.

The structure can be considered totally degraded once the total dissipated hysteretic energy, due to cyclic loading, attains a value equals to the energy dissipation capacity. Usually, \( E_{capacity} \) is calculated as a function of the strain energy up to yield through the following equation:

\[ E_{capacity} = \gamma_{str} F_y \delta_y \]  

(3.3)

Where:

\( F_y \) and \( \delta_y \) = Initial yield strength and deformation respectively and

\( \gamma_{str} \) = Constant.

The values of \( \gamma_{str} \) and \( C_{str} \) are calibrated for each material by means of experimental data.
Unloading and re-loading on the elastic branch do not cause any deterioration since no hysteretic energy is dissipated. Hence, deterioration can not be considered complete and the yield strength remains at its original value.

A complete deterioration in either the positive or negative side is achieved if one of the following conditions occurs during analysis:

\[ (E_{\text{capacity}} - \sum_{j=1}^{i} E_j) \leq E_i \]  

(3.4)

or if the term \( \beta_{str}^i \) is greater than 1.

Figure 3.4 represents the degraded envelope and corresponding decrease in yield force due to strength degradation.

**Figure 3.4** Strength Degradation for Pinching Model
3.1.1.2 Unloading Stiffness Degradation

Unloading stiffness degradation refers to the unloading stiffness as a function of the loading history similar to yield (strength) degradation. The parameter $\beta_{unl}^i$ used in the unloading stiffness degradation is also energy dependent but differs from the one of the strength degradation in the values of $C$ and $\gamma$. They are referred to by $C_u$ and $\gamma_u$. The modified unloading stiffness can be calculated through the following equation:

$$k_{unl}^i = k_{unl}^{i-1}(1 - \beta_{unl}^i) \quad (3.5)$$

Where:

$k_{unl} = $ Unloading stiffness.

Figure 3.5 represents the effect of unloading stiffness degradation on the hysteretic loop.

![Figure 3.5 Unloading Stiffness Degradation for Pinching Model](image)
3.1.1.3 Accelerated Stiffness Degradation

The reloading stiffness degrades as a function of cumulative loading in the peak-oriented models. This effect can be taken into consideration in the analytical hysteretic model by modifying the target point to which the loading is directed referred to as accelerated stiffness degradation. The accelerated stiffness degradation parameter $\beta_{acc}^i$ is similar to the strength degradation and stiffness degradation except for the values of $C$ and $\gamma$. They are referred to by $C_{acc}$ and $\gamma_{acc}$. The displacement value of the target point can be calculated through the following equation:

$$\delta_{tar}^i = \delta_{tar}^{i-1} (1 + \beta_{acc}^i)$$  \hspace{1cm} (3.6)

Where:

$\delta_{tar}^i$ = Displacement of the target point.

The effect of the accelerated stiffness degradation on the hysteretic behavior is represented in figure 3.6.

![Figure 3.6 Accelerated Stiffness Degradation for Pinching Model](image-url)
3.1.1.4 Cap Degradation

From experimental results, it is observed that the onset point of softening moves inwards as a result of cumulative damage. This is referred to as cap degradation. If the cap slope reaches the displacement axis, then collapse of the system in one direction is assumed. The cap degradation parameter $\beta_{\text{cap}}^i$ is similar to the strength, stiffness and accelerated degradations except for the values of $C$ and $\gamma$. They are referred to by $C_{\text{cap}}$ and $\gamma_{\text{cap}}$. The onset point of softening can be modified through the following equation:

$$\delta_{\text{cap}}^i = \delta_{\text{cap}}^{i-1} (1 - \beta_{\text{cap}}^i)$$

(3.7)

Where:

$\delta_{\text{cap}}^i = \text{Displacement of the onset point of softening.}$

The modified envelope due to cap degradation is represented in figure 3.7.

Figure 3.7 Cap Degradation for Pinching Model
3.1.2 Effect of Degradation on Inelastic Systems Behavior

Figures 3.9 to 3.40 represent the effect of degradation on the various material models previously explained. Numerical simulations varied from no degradation to low, moderate and severe degradation. Static loads were imposed on each system analyzed. Figures 3.8, 3.12 and 3.16 are for a non-degraded system for bilinear, modified Clough and pinching models respectively. They all share a very important characteristic which is that all the load cycles result in an envelope form defining clearly the material model used.

Once degradation is introduced in any of the material models, the load cycles begin to form a decreasing loop instead of the envelope. The number of loops or load cycles sustained by the system before collapse is influenced by the level of degradation specified. The more intensity the degradation level gets, the fewer load cycles the system sustains and the faster collapse occurs.

Figures 3.20, 3.24 and 3.28 focus on the strength degradation only for bilinear, modified Clough and pinching respectively. The graph reveals a strength reduction in each consecutive cycle lowering the yield value of the reloading cycle. The unloading stiffness degradation is presented in figures 3.21, 3.25 and 3.29 for the three models. The slope of the force-displacement curve is decreased each reloading cycle. The accelerated stiffness degradation follows the same pattern as explained in the preceding section. The reloading stiffness target a further point on the force-displacement graph moving the system towards the cap and hence accelerating failure. Figures 3.22, 3.26 and 3.30 demonstrate an example for bilinear, modified Clough and pinching models respectively.
The effect of cap degradation is illustrated in figures 3.23, 3.27 and 3.31. Each reloading cycle moves the cap branch closer to the origin which, eventually, accelerates the collapse of the system. Figures 3.32 to 3.40 show the effect of several combinations of different types of degradation on the inelastic behavior of the three models.
Figure 3.8  Bilinear Model – No Degradation

Figure 3.9  Bilinear Model – Low Degradation
Figure 3.10  Bilinear Model – Moderate Degradation

Figure 3.11  Bilinear Model – Severe Degradation
Figure 3.12  Clough Model – No Degradation

Figure 3.13  Clough Model – Low Degradation
Figure 3.14  Clough Model – Moderate Degradation

Figure 3.15  Clough Model – Severe Degradation
Figure 3.16 Pinching Model – No Degradation

Figure 3.17 Pinching Model – Low Degradation
Figure 3.18 Pinching Model – Moderate Degradation

Figure 3.19 Pinching Model – Severe Degradation
Figure 3.20  Bilinear Model – Strength Degradation

Figure 3.21  Bilinear Model – Stiffness Degradation
Figure 3.22  Bilinear Model – Accelerated Degradation

Figure 3.23  Bilinear Model – Cap Degradation
Figure 3.24  Bilinear Model – Strength and Stiffness Degradation

Figure 3.25  Bilinear Model – Accelerated and Cap Degradation
Figure 3.26  Bilinear Model – Strength, Stiffness and Accelerated Degradation

Figure 3.27  Clough Model – Strength Degradation
Figure 3.28  Clough Model – Stiffness Degradation

Figure 3.29  Clough Model – Accelerated Degradation
Figure 3.30  Clough Model – Cap Degradation

Figure 3.31  Clough Model – Strength and Accelerated Degradation
Figure 3.32  Clough Model – Stiffness and Cap Degradation

Figure 3.33  Clough Model – Stiffness, Accelerated and Cap Degradation
Figure 3.34  Pinching Model – Strength Degradation

Figure 3.35  Pinching Model – Stiffness Degradation
Figure 3.36 Pinching Model – Accelerated Degradation

Figure 3.37 Pinching Model – Cap Degradation
Figure 3.38 Pinching Model – Strength and Cap Degradation

Figure 3.39 Pinching Model – Stiffness and Accelerated Degradation
3.1.3 Collapse of Structural Elements

A structural element is considered to have experienced complete collapse if any of the following two criteria is established:

- The displacement has surpassed the value of the intersection point of the softening (cap) slope with the x-axis, which is known as a cap failure (figure 3.41), or
- The scalar parameter $\beta$, in any of the degradation types, has exceeded a value of 1 which, in this case, is known as cyclic degradation failure (figure 3.42).
It is important to note that an element might fail in one direction of loading (e.g. compression), while still demonstrating resistance in the other direction of loading (e.g. tension). A reverse loading condition can always push the element to the direction that still shows some resistance. In this case, the element can not be considered as a collapsed structure. However, in this study, complete collapse for SDOF systems is considered if any direction of loading shows no resistance. Such assumption is considered to be conservative from a design perspective. For MDOF systems though, such assumption is not considered.
3.1.4 Experimental Verification of Material Models

Several studies were performed to calibrate the material models proposed with the actual force-displacement data obtained from experimental specimens. As formerly explained, each material model represents the characteristics of a specific material: steel, concrete, or wood. The goal of the calibration procedure is to define a $\gamma$ value representing the behavior under cyclic loading. The coefficient $\gamma$ consists of four sub-coefficients each describing a type of degradation. For simplicity, $\gamma$ will be assumed to be equal in the four types of degradation (i.e. $\gamma_s = \gamma_k = \gamma_a = \gamma_d = \gamma$)
Where:

\[ \gamma_s = \text{Strength degradation parameter,} \]
\[ \gamma_k = \text{Stiffness degradation parameter,} \]
\[ \gamma_a = \text{Accelerated stiffness degradation parameter, and} \]
\[ \gamma_d = \text{Cap degradation parameters.} \]

To calibrate the degradation parameters for existing structures, correlation studies with different experimental specimens are conducted. The Bilinear model was used to simulate the behavior of the steel beam specimen tested by Krawinkler et al. (1983). Using trial and error methods, it was found that a value of \( \gamma = 100 \) for all degradation parameters produces an excellent correlation with the experimental results, as shown in figure 3.43 (a) and (b). In a recent study, Ibarra and Krawinkler (2005) stated that \( \gamma = 130 \) would be more accurate for a bilinear model. Four specimens were used applying two different loading protocols. The results showed a satisfying correlation between the experimental and the model load-deformation graph.

The same exercise was performed on a concrete column specimen to calibrate it with the modified Clough model. The column specimen was tested by Sezen and Moehle (2004), and the corresponding load-deformation data was obtained from the PEER Structural Performance Database. From the analytical simulations, it was found that a value of \( \gamma = 50 \) produces the best results as compared to the experimental ones and shown in figure 3.44 (a) and (b). The same study was conducted on a timber shear wall specimen tested at UC Irvine by Pardoen et al. (2001). A pinching model with a value of \( \gamma = 200 \) for all degradation parameters produced the best correlation as shown in figure 3.45 (a) and (b).
Figure 3.43 Bilinear Model (a) Experimental (b) Analytical

Figure 3.44 Clough Model (a) Experimental (b) Analytical

Figure 3.45 Pinching Model (a) Experimental (b) Analytical
3.2 Earthquake Records

A large database set consisting of 80 earthquake records is used in this research. Krawinkler et al. (1999, 2001) have used these records in several earlier studies. The set of records consist four bins representing different pairs of magnitude (M) and distance from fault (R) as shown herein below:

- Bin I (SMSR): small M – small R; (M < 6.5) and (R < 30 km).
- Bin II (SMLR): small M – large R; (M < 6.5) and (R > 30 km).
- Bin III (LMSR): large M – small R; (M > 6.5) and (R < 30 km).
- Bin IV (LMLR): large M – large R; (M > 6.5) and (R > 30 km).

Each of the above mentioned bins constitutes of 20 earthquake records which were recorded in California and correspond to soil types C or D (stiff soil or soft rock) as per the NEHRP soil classification. Figure (3.46) represents the magnitude-distance distribution of the 80 records according to records details tables 3.1 to 3.8.

![Magnitude-Distance Distribution of the 80 Earthquake Records](Medina2000)

**Figure 3.46** Magnitude-Distance Distribution of the 80 Earthquake Records (Medina 2000)
3.2.1 Database of Earthquake Records

The large database of records used in this study is presented herein in tables 3.1 to 3.4. The records were sorted, as previously explained, into four different bins. Tables 3.5 to 3.8 contained more detailed properties for each record in terms of number of points, time step, and total time to facilitate the use of these records in analysis.

Table 3.1 Earthquakes Having Small Magnitude and Small Distance from Fault

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<thead>
<tr>
<th>Earthquake</th>
<th>Date</th>
<th>Station</th>
<th>Legend</th>
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Table 3.2  Earthquakes Having Small Magnitude and Large Distance from Fault

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# Table 3.3 Earthquakes Having Large Magnitude and Small Distance from Fault

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Table 3.4  Earthquakes Having Large Magnitude and Large Distance from Fault

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Records Details of SMSR

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Table 3.8  Records Details of LMLR

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3.2.2 Scaling of Earthquake Records

Cornell and his co-workers (1999) showed in an earlier study that proper scaling of earthquake records in general, does not introduce any preconception to the response. Consequently, it will reduce the necessity of the number of analysis needed for statistical evaluation. Moreover, proper scaling ensures that all records used in the study fall within the same hazard level defined by codes of practice. Cornell’s study proved that scaling an ensemble of records, even if they don’t fall initially within the same hazard level, to the median spectral acceleration value will not change the median values of the response quantities, but reduces considerably the variability in results. The conclusion he reached was also applicable to scaling to any value of spectral acceleration, whether it is higher or lower than the median value. Cornell’s approach was based mainly on statistical analysis of non-degrading simple bilinear structural systems.

In a recent study, Ayoub and Mijo (2006) studied Cornell’s approach taking into consideration the degradation effects for different material models such as bilinear, modified Clough, and pinching. First, to confirm the reduction in variability due to scaling of degraded structures, he calculated the mean (denoted by $\bar{\mu}$) and dispersion values (denoted by $\delta$) of two demand parameters; ductility ($\mu$) and normalized hysteretic energy (NHE). Those two statistical properties were calculated for each bin for both, the set of un-scaled records, and the set of records scaled to the mean spectral acceleration value. The results obtained for bin I for four different cases of degradation for a SDOF system with period 1 sec, and for yield value characterized by $\eta = 0.09$, where $\eta$ is the ratio of the yield force to the weight, are represented in tables 3.9 to 3.14.
These results are compared with the non-degrading case for bilinear, modified Clough, and pinching models. The cases considered by Ayoub and Mijo were:

- Case (1): No degradation,
- Case (2): Cap slope $= -6\%$ and no cyclic degradation,
- Case (3): Cap slope $= -6\%$ and all degradation parameters $\gamma = 150$,
- Case (4): Cap slope $= -6\%$ and all degradation parameters $\gamma = 100$, and
- Case (5): Cap slope $= -6\%$ and all degradation parameters $\gamma = 50$.

The value of $\eta = 0.09$ was noted to be corresponding to a single common strength reduction factor $R$ value for each record in the scaled set, but to different $R$ values for each record in the un-scaled set.

### Tables 3.9-14 Median and Dispersion Values of Ductility ($\mu$) and Normalized Hysteretic Energy (NHE)

**Table 3.9**
**Bilinear Un-scaled** $T = 1s$, $\eta = 0.09$

<table>
<thead>
<tr>
<th>Case</th>
<th>$\hat{\mu}$</th>
<th>$\delta_\mu$</th>
<th>$\hat{NHE}$</th>
<th>$\delta_{NHE}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1)</td>
<td>2.608</td>
<td>0.762</td>
<td>6.744</td>
<td>1.522</td>
</tr>
<tr>
<td>(2)</td>
<td>2.651</td>
<td>0.869</td>
<td>6.744</td>
<td>1.485</td>
</tr>
<tr>
<td>(3)</td>
<td>2.652</td>
<td>0.855</td>
<td>6.771</td>
<td>1.480</td>
</tr>
<tr>
<td>(4)</td>
<td>2.655</td>
<td>0.848</td>
<td>6.792</td>
<td>1.475</td>
</tr>
<tr>
<td>(5)</td>
<td>2.651</td>
<td>0.821</td>
<td>6.853</td>
<td>1.431</td>
</tr>
</tbody>
</table>

**Table 3.10**
**Bilinear Scaled** $T = 1s$, $\eta = 0.09$

<table>
<thead>
<tr>
<th>Case</th>
<th>$\hat{\mu}$</th>
<th>$\delta_\mu$</th>
<th>$\hat{NHE}$</th>
<th>$\delta_{NHE}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1)</td>
<td>2.796</td>
<td>0.323</td>
<td>6.872</td>
<td>0.519</td>
</tr>
<tr>
<td>(2)</td>
<td>2.799</td>
<td>0.326</td>
<td>6.907</td>
<td>0.515</td>
</tr>
<tr>
<td>(3)</td>
<td>2.791</td>
<td>0.328</td>
<td>6.995</td>
<td>0.328</td>
</tr>
<tr>
<td>(4)</td>
<td>2.79</td>
<td>0.328</td>
<td>7.006</td>
<td>0.524</td>
</tr>
<tr>
<td>(5)</td>
<td>2.784</td>
<td>0.328</td>
<td>7.106</td>
<td>0.533</td>
</tr>
</tbody>
</table>
Table 3.11
Clough Un-scaled T = 1s, $\eta = 0.09$

<table>
<thead>
<tr>
<th>Case</th>
<th>$\hat{\mu}$</th>
<th>$\hat{\sigma}_\mu$</th>
<th>$\hat{NHE}$</th>
<th>$\hat{\delta}_{NHE}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1)</td>
<td>2.558</td>
<td>0.857</td>
<td>8.962</td>
<td>1.054</td>
</tr>
<tr>
<td>(2)</td>
<td>2.587</td>
<td>0.951</td>
<td>8.962</td>
<td>0.98</td>
</tr>
<tr>
<td>(3)</td>
<td>2.570</td>
<td>0.948</td>
<td>8.914</td>
<td>0.956</td>
</tr>
<tr>
<td>(4)</td>
<td>2.569</td>
<td>0.951</td>
<td>8.927</td>
<td>0.943</td>
</tr>
<tr>
<td>(5)</td>
<td>2.574</td>
<td>0.886</td>
<td>8.908</td>
<td>0.927</td>
</tr>
</tbody>
</table>

Table 3.12
Clough Scaled T = 1s, $\eta = 0.09$

<table>
<thead>
<tr>
<th>Case</th>
<th>$\hat{\mu}$</th>
<th>$\hat{\sigma}_\mu$</th>
<th>$\hat{NHE}$</th>
<th>$\hat{\delta}_{NHE}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1)</td>
<td>2.686</td>
<td>0.372</td>
<td>8.168</td>
<td>0.487</td>
</tr>
<tr>
<td>(2)</td>
<td>2.692</td>
<td>0.369</td>
<td>8.231</td>
<td>0.485</td>
</tr>
<tr>
<td>(3)</td>
<td>2.678</td>
<td>0.374</td>
<td>8.206</td>
<td>0.482</td>
</tr>
<tr>
<td>(4)</td>
<td>2.683</td>
<td>0.378</td>
<td>8.203</td>
<td>0.481</td>
</tr>
<tr>
<td>(5)</td>
<td>2.689</td>
<td>0.379</td>
<td>8.165</td>
<td>0.486</td>
</tr>
</tbody>
</table>

Table 3.13
Pinching Un-scaled T = 1s, $\eta = 0.09$

<table>
<thead>
<tr>
<th>Case</th>
<th>$\hat{\mu}$</th>
<th>$\hat{\sigma}_\mu$</th>
<th>$\hat{NHE}$</th>
<th>$\hat{\delta}_{NHE}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1)</td>
<td>2.624</td>
<td>0.818</td>
<td>8.333</td>
<td>0.937</td>
</tr>
<tr>
<td>(2)</td>
<td>2.647</td>
<td>0.876</td>
<td>8.333</td>
<td>0.886</td>
</tr>
<tr>
<td>(3)</td>
<td>2.663</td>
<td>0.882</td>
<td>8.362</td>
<td>0.874</td>
</tr>
<tr>
<td>(4)</td>
<td>2.668</td>
<td>0.884</td>
<td>8.381</td>
<td>0.868</td>
</tr>
<tr>
<td>(5)</td>
<td>2.707</td>
<td>0.901</td>
<td>8.414</td>
<td>0.824</td>
</tr>
</tbody>
</table>

Table 3.14
Pinching Scaled T = 1s, $\eta = 0.09$

<table>
<thead>
<tr>
<th>Case</th>
<th>$\hat{\mu}$</th>
<th>$\hat{\sigma}_\mu$</th>
<th>$\hat{NHE}$</th>
<th>$\hat{\delta}_{NHE}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1)</td>
<td>2.782</td>
<td>0.382</td>
<td>7.274</td>
<td>0.542</td>
</tr>
<tr>
<td>(2)</td>
<td>2.787</td>
<td>0.390</td>
<td>7.275</td>
<td>0.543</td>
</tr>
<tr>
<td>(3)</td>
<td>2.767</td>
<td>0.396</td>
<td>7.086</td>
<td>0.569</td>
</tr>
<tr>
<td>(4)</td>
<td>2.772</td>
<td>0.40</td>
<td>7.065</td>
<td>0.576</td>
</tr>
<tr>
<td>(5)</td>
<td>2.776</td>
<td>0.41</td>
<td>6.889</td>
<td>0.617</td>
</tr>
</tbody>
</table>

From Ayoub’s results, two conclusions are drawn. First, for most of the cases, the mean value of response quantities was not influenced much by the scaling procedure. Typically, a difference of less than 10% between the scaled and un-scaled response is observed. This conclusion ensures that no prejudice is introduced by the scaling procedure. Second, the dispersion in results is significantly lower for the set of scaled records, which confirms the reduction in variability of results.
The same study was performed for the case of bin IV scaled to the median spectral acceleration of bin I. The results showed that the mean value of ductility was not much affected and the dispersion values were also significantly lower than those of the un-scaled set of records. Tables 3.15 to 3.20 represent the results for the $\eta_{\text{fail}}$ values for both the scaled and un-scaled records. These results also confirm that scaling process did not prejudice the $\eta_{\text{fail}}$ values, but rather reduced the variability of results as confirmed by the low dispersion values.

### Tables 3.15-20 Median and Dispersion Values of Strength at Failure ($\eta_{\text{fail}}$)

#### Table 3.15
**Bilinear Un-scaled T = 1s**

<table>
<thead>
<tr>
<th>Case</th>
<th>$\hat{\eta}_{\text{fail}}$</th>
<th>$\delta_{\eta_{\text{fail}}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>(2)</td>
<td>0.022</td>
<td>0.773</td>
</tr>
<tr>
<td>(3)</td>
<td>0.023</td>
<td>0.712</td>
</tr>
<tr>
<td>(4)</td>
<td>0.026</td>
<td>0.616</td>
</tr>
<tr>
<td>(5)</td>
<td>0.0365</td>
<td>0.556</td>
</tr>
</tbody>
</table>

#### Table 3.16
**Bilinear Scaled T = 1s**

<table>
<thead>
<tr>
<th>Case</th>
<th>$\hat{\eta}_{\text{fail}}$</th>
<th>$\delta_{\eta_{\text{fail}}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>(2)</td>
<td>0.0215</td>
<td>0.356</td>
</tr>
<tr>
<td>(3)</td>
<td>0.0215</td>
<td>0.363</td>
</tr>
<tr>
<td>(4)</td>
<td>0.024</td>
<td>0.352</td>
</tr>
<tr>
<td>(5)</td>
<td>0.0335</td>
<td>0.299</td>
</tr>
</tbody>
</table>

#### Table 3.17
**Clough Un-scaled T = 1s**

<table>
<thead>
<tr>
<th>Case</th>
<th>$\hat{\eta}_{\text{fail}}$</th>
<th>$\delta_{\eta_{\text{fail}}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>(2)</td>
<td>0.0235</td>
<td>0.752</td>
</tr>
<tr>
<td>(3)</td>
<td>0.0255</td>
<td>0.723</td>
</tr>
<tr>
<td>(4)</td>
<td>0.028</td>
<td>0.742</td>
</tr>
<tr>
<td>(5)</td>
<td>0.034</td>
<td>0.629</td>
</tr>
</tbody>
</table>

#### Table 3.18
**Clough Scaled T = 1s**

<table>
<thead>
<tr>
<th>Case</th>
<th>$\hat{\eta}_{\text{fail}}$</th>
<th>$\delta_{\eta_{\text{fail}}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>(2)</td>
<td>0.0235</td>
<td>0.440</td>
</tr>
<tr>
<td>(3)</td>
<td>0.027</td>
<td>0.417</td>
</tr>
<tr>
<td>(4)</td>
<td>0.0265</td>
<td>0.472</td>
</tr>
<tr>
<td>(5)</td>
<td>0.0345</td>
<td>0.409</td>
</tr>
</tbody>
</table>
The conclusion of Cornell’s and Ayoub’s work is that proper scaling can reduce significantly the variability in results, and hence a much smaller number of non-linear analyses is required to conduct statistical studies. Figures 3.47 to 3.50 show normalized scaled spectral acceleration for the four bins all scaled at a period value of 0.7 sec. The solid thick line in each of these graphs represents the median value of the records whereas the dotted thick line corresponds to the 84th percentile. It is noted that the scaled factor of all the records is equal to 1.0 at $T = 0.7$ sec since this is the target period.
Figure 3.47  Bin I (SMSR) Scaled to T= 0.7 sec

Figure 3.48  Bin II (SMLR) Scaled to T= 0.7 sec
Figure 3.49  Bin III (LMSR) Scaled to T= 0.7 sec

Figure 3.50  Bin IV (LMLR) Scaled to T= 0.7 sec
CHAPTER 4
ASSESSMENT OF DEGRADED SDOF STRUCTURES

4.1 Introduction

The number of modes necessary for an accurate dynamic analysis is a function of two parameters: modal contribution factor and spectral ordinates associated with the modal response equation. The following equation by Chopra (2005) explained the consequence of choosing the first $J$ modes in analysis on the error in the static response:

$$e_J = 1 - \sum_{n=1}^{J} \bar{F}_n$$  (4.1)

Where:

$e_J$ = Error in the static response and

$\bar{F}_n$ = Modal contribution factor.

Accordingly, the modal analysis can be reduced when the magnitude of error becomes adequately small for the target response quantity. Chopra (2005) suggested that in order to attain the desired accuracy of dynamic analysis:

- More modes should be considered for taller buildings having longer periods rather than shorter buildings with smaller periods, and
More modes should be considered for shear wall building having a higher beam-to-column stiffness ratio rather than moment-resisting frames buildings having smaller beam-to-column stiffness ratio.

Currently, the rule of thumb is to include the number of modes equal to one tenth the number of floors to limit the static response error to 10%. For example, a building having five floors would require the first modal analysis only to obtain an acceptable error. A seventeen floor structure would require at least considering the first two modes. This rule is proposed based on an assumption that the buildings systems are single bay frames.

The purpose of this chapter is to investigate the effect of degradation on the behavior of SDOF systems, to develop a new numerical procedure for predicting maximum inelastic displacements of SDOF and first mode-dominant degrading building structures, and to predict collapse under seismic excitations. Seismic fragility curves for a collapse criterion, defined as the probability of the system to collapse are also developed for different structural systems. The findings provide necessary background for the design evaluation phase of a general performance-based earthquake design process. Investigation of the degradation effect on the behavior of SDOF structures is conducted first.

4.2 Degradation Effect on SDOF Systems Under Seismic Excitations

Figures 4.1 to 4.2 investigate the effect of degradation on SDOF systems. A bilinear system equivalent to a 3-story structure was selected. The period of the structure $T = 0.294$ and its damping ratio $\zeta = 5\%$. The degradation parameters $(\gamma)$ used was equal to 0 and 50 for no and severe degradation respectively. The Imperial Valley
earthquake (IV79e01) recorded at station El Centro 1 was used in the analysis. This record falls into the most severe bin characterized by large magnitude and small distance from fault. The duration of this record was 40 sec in total as shown in figure 4.1. The non-degraded system doesn’t experience collapse as shown in figure 4.1, while the degraded system experienced collapse after 8.6 sec as shown in figure 4.2. The force-displacement diagrams for both non-degraded and degraded cases shown in figures 4.3 and 4.4 respectively reflect the behavior of the system. Collapse, which is denoted by a ‘*’ symbol on the graph 4.4, occurred at 2.03 inches while the maximum displacement for the non-degraded system was 1.71 inches. The behavior of the non-degraded system in the force-displacement graph was bounded by the original envelope. Initially, the behavior was in the elastic and strain hardening zone. In the last few cycles, the behavior reached the cap negative slope. In the degraded system, though, the behavior reached the cap in the first few cycles, and was eventually driven to collapse at a displacement that equals 2.03 inches.
Figure 4.1  
SDOF Time History for Roof Displ., 3 Floors, Bilinear, No Degradation

Figure 4.2  
SDOF Time History for Roof Displ., 3 Floors, Bilinear, Severe Degradation
Figure 4.3  SDOF Force-Displacement, 3 Floors, Bilinear and No Degradation

Figure 4.4  SDOF Force-Displacement, 3 Floors, Bilinear and Severe Degradation
The same material model and earthquake record were used on a system representing a ten-story structure. The period of the system was $T = 0.725$ and its damping ratio was $\zeta = 5\%$. The degradation parameters ($\gamma$) used was equal to 0 and 50 for no and severe degradation respectively. The overall behavior shown in figures 4.5 to 4.8 is similar to that of the 3-story structure with the exception of the displacement values. The maximum roof displacement in the case of no degradation was equal to 3.86 inches compared to 1.71 in for the 3-floor structure. Collapse occurred for a severely degraded case after 17 sec with a roof displacement value of 4.07 inches. The 10 stories system lasted longer than the three stories leading to more loading cycles as shown in figure 4.6.

![Figure 4.5 SDOF Time History for Roof Displ., 10 Floors, Bilinear and No Degradation](image-url)
Figure 4.6  SDOF Time History for Roof Displ., 10 Floors, Bilinear and Severe Degradation

Figure 4.7  SDOF Force-Displacement, 10 Floors, Bilinear and No Degradation
4.3 Displacement Estimates of SDOF Degraded Structures

The goal of this part of the study is to predict collapse of SDOF systems, and to provide an estimate for the maximum inelastic displacements in case collapse does not occur. A large set of building structures is selected for the study. The periods of these structures are 0.1, 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1.0, 1.2, 1.5, 1.8 and 2.0 sec. Three values for the strength reduction factor (R) were also used in this study: 4, 6, and 8. This wide range of periods and strength reduction factors allowed a thorough observation of the behavior of the SDOF systems. The 4 bins of earthquake records recorded in California, and described earlier in chapter 3 are used to conduct the numerical study. The material models used are bilinear, modified Clough, and pinching described earlier. Three different degradation cases for each of the three material models are considered.
and compared to a corresponding non-degrading system. These cases represent low 
\( \gamma = 150 \), moderate \( \gamma = 100 \), and severe degradation \( \gamma = 50 \) respectively. The cap 
displacement is assumed to equal 4 times the yield displacement, and its slope equals 6\% 
of the initial slope. The residual strength is assumed to equal zero. Plots of ratio of 
maximum inelastic displacements to maximum elastic displacements for different period 
values and for the different strength reduction factors \( R \) are generated for all 
degradation cases. The results for the case of Bins I-IV scaled to a spectral acceleration 
according to USGS values LA 10/50 are shown in figures 4.10 to 4.27. Mean collapse is 
defined when more than 50\% of the records failed. The last point before collapse of the 
system is identified with a ‘*’ in the plots generated, and no corresponding point for non-
degraded systems exist.

Several variables in the analysis had to be determined before conducting the 
analysis such as eita \( \eta \) defined as the ratio of yield force to weight of the system. Table 
4.1 presents values of \( \eta \) used for different strength reduction factor \( R \) and periods.

<table>
<thead>
<tr>
<th>Period</th>
<th>( \eta_{R=1} )</th>
<th>( \eta_{R=4} )</th>
<th>( \eta_{R=6} )</th>
<th>( \eta_{R=8} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>1.1111</td>
<td>0.2778</td>
<td>0.1852</td>
<td>0.1389</td>
</tr>
<tr>
<td>0.2</td>
<td>1.1111</td>
<td>0.2778</td>
<td>0.1852</td>
<td>0.1389</td>
</tr>
<tr>
<td>0.3</td>
<td>1.1111</td>
<td>0.2778</td>
<td>0.1852</td>
<td>0.1389</td>
</tr>
<tr>
<td>0.4</td>
<td>1.1111</td>
<td>0.2778</td>
<td>0.1852</td>
<td>0.1389</td>
</tr>
<tr>
<td>0.5</td>
<td>1.1111</td>
<td>0.2778</td>
<td>0.1852</td>
<td>0.1389</td>
</tr>
</tbody>
</table>
Table 4.1 (Continued)

<table>
<thead>
<tr>
<th>0.6</th>
<th>1.1111</th>
<th>0.2778</th>
<th>0.1852</th>
<th>0.1389</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.7</td>
<td>0.9524</td>
<td>0.2381</td>
<td>0.1587</td>
<td>0.1191</td>
</tr>
<tr>
<td>0.8</td>
<td>0.8333</td>
<td>0.2083</td>
<td>0.1389</td>
<td>0.1042</td>
</tr>
<tr>
<td>0.9</td>
<td>0.7407</td>
<td>0.1852</td>
<td>0.1235</td>
<td>0.0926</td>
</tr>
<tr>
<td>1.0</td>
<td>0.6667</td>
<td>0.1667</td>
<td>0.1111</td>
<td>0.0833</td>
</tr>
<tr>
<td>1.2</td>
<td>0.5556</td>
<td>0.1389</td>
<td>0.0926</td>
<td>0.0695</td>
</tr>
<tr>
<td>1.5</td>
<td>0.4444</td>
<td>0.1111</td>
<td>0.0741</td>
<td>0.0556</td>
</tr>
<tr>
<td>1.8</td>
<td>0.3700</td>
<td>0.0925</td>
<td>0.0617</td>
<td>0.0463</td>
</tr>
<tr>
<td>2.0</td>
<td>0.3333</td>
<td>0.0833</td>
<td>0.0556</td>
<td>0.0417</td>
</tr>
</tbody>
</table>

The ratios of maximum inelastic to maximum elastic displacements for a strength reduction factor $R = 4$ are shown in figures 4.10 to 4.15. Figures 4.10, 4.11 and 4.12 show the results for a bilinear, Clough and pinching model respectively for mean values. While figures 4.13, 4.14 and 4.15 show the results for the same material models but for 84th percentile values. This same set of plots is repeated again for a strength reduction factor value of $R = 6$ in figures 4.16 to 4.21. The results are also presented for strength reduction factor $R = 8$ in figures 4.22 to 4.27. Several conclusions can be extracted from those graphs to better understand the effect of different variables on the ratio of maximum inelastic displacement to maximum elastic displacement.

From all figures, it is clear that, not only degradation did not affect the behavior of long period structures, but also in this range, the equal displacement rule still applies even for degraded systems. The effect of degradation becomes apparent for short period
structures ($T < 0.5$ sec) for both mean and 84th percentile values. In this range, degradation increases the maximum inelastic displacements for all three material models. This conclusion applies as well for the different strength reduction factors. For very short periods ($T < 0.2$ sec), degraded system typically collapse at any level of degradation.

The difference between mean and 84th percentile values is clearly shown when comparing ratios at periods $T = 0.3$ sec. For example, when examining figures 4.10 and 4.13 we notice that severely degraded systems collapse only for the 84th percentile values. This finding is justified by the fact that the 84th percentile values are more stringent than median values. Higher values of strength reduction factor also influence the collapse criteria. The evaluation of plots in figures 4.11, 4.17 and 4.23 illustrates this influence. Those three graphs share a Clough model with moderate degradation and median values but differ in the value of $R$. For $R = 4$, collapse occurs at $T = 0.1$ sec while $T$ is equal to 0.2 sec for $R = 6$ and $T = 0.3$ sec for $R = 8$. This can be explained by examining figure 4.9 which demonstrates that increasing the value of $R$ is equivalent to decreasing the yield force value resulting into a more conservative model and consequently escalating the collapse probability.

![Figure 4.9](image-url)  
**Figure 4.9** Effect of Strength Reduction Factor ($R$) on Yield Force ($F_y$)
In figures 4.10, 4.11 and 4.12, for strength reduction factor $R = 4$ and $T = 0.3$, the ratios of maximum inelastic to elastic displacement for severely degraded systems equals to 1.48, 2.04 and 2.15 for bilinear, Clough and pinching models respectively. For $R = 6$ and $T = 0.5$ in figures 4.16, 4.17 and 4.18, the ratios equal 1.21, 1.50 and 1.51 for bilinear, Clough and pinching models. Similarly, at $R = 8$ and $T = 0.8$, the ratios in figures 4.22, 4.23 and 4.24 equal to 0.97, 1.01 and 1.01. The ratio in bilinear model leans to be lower than its corresponding values in Clough and pinching models. Moreover, the Clough and pinching values are almost identical. The previous observation is justified by reviewing the characteristics of the material models explained earlier in chapter 3.

Difference in material models characteristics is also noticed when examining collapse of severely degraded systems for the different cases of strength reduction factor. In figure 4.10 collapse occurs at $T = 0.3$ for $R = 4$. For the same conditions except for $R = 6$, collapse takes place at $T = 0.5$ as shown in figure 4.16 with a 66% increase in the period value. This value equals to 0.8 sec in figure 4.22 when $R$ reaches a value of 8 denoting a 60% increase from the previous value. For Clough models in figures 4.11, 4.17 and 4.23 collapse occurs at $T = 0.2$, 0.3 and 0.4 for $R = 4$, 6 and 8 respectively with 50% and 33% increase. Likewise, in figures 4.12, 4.18 and 4.24 severely degraded pinching systems collapse at a period value of 0.3, 0.4 and 0.5 for $R = 4$, 6 and 8 respectively with 33% and 25% increase respectively. The previous results confirm the fact that degradation has a major effect on the inelastic behavior of short period structures, and on the potential of collapse of these systems.
Figure 4.10  Bilinear Model, Median and R=4

Figure 4.11  Clough Model, Median and R=4
Figure 4.12 Pinching Model, Median and R=4

Figure 4.13 Bilinear Model, 84th % and R=4
Figure 4.14    Clough Model, 84\textsuperscript{th} % and R=4

Figure 4.15    Pinching Model, 84\textsuperscript{th} % and R=4
Figure 4.16  Bilinear Model, Median and R=6

Figure 4.17  Clough Model, Median and R=6
Figure 4.18 Pinching Model, Median and R=6

Figure 4.19 Bilinear Model, 84th % and R=6
Figure 4.20  Clough Model, 84th % and R=6

Figure 4.21  Pinching Model, 84th % and R=6
Figure 4.22  Bilinear Model, Median and R=8

Figure 4.23  Clough Model, Median and R=8
Figure 4.24 Pinching Model, Median and R=8

Figure 4.25 Bilinear Model, 84th % and R=8
**Figure 4.26** Clough Model, 84th % and R=8

**Figure 4.27** Pinching Model, 84th % and R=8
4.4 Incremental Dynamic Analysis and Fragility of Collapse for SDOF

Vamvatsikos and Cornell (2002) defined the Incremental Dynamic Analysis (IDA) as a parametric analysis method described in several different forms to estimate more thoroughly structural performance under seismic loads. The process involves subjecting a structural model to one (or more) ground motion record(s) which are scaled to multiple levels of intensity and hence, resulting in one (or more) curve(s) of response parameters versus intensity level. The Incremental Dynamic Analysis (IDA) plots establish a relationship between seismic demand parameters and strength parameters. Examples of seismic demand parameters could be ductility ($\mu$) or inter-story drift, whereas examples of strength parameters could be spectral acceleration ($S_a$) or strength reduction factor ($R$) commonly used in codes of practices. In this study, several IDA ($\mu - R$) curves are plotted for a variety of degrading structures.

To better understand the relationship between ($R$) and ($\mu$) shown in figure 4.28, the following relations are introduced:

\[ R = \frac{F_c}{F_y} = \frac{\delta_x}{\delta_y} \]  \hspace{1cm} (4.2)

\[ \mu = \frac{\delta_m}{\delta_y} \]  \hspace{1cm} (4.3)

\[ \frac{R}{\mu} = \frac{\delta_x}{\delta_y} \times \frac{\delta_y}{\delta_m} = \frac{\delta_r}{\delta_m} \]  \hspace{1cm} (4.4)
Where:

\[ F_e = \text{Value of maximum Force if the structure remains elastic}, \]

\[ F_y = \text{Value of Force if the structure is yielding}, \]

\[ \delta_e = \text{Value of Displacement corresponding to elastic Force}, \]

\[ \delta_y = \text{Value of Displacement corresponding to yield Force}, \] and

\[ \delta_m = \text{Value of maximum Displacement attained by the yielded structure}. \]

Using the above equations, we can derive that the starting point on a \( R - \mu \) graph would be \((1,1)\) as shown in figure 4.28. The basis of this finding is that the least value of maximum displacement \( \delta_m \) in a yielding system would be the yield displacement value \( \delta_y \) itself. Furthermore, a case with no strength reduction factor means that the elastic point and the yield point coincide and their ratio would equal 1. Although collapse occurs at high ductility values, it is desirable to increase the ductility to prevent brittle failure. Ductility allows for hysteric energy to be dissipated which adds more damping to the system. Points that correspond to collapse are identified with a (*) in the IDA plot. The coordinates of the collapse points \((\eta_{\text{fail}}, R_{\text{fail}})\) represent the ductility capacity and the strength reduction factor at collapse.
4.4.1 Ductility Capacity and Strength Reduction Factor at Collapse

To determine the ductility capacity and strength reduction factor at collapse, the median \((\mu - R)\) curves of an ensemble of structures are plotted. The structures are assumed to be excited with all 80 records scaled to a common value. Four periods are selected for this study: 0.2 sec, 0.5 sec., 1 sec., and 2 sec. Damping was assumed constant and equals 5%. All three material models, bilinear, Clough and pinching models are used in the study. The results for a structure with period that equals 0.2 sec. are discussed first.

4.4.1.1 Short Period Structures \((T = 0.2\text{ sec})\)

Figure 4.29 shows the median values of the \((\mu - R)\) curve for a bilinear model with fixed envelope values but with different degradation parameters. All degradation parameters \(\gamma\), namely for strength, unloading stiffness, accelerated stiffness, and cap are assumed to be equal. A value of \(\gamma = 150\) corresponds to a system with low degradation, a value of \(\gamma = 100\) corresponds to a system with moderate degradation, and a value of \(\gamma = 50\) corresponds to a system with severe degradation. The cap displacement \(\delta_{cap}\) is
assumed to equal 4 times the yield displacement. From figure 4.29, it is rather obvious that degradation had a great effect on the value of $\mu_{\text{fail}}$. The ductility capacity of a system with no degradation, which fails mainly due to softening effects, is $\mu_{\text{fail}} = 21.35$. The ductility capacity of systems with low, moderate, and high degradation is 12, 9.5, and 6.6 respectively. Degradation can thus reduce the ductility capacity by a value that could be greater than 3. The strength reduction factor at collapse $R_{\text{fail}}$ is also reduced from 5.4 for the case of no degradation to 3.4 for the case of highly degraded structures.

Figure 4.30 shows the median plot for the same system with a fixed degradation value $\gamma = 100$, but with different cap displacement values $\delta_{\text{cap}} = 1, 4, 6, 8$ and $\infty$. The case with $\delta_{\text{cap}} = \infty$ corresponds to a system with cyclic degradation only, and the case with $\delta_{\text{cap}} = 1$ corresponds to a brittle-fracture system where cap softening starts right after the elastic branch. The ductility capacity, except for the latter case, was not affected much by the onset of softening and ranges between 9.3 and 10.9 with the corresponding $R_{\text{fail}}$ value ranging between 4.2 and 4.6. These results suggest that for bilinear non-brittle systems, failure is most likely due to cyclic degradation effects, rather than to softening effects. For the brittle case, the ductility capacity is dramatically reduced to a value that equals 4.38 with a corresponding $R_{\text{fail}}$ value of 2.2. In this case, the early presence of the cap dominates the response and drives the system quickly into failure.

Figures 4.31 and 4.32 show plots similar to the ones described above, but for a Clough model. The effect of degradation is clearly manifested in figure 4.31 where $\mu_{\text{fail}}$ decreases from a value of 19.1 for the non-degraded case to a value of 5.1 for the highly degraded case. The corresponding strength reduction factor $R_{\text{fail}}$ is also decreased from
4.2 to 2.6. The effect of the presence of the cap is illustrated in figure 4.32. A system with no cap has a ductility capacity that equals 18, while a system with a cap has a ductility capacity that ranges between 6.5 and 10.3. These results reveal that Clough systems fail mainly due to softening effects. Cyclic degradation accelerates failure, but to a much lesser extent than for Bilinear models since the hysteretic energy dissipated in this case is much less than for a Bilinear case. The final outcome is that bilinear systems are actually more ductile than Clough systems, even though they dissipate more energy. This fact is mainly due to the different failure mode of each system.

The same results of a pinched model are rather interesting, and are shown in figures 4.33 and 4.34. The ductility capacity of a non-degraded system is actually lower than that of degraded systems. The pinching stress is originally assumed to equal half the yield value. Due to degradation effects, the pinching point moves away from the origin, and accelerated degradation dominates the response. Displacements are thus increased, and so is the ductility capacity. Figure 4.33 shows that $\mu_{\text{fail}}$ for a non-degraded system equals 13, while it equals 16.42 for a system with moderate degradation. The corresponding $R_{\text{fail}}$ value decreases with degradation though. It equals 3.8 for a non-degraded system, and 3.4 for a system with moderate degradation. Degraded pinching systems are considered thus more ductile than corresponding Bilinear and Clough systems, while non-degraded pinching systems are more brittle than Bilinear and Clough ones.
4.4.1.2 Medium Period Structures \( (T = 0.5 \text{ sec}) \)

Figure 4.35 shows the \((\mu - R)\) curve for a bilinear system. Just like the previous case, degradation has a big effect on the ductility capacity. The value of \(\mu_{\text{fail}}\) equals 19.9 for a non-degraded system, and drops to a value of 12.5 for moderately degraded systems, and 7.6 for severely degraded systems. The \(R_{\text{fail}}\) value is considerably higher than in the case of \(T = 0.2 \text{ sec}\), and equals 9.4 for non-degraded systems and 5.8 for severely degraded systems. Figure 4.36 shows the same plot for different cap displacements. As for the case of \(T = 0.2 \text{ sec}\), the onset of softening did not affect the ductility capacity, except for very brittle cases, which suggests that failure, except for the latter case, is mainly due to cyclic degradation.

The effect of degradation on ductility capacities for a Clough model is not as high as for a Bilinear model as illustrated in figure 4.37. A severely degraded system has a ductility capacity of 11.4, while a non-degraded system has a value of 15.2. A system with low degradation has a ductility capacity value slightly higher than a non-degraded system due to accelerated degradation effects. Figure 4.38 also shows that the presence of a cap did have a major influence on the value of \(\mu_{\text{fail}}\). The value of \(R_{\text{fail}}\) is in the same range as for the bilinear system. The behavior of a pinched model in this case is similar to that of the Clough model as illustrated in figures 4.39 and 4.40. Both of these models fail mainly due to softening effects, with the cyclic degradation accelerating the failure rate.

4.4.1.3 Long Period Structures \( (T = 1.0 \text{ sec}) \)

It is well known that structures in this region follow the equal displacement rule, where maximum inelastic displacements equal equivalent maximum elastic
displacements. It is interesting to note that the previous conclusion holds true also for degrading systems in case collapse doesn’t occur. Degradation can, however, accelerate the failure of the structure. The previous conclusion could be affirmed by examining figure 4.41. The load-deformation ($\mu - R$) dynamic response for cases with or without degradation seems to follow the same trend confirming the previous conclusion. The ductility capacity though equals 10.4 for a non-degraded system, 7.8 for a moderately degraded system, and 5.8 for a severely degraded system. The corresponding $R_{fail}$ values are 9.4, 7.4, and 5.8 respectively. The presence of the cap, however, contributes strongly to the failure mode as shown in figure 4.42. The earlier the onset of softening, the weaker the overall behavior and the earlier failure occurs. The $R_{fail}$ value drops from a value of 15.8 for a system with cyclic degradation only to 4.2 for a brittle system.

The same conclusion drawn for bilinear systems applies for Clough systems, as shown in figures 4.43 and 4.44. The only exception is that Clough systems dissipate less energy, and thus the strength level is higher than in bilinear systems. The corresponding $R_{fail}$ value is thus considerably larger than for bilinear systems. The value for pinching systems is even larger than for Clough systems, as shown in figures 4.45 and 4.46, since their hysteretic energy dissipation is minimal.

### 4.4.1.4 Long Period Structures ($T = 2.0 \text{ sec}$)

The behavior of structures with period of 2sec. is similar to that of structures with period of 1sec., as shown in figures 4.47 to 4.52. The load-deformation dynamic response for degrading and non-degrading systems follows the same trend, with degradation affecting only the failure point. Hysteretic energy dissipation is much less than in
structures with period of 1sec. due to the fact that a smaller number of cycles are observed, which resulted in smaller $R_{fail}$ values. For Bilinear systems, the $R_{fail}$ value ranges between 9.8 for severely degrading structures and 19.0 for non-degraded structures. For Clough systems, the $R_{fail}$ value is even higher and ranges between 11.0 and 19.4 for severely degraded and non-degraded systems respectively. Pinching systems exhibit the highest $R_{fail}$ value, ranging between 14.2 and 23.8 for severely degraded and non-degraded systems respectively.

Figure 4.29  Bilinear Model Ductility, $R = 4$ and $T = 0.2$ sec
Figure 4.30  Bilinear Model Ductility, $\gamma = 100$ and $T = 0.2$ sec

Figure 4.31  Clough Model Ductility, $R = 4$ and $T = 0.2$ sec
**Figure 4.32**  Clough Model Ductility, $\gamma = 100$ and $T = 0.2$ sec

**Figure 4.33**  Pinching Model Ductility, $R = 4$ and $T = 0.2$ sec
Figure 4.34 Pinching Model Ductility, $\gamma = 100$ and $T = 0.2$ sec

Figure 4.35 Bilinear Model Ductility, $R = 4$ and $T = 0.5$ sec
Figure 4.36  Bilinear Model Ductility, $\gamma = 100$ and $T = 0.5$ sec

Figure 4.37  Clough Model Ductility, $R = 4$ and $T = 0.5$ sec
Figure 4.38  Clough Model Ductility, $\gamma = 100$ and $T = 0.5$ sec

Figure 4.39  Pinching Model Ductility, $R = 4$ and $T = 0.5$ sec
Figure 4.40  Pinching Model Ductility, $\gamma = 100$ and $T = 0.5$ sec

Figure 4.41  Bilinear Model Ductility, $R = 4$ and $T = 1.0$ sec
Figure 4.42  Bilinear Model Ductility, $\gamma = 100$ and $T = 1.0$ sec

Figure 4.43  Clough Model Ductility, $R = 4$ and $T = 1.0$ sec
Figure 4.44  Clough Model Ductility, $\gamma = 100$ and $T = 1.0$ sec

Figure 4.45  Pinching Model Ductility, $R = 4$ and $T = 1.0$ sec
Figure 4.46 Pinching Model Ductility, $\gamma = 100$ and $T = 1.0$ sec

Figure 4.47 Bilinear Model Ductility, $R = 4$ and $T = 2.0$ sec
Figure 4.48  Bilinear Model Ductility, $\gamma = 100$ and $T = 2.0$ sec

Figure 4.49  Clough Model Ductility, $R = 4$ and $T = 2.0$ sec
Figure 4.50  Clough Model Ductility, $\gamma = 100$ and $T = 2.0$ sec

Figure 4.51  Pinching Model Ductility, $R = 4$ and $T = 2.0$ sec
Figure 4.52  Pinching Model Ductility, \( \gamma = 100 \) and \( T = 2.0 \) sec

4.5  Seismic Fragility Analysis

Seismic fragility curves are plots that describe the probability of a system to reach or exceed different degrees of damage, including possible collapse. Earlier work focused on developing seismic fragility curves of systems for several values of a calibrated damage index. A damage index is a factor that represents the degree of damage of the structure, and typically ranges from 0 to 1, with the value of 1 representing complete collapse. Collapse was therefore expressed implicitly as the state of the structure when its damage index approaches a value of 1. In this study, seismic fragility curves for a collapse criterion are developed in an explicit form using the new degraded material models. A relationship between IDA plots, such as the ones presented earlier, and fragility curves exist. The relationship is described next.
4.5.1 IDA and Fragility Relationship

Figure 4.53 explains the relationship between the Incremental Dynamic Analysis (IDA) and Fragility curves. The solid dot on the IDA curve represents the collapse point at a 50% probability, and the corresponding $R$ value represents the mean strength reduction factor at collapse. The challenge arises when we need to find values of strength reduction factor ($R$) corresponding to a specific probability of collapse other than 50% for design purposes. Fragility curves offer this advantage as they express the entire spectrum of collapse probability.

![Diagram of IDA and Fragility Curves](image)

**Figure 4.53  Relationship Between IDA & Fragility Curves**

While fragility curves are typically expressed as a function of spectral accelerations for a specific yield force $\eta$, they could be easily extended to cover different values of $\eta$. In this case, a relationship between spectral accelerations ($S_a$), and the global yield force $\eta$ has to be established to give the structural designer the flexibility to use fragility curves for any yield value. The yield force $\eta$ is defined as follow:
\( \eta \) = The ratio of yield force \( (F_y) \) to total weight of the structure \( (W) \).

It is well known that the response of a system to a scaled earthquake record is identical to that of the same system with a yield force reduced by the same scale value and subject to the original unscaled earthquake record. Consequently, the yield force \( \eta \) is assumed to be inversely proportional to the spectral acceleration \( (S_a) \). This relationship could be used to modify seismic fragility curves, as explained in the next example.

Assume a fragility curve, shown in figure 4.54, is drawn for \( \eta = 1.8 \). The structural designer has found that the subject structure has \( \eta = 0.09 \) and he needs to find the probability of collapse of this building if hit by an earthquake having a spectral acceleration \( (S_a) \) of 1.2 g. In order to use this curve, the designer should modify the spectral acceleration \( (S_a) \) used to get the collapse probability. The new value will be:

\[
(S_a)_{modified} = 1.2 g \times \frac{1.8}{0.09} = 24 g
\]

It is worth mentioning that the combination of a specific \( \eta \) and \( (S_a) \) values correspond to a unique strength reduction factor \( R \), which is defined as:

\[
R = \frac{\eta_e}{\eta} \quad (4.5)
\]

Where:

\( \eta_e \) = The value of \( \eta \) when \( R = 1 \).
4.5.2 **Strength Reduction Factor** ($R$) and **Ratio of Yield Force to Total Weight** ($\eta$)

In the next discussion, the fragility curves for a collapse criterion, for the same ensemble of SDOF systems investigated earlier are developed, and are shown in figures 4.55 to 4.78. The data are smoothed using lognormal distribution functions. The 50% collapse probability point corresponds to the point identified with a ‘*’ in the mean IDA plots presented earlier, as explained before. The earthquake records were scaled to a common value. The yield force $\eta$ is assumed to equal 0.2, however plots for different values of $\eta$ could be easily estimated through proper scaling, as discussed earlier. A discussion on the behavior of the different structures investigated is presented next.

4.5.2.1 **Short Period Structures** ($T = 0.2$ sec)

Figure 4.55 shows the fragility curve for a collapse criterion for a bilinear model with fixed envelope values but with different degradation parameters. All degradation...
parameters $\gamma$ are assumed to be equal. A value of $\gamma = 150$ corresponds to a system with low degradation, a value of $\gamma = 100$ corresponds to a system with moderate degradation, and a value of $\gamma = 50$ corresponds to a system with severe degradation. As in the previous study, the cap displacement $\delta_{cap}$ is assumed to equal 4 times the yield displacement. From the plot, it can be shown that short period structures are susceptible to failure under earthquake excitations. A system with a cap and with no cyclic degradation has a 90% probability of failure if subject to a record with 1.8g spectral acceleration. A similar system with $R = 8$ will have the same failure probability if the spectral acceleration equals 0.9g. Cyclic degradation tends to increase the failure probability. A severe degradation system with $R = 4$ has a 90% failure probability if the spectral acceleration equals 1.08g.

Figure 4.56 shows the plot for the same system with a fixed degradation value $\gamma = 100$, but with different cap displacement values $\delta_{cap} = 1, 4, 6,$ and 8. The collapse probability, except for the first case, was not affected much by the onset of softening. The 90% collapse probability for all cases but the first is at a spectral acceleration of 1.5g, while it is at 0.75g for the first case. As discussed before, these results agree with the previously derived conclusions that suggest that for bilinear non-brittle systems, failure is most likely due to cyclic degradation effects, rather than to softening effects. For the brittle case, the early presence of the cap dominates the response and drives the system quickly into failure.

Figures 4.57 and 4.58 show plots similar to the ones described above, but for a Clough model. Degradation affected the failure rate of the structure; a system with a cap and no cyclic degradation has a 90% collapse probability if the spectral acceleration is
1.5g, while systems with low and severe degradation reach the same collapse probability if the spectral acceleration is 1.25g and 0.92g respectively. The effect of the presence of the cap is illustrated in figure 4.58. A system with no cap has a 90% collapse probability at a spectral acceleration of 1.8g, while systems with caps reach the same probability at a spectral acceleration around 1.25g irrespective of the value of the cap. These results confirm that Clough systems fail mainly due to softening effects as discussed earlier. Cyclic degradation accelerates failure, but to a much lesser extent than for Bilinear models. Also, from the preceding plots, it is concluded that Clough systems have a different failure mode than bilinear systems, but overall fail with a faster rate. The same results of a pinched model are shown in figures 4.59 and 4.60. The trend of the fragility curves is similar to that of Clough models. Systems with no cap tend to fail with a slower rate, while degraded systems have a collapse probability close to that of a similar Clough model.

4.5.2.2 Medium Period Structures \((T = 0.5 \text{ sec})\)

Figure 4.61 shows the collapse probability for a bilinear system. In this case, degradation has a big effect on the overall collapse probability. A system with low degradation has a 90% collapse probability at a spectral acceleration of 2.92g, while a system with severe degradation has the same collapse probability at a 1.8g spectral acceleration. These values are considerably higher than those of the case of short period structures, implying that medium period structures have a lower collapse probability if subject to the same earthquake record. Figure 4.62 shows the same plot for different cap displacements. As for the case of short period structures, very brittle systems failed with a
much faster rate (90% collapse probability at a spectral acceleration of 1g), while structures with different values of cap displacements reached the same collapse probability at a spectral acceleration that ranges between 2.44g and 3.32g.

The collapse probability of Clough models is also lower than the corresponding one for short period structures. A severely degraded system has a 90% collapse probability at a spectral acceleration of 2.12g, and a system with low degradation has the same probability at a spectral acceleration of 2.76g as shown in figure 4.63. Figure 4.64 also shows that the presence of a cap increased the 90% collapse probability from 2.6g for a case of cap displacement of 4 to 3.08g for a case of cap displacement of 8. The collapse probability of pinched models has a slower rate than that of both bilinear and Clough models, as shown in figures 4.65 and 4.66 implying that pinched models are less likely to collapse if subject to the same earthquake record.

4.5.2.3 Long Period Structures \((T = 1.0 \text{ sec and } T = 2.0 \text{ sec})\)

Figure 4.67 shows the fragility curve for a bilinear model with different degradation parameters. The collapse rate in general is slower than that of medium period structures. Degradation had a considerable effect on the collapse rate. A severely degraded system has a 90% probability of collapse at a spectral acceleration of 2.52g, while a system with low degradation has the same probability at 4.04g. Figure 4.68 shows the effect of the cap displacement on the behavior. Similar to medium period structures, the brittle case has a fast collapse rate, while non-brittle cases showed a much slower one. The behavior of Clough models shows a slower collapse rate than that of bilinear models, as shown in figure 4.69. The same is true for the case of a variable cap
displacement, shown in figure 4.70. The same trend is true if we compare the behavior of pinching models, shown in figures 4.71 and 4.72, to that of Clough models. In this case, a 90% collapse probability of a pinching model with moderate degradation and no cap is outside the limit of the graph. The collapse rate for long period structures with period T=2 sec. is the slowest for all cases, as shown in figures 4.73 to 4.78. A high spectral acceleration value is needed in this case in order to produce a 90% collapse probability. Degradation, in all cases, accelerated the rate of collapse.

Figure 4.55  Bilinear Model Fragility, $R = 4$ and $T = 0.2$ sec
Figure 4.56  Bilinear Model Fragility, $\gamma = 100$ and $T = 0.2$ sec

Figure 4.57  Clough Model Fragility, $R = 4$ and $T = 0.2$ sec
Figure 4.58  Clough Model Fragility, $\gamma = 100$ and $T = 0.2$ sec

Figure 4.59  Pinching Model Fragility, $R = 4$ and $T = 0.2$ sec
Figure 4.60  Pinching Model Fragility, $\gamma = 100$ and $T = 0.2$ sec

Figure 4.61  Bilinear Model Fragility, $R = 4$ and $T = 0.5$ sec
Figure 4.62  Bilinear Model Fragility, $\gamma = 100$ and $T = 0.5$ sec

Figure 4.63  Clough Model Fragility, $R = 4$ and $T = 0.5$ sec
Figure 4.64  Clough Model Fragility, $\gamma = 100$ and $T = 0.5$ sec

Figure 4.65  Pinching Model Fragility, $R = 4$ and $T = 0.5$ sec
Figure 4.66  Pinching Model Fragility, $\gamma = 100$ and $T = 0.5$ sec

Figure 4.67  Bilinear Model Fragility, $R = 4$ and $T = 1.0$ sec
Figure 4.68  Bilinear Model Fragility, $\gamma = 100$ and $T = 1.0$ sec

Figure 4.69  Clough Model Fragility, $R = 4$ and $T = 1.0$ sec
Figure 4.70  Clough Model Fragility, $\gamma = 100$ and $T = 1.0$ sec

Figure 4.71  Pinching Model Fragility, $R = 4$ and $T = 1.0$ sec
Figure 4.72  Pinching Model Fragility, $\gamma = 100$ and $T = 1.0$ sec

Figure 4.73  Bilinear Model Fragility, $R = 4$ and $T = 2.0$ sec
Figure 4.74  Bilinear Model Fragility, $\gamma = 100$ and $T = 2.0$ sec

Figure 4.75  Clough Model Fragility, $R = 4$ and $T = 2.0$ sec
Figure 4.76  Clough Model Fragility, $\gamma = 100$ and $T = 2.0$ sec

Figure 4.77  Pinching Model Fragility, $R = 4$ and $T = 2.0$ sec
Figure 4.78 Pinching Model Fragility, $\gamma = 100$ and $T = 2.0$ sec

4.5.3 Standard Deviation Parameter in Fragility Curves

The fragility curves in this research are based on the two parameter lognormal distribution function to get the S-shape curve. This approach was used by several researchers (Shinozuka et al. 2000) and proved to give precise results. The Probability Density Function (PDF) and the Cumulative Density Function (CDF) of the fragility curves follow the subsequent equations:

$$PDF : f(t) = \frac{1}{\sigma \sqrt{2\pi}} e^{-\left(\frac{1}{2\sigma^2}\right)(\ln(t) - \ln T_0)^2} \quad (4.6)$$

$$CDF : F(t) = \int_0^t \frac{1}{\sigma \sqrt{2\pi}} e^{-\left(\frac{1}{2\sigma^2}\right)(\ln(t) - \ln T_0)^2} \, dt \quad (4.7)$$
Where:

\[ \sigma = \text{The standard deviation, and} \]
\[ T_{50} = \text{The median of the results.} \]

The two parameters required for plotting the lognormal curve are the mean \( \mu \) and standard deviation \( \sigma \). For the lognormal distribution,

\[ \mu = \ln(\text{median}) \]  \hspace{1cm} (4.8)

In our case, \( T_{50} \) represents the value of spectral acceleration \( S_\alpha \) at probability of collapse of 50%. The variable \( t \) is the number of records that caused collapse to the system. The standard deviation parameter \( \sigma \) can be calculated by minimizing the sum of squared of the residual between the data and the lognormal model using a solver module. Figure 4.79 shows the effect of the standard deviation parameter \( \sigma \) on the fragility curve shape.

**Figure 4.79  Example of Use of Sigma in Fragility Curve**
A more general 3-parameter equation of the lognormal incorporates an additional parameter ($\theta$) which is called (shift) or (location) parameter. The 3-parameter equation is the same as for the 2-parameter except that ($t$) is replaced by ($t - \theta$). No collapse can take place before ($\theta$) or 0 in our research case. Figure 4.80 demonstrates the 3-parameter equation and the effect of ($\theta$) on the fragility curve.

The 3-parameter equation didn’t show significant reduction in the residual, therefore, the 2-parameter equation was found to be satisfactory to use.

Another simple technique to develop fragility curves using 2-parameter lognormal equations is to provide the designer with the median Incremental Dynamic Analysis (IDA) curve, and the corresponding 50% probability of collapse, in addition to the value of standard deviation parameter ($\sigma$). This process will give flexibility to get the probability of collapse at any value. For example, let us assume that it is required to get the probability of collapse at 84%. Instead of performing multiple dynamic analyses to
construct the $84^{\text{th}}$ percentile Incremental Dynamic Analysis (IDA) curve till collapse or to develop a complete fragility curve, one can plot the fragility curve knowing the standard deviation parameter ($\sigma$) and the probability of collapse at 50% from the regular median IDA plot.
CHAPTER 5
ASSESSMENT OF DEGRADED MDOF STRUCTURES

5.1 Degradation Effect on MDOF Systems Under Seismic Excitations

Degradation plays an important role in the behavior of MDOF structures under seismic excitations. To illustrate the effect of degradation, the response of a degraded MDOF structures is evaluated and compared to a similar non-degraded one. The structure selected is a 3-story building, whose equivalent SDOF system was evaluated earlier in chapter 4. Figures 5.1 and 5.2 show the response of the non-degraded building modeled as a multi degree of freedom system. The results are compared to those of the equivalent single degree of freedom system shown in figure 4.1. The record used is also the Imperial Valley 1979 recorded at station El Centro 1. The time history trend for the roof displacement of a non-degraded MDOF system in figure 5.1 is almost identical to the SDOF response shown in figure 4.1.

As we introduce low intensity degradation to the structure in figure 5.3, the failure occurs after 8.8 sec with a maximum roof displacement of 2.92 inches. The moment-rotation diagram for the rotational spring at column base is plotted in figure 5.4. Collapse, which is still designated by the (*) symbol, was found to be at 6.44 E-03 radians.

The effect of different levels of degradation becomes more obvious when studying figures 5.5 and 5.6 of a severe degradation case. Collapse is recorded after 8.6
sec and a roof displacement of 2.47 inches marking a faster failure to the same building with the same variables except the level of degradation. When comparing these results to its corresponding SDOF system, collapse was found to occur at the same time but with a larger roof displacement value. This signifies that the subject structure will experience higher straining actions requiring a higher capacity to withstand the new displacement demands. In other words, MDOF analysis incorporating degradation effects will result into a more accurate and safer design. The difference in roof displacement values between a non degraded SDOF and a severely degraded MDOF can reach up to 44% which is a significant value worth considering in structural design.

![MDOF Time History for Roof Displ., 3 Floors, Bilinear and No Degradation](image)

**Figure 5.1** MDOF Time History for Roof Displ., 3 Floors, Bilinear and No Degradation
Figure 5.2  MDOF Force-Displacement, 3 Floors, Bilinear and No Degradation

Figure 5.3  MDOF Time History for Roof Displ., 3 Floors, Bilinear and Low Degradation
Figure 5.4  MDOF Force-Displacement, 3 Floors, Bilinear and Low Degradation

Figure 5.5  MDOF Time History for Roof Displ., 3 Floors, Bilinear and Severe Degradation
Variation of material models used in analysis leads also to some deviation in displacement results. Figures 5.7 and 5.9 represent the time history for roof displacement and figures 5.8 and 5.10 represent the moment-rotation plots at column base for Clough and pinching models respectively for a severe degradation case. For the Clough model, collapse was recorded at 8.7 sec with 2.32 inches of roof displacement and a rotation of 5.6E-03 radians. Failure in the pinching model was more severe in its effect. Even though collapse took place at 8.7 sec too, the displacement achieved was equal to 2.72 inches, 10% more than that of an equivalent bilinear model and 17% more than that of an equivalent Clough model. As discussed earlier, this is probably due to the accelerated degradation effect. The rotation of the column base spring at collapse in the pinching model was equal to 6.6E-03 radians. The moment-rotation curves for Clough and pinching models reflected the material models characteristics previously explained.
Figure 5.7 MDOF Time History for Roof Displ., 3 Floors, Clough and Severe Degradation

Figure 5.8 MDOF Force-Displacement, 3 Floors, Clough and Severe Degradation
Figure 5.9  MDOF Time History for Roof Displ., 3 Floors, Pinching and Severe Degradation

Figure 5.10  MDOF Force-Displacement, 3 Floors, Pinching and Severe Degradation
The number of floors of the structure under study plays a key role in the results. Figures 5.11 and 5.13 display roof displacement time history plots of a 10 story building for low and severely degraded systems respectively. Imposing a low degradation to the material models, roof displacement at failure was equal to 5.66 inches at 16.8 sec compared to 2.92 inches for a similar 3 story. Roof displacement for a severely degraded MDOF system was equal to 5.92 inches but collapse occurred after 8.8 sec only. The equivalent displacement in the three-story structure was 2.47 inches.

Moment-rotation graphs for low and severe degradation of a ten-story structure are presented in figures 5.12 and 5.14. The low degradation case experienced more cycles of loading and unloading than the severely degraded one since it resisted collapse for almost twice the duration. The rotation of the base spring was equal to 7.73 E-03 and 9.43 E-03 radians for low and severe degradation respectively. The corresponding 3-story values were 6.44 E-03 and 5.61 E-03.
Figure 5.11  MDOF Time History for Roof Displ., 10 Floors, Bilinear and Low Degradation

Figure 5.12  MDOF Force-Displacement, 10 Floors, Bilinear and Low Degradation
Figure 5.13  MDOF Time History for Roof Displ., 10 Floors, Bilinear and Severe Degradation

Figure 5.14  MDOF Force-Displacement, 10 Floors, Bilinear and Severe Degradation
5.2 Effect of Higher Modes in MDOF Structures

Displacements of multistory buildings may not be always accurately estimated from analysis of an equivalent SDOF system. Chopra (2005) reviewed a comparative study carried out on several buildings to observe the difference between the actual displacement and the one obtained from the equivalent SDOF analysis. The results revealed a large discrepancy between both cases due to the effects of higher modes. Errors were brought to the fact that, for individual ground motions, the SDOF system may drastically deviate the yielding-stimulated permanent drift in the building response.

The effect of higher modes has not been accounted for in most seismic codes of practice for buildings, even in the recent FEMA-356 guidelines. The coefficients adopted in these guidelines are based on analysis of equivalent SDOF systems. The effect of higher modes was introduced in earlier studies through an additional coefficient $c_{MDOF}$ by Nassar and Krawinkler (1991). This new coefficient, though important, was not introduced in the FEMA guidelines for simplicity. A recent effort to improve the non-linear static seismic analysis procedure adopted in FEMA-356 was presented by Comartin et al. (2004). The study, however, still focused on equivalent SDOF systems and did not consider higher mode effects. In addition, some of the current coefficients values recommended in FEMA-356 are not confirmed by research results. Chopra (2005) gave an example of the coefficient limitation in the FEMA-356 equation. The $c_i$ factor, for instance, is restricted to a maximum value of 1.5, while this value is considered small when compared with dynamic response analysis results. Furthermore, the current procedures are still unable to determine the global MDOF collapse in an explicit form, which might be different than that of an equivalent SDOF system.
The abovementioned discussion exposes the need of a more accurate procedure to provide guidance for code user of SDOF systems to accurately estimate target displacement in MDOF systems. A new factor ($\Omega_{MDOF}$) is herein introduced that accounts for higher mode effects considering the presence of $P - \Delta$ due to gravity load together with material models degradation. A numerical study using a large ensemble of earthquake records is conducted to study this effect for a series of building models described afterward.

5.3 Building Models

The MDOF model used in this research was selected amongst three types of regular 2-dimensional single bay frames commonly used by researchers. Nassar and Krawinkler (1991) used these models, followed by Seneviratna and Krawinkler (1997), then Medina and Krawinkler (2003). The reason behind using these kinds of models was to examine the basic inelastic dynamic behavior patterns. Therefore, the torsional effects of 3-dimensional structures are not encountered. Plastic hinges are introduced to demonstrate different material models (e.g. bilinear, Clough and pinching).

The three models are illustrated in figures 5.15 to 5.17. They differ from each other in their yield mechanism. The first model, which is the “beam hinge” (BH), represents structures designed following the strong column – weak beam philosophy. In this model, plastic hinges will be formed only in beams ends and columns supports.
The second model, designated as “column hinge” (CH), represents structures designed following the weak column – strong beam philosophy. In this model, plastic hinges will be formed only in columns ends between stories and at columns supports.
The last model, referred to as “weak story” (WS), represents structures having a strength discontinuity in their first story. In this model, and unlike the two previous ones, the plastic hinges will be formed only in the first story.

Figure 5.17  Weak Story Model

The beam-hinge model was selected in this study since its collapse scenario is quite similar to a wide range of structures in common practice. New parameters were introduced to the building model to include their effect, such as degradation in material models, and $P - \Delta$ effect due to gravity loads.

In order to achieve the yield mechanism described above, the relative members’ strengths were tuned so that, under the 2003 IBC equivalent static load pattern, the plastic hinges in all beams and supports form simultaneously.
As for the relative member stiffness of the three models, they were also tuned so that, under the 2003 IBC equivalent static load pattern, the interstory drift in each story is identical leading to a straight line deflected shape.

In addition, the stiffnesses are selected such that the first mode period of each structure is equal to that given by the IBC code equation:

\[ T = 0.02h_{n}^{3/4} \]  \hspace{1cm} (5.1)

Where

\( T \) = First mode period in sec, and
\( h_{n} \) = Height of the building in feet above the base.

### 5.4 Selection of Representative Buildings

The period formula used in this thesis was the one specified in U.S. building codes such as IBC (2003), ATC3-06 (ATC 1978), SEAOC-96 (SEAOC 1996), and NEHRP-94 (NEHRP 1994). The formula is:

\[ T = C_{i}H^{3/4} \]  \hspace{1cm} (5.2)

Where

\( T \) = first mode period in sec;
\( H \) = height of the building in feet above the base; and
\( C_{i} \) = coefficient equals to 0.030 and 0.035 for R.C. and steel moment resisting frames (MRF) buildings respectively.

Medina and Krawinkler (2003) used another formula in their study:

\[ T = CN \]  \hspace{1cm} (5.3)

Where
\( T = \) first mode period in sec;

\( C = \) coefficient equals to 0.1 and 0.2 for steel and R.C. MRF respectively; and

\( N = \) number of stories in this building.

According to Goel and Chopra (1997), the formula \( T = 0.1N \) was recommended in the NEHRP-94 provisions as an alternative formula for R.C. and steel MRF buildings. But this simple formula was restricted to buildings not exceeding 12 stories in height and a minimum story height of 10 ft.

Tables 5.1 to 5.5 show, for each building analyzed in this study, the modal periods along with their corresponding damping ratios.

<table>
<thead>
<tr>
<th>Table 5.1</th>
<th>One-Story Period and Damping Ratios</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Period</td>
</tr>
<tr>
<td>1(^{st}) mode</td>
<td>0.129</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 5.2</th>
<th>Two-Story Periods and Damping Ratios</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Period</td>
</tr>
<tr>
<td>1(^{st}) mode</td>
<td>0.217</td>
</tr>
<tr>
<td>2(^{nd}) mode</td>
<td>0.064</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 5.3</th>
<th>Three-Story Periods and Damping Ratios</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Period</td>
</tr>
<tr>
<td>1(^{st}) mode</td>
<td>0.294</td>
</tr>
<tr>
<td>2(^{nd}) mode</td>
<td>0.099</td>
</tr>
<tr>
<td>3(^{rd}) mode</td>
<td>0.046</td>
</tr>
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</table>
Table 5.4  Five-Story Periods and Damping Ratios

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<th>Mode</th>
<th>Period</th>
<th>Damping Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1\textsuperscript{st} mode</td>
<td>0.431</td>
<td>0.0500</td>
</tr>
<tr>
<td>2\textsuperscript{nd} mode</td>
<td>0.160</td>
<td>0.0231</td>
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<tr>
<td>3\textsuperscript{rd} mode</td>
<td>0.085</td>
<td>0.0235</td>
</tr>
<tr>
<td>4\textsuperscript{th} mode</td>
<td>0.051</td>
<td>0.0356</td>
</tr>
<tr>
<td>5\textsuperscript{th} mode</td>
<td>0.034</td>
<td>0.0582</td>
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</table>

Table 5.5  Ten-Story Periods and Damping Ratios

<table>
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<th>Mode</th>
<th>Period</th>
<th>Damping Ratio</th>
</tr>
</thead>
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<td>1\textsuperscript{st} mode</td>
<td>0.725</td>
<td>0.0500</td>
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<tr>
<td>2\textsuperscript{nd} mode</td>
<td>0.287</td>
<td>0.0216</td>
</tr>
<tr>
<td>3\textsuperscript{rd} mode</td>
<td>0.169</td>
<td>0.0160</td>
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<td>4\textsuperscript{th} mode</td>
<td>0.114</td>
<td>0.0162</td>
</tr>
<tr>
<td>5\textsuperscript{th} mode</td>
<td>0.081</td>
<td>0.0198</td>
</tr>
<tr>
<td>6\textsuperscript{th} mode</td>
<td>0.061</td>
<td>0.0265</td>
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<td>7\textsuperscript{th} mode</td>
<td>0.047</td>
<td>0.0359</td>
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<td>8\textsuperscript{th} mode</td>
<td>0.037</td>
<td>0.0479</td>
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<tr>
<td>9\textsuperscript{th} mode</td>
<td>0.030</td>
<td>0.0623</td>
</tr>
<tr>
<td>10\textsuperscript{th} mode</td>
<td>0.025</td>
<td>0.0790</td>
</tr>
</tbody>
</table>
Figure 5.18 Node Numbering
Figure 5.18 show the node numbering for the different floor levels models used in this research. At Columns bases, 2 nodes in series are introduced to connect the regular column section to the rotational spring. The same concept is applied to the floor beam ends where rotational springs are present at each extremity. Each floor beam in these models is divided by three nodes. The gravity loads are then applied at those nodes with a ratio of 25% of the load at right and left, and 50% at the middle. Tables 5.6 to 5.15 show the structural characteristics of each building analyzed.

**Table 5.6 ** One-Story Model Characteristics

<table>
<thead>
<tr>
<th>Floor</th>
<th>Column Inertia</th>
<th>Beam Spring Stiffness</th>
<th>Base Spring Stiffness</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st</td>
<td>20195.5</td>
<td>12201460.0</td>
<td>6100729.8</td>
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</tbody>
</table>

**Table 5.7 ** Two-Story Model Characteristics

<table>
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<th>Column Inertia</th>
<th>Beam Spring Stiffness</th>
<th>Base Spring Stiffness</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st</td>
<td>25614.5</td>
<td>15475440.0</td>
<td>7737720.2</td>
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<tr>
<td>2nd</td>
<td>12022.4</td>
<td>7263535.9</td>
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**Table 5.8 ** Three-Story Model Characteristics

<table>
<thead>
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<th>Column Inertia</th>
<th>Beam Spring Stiffness</th>
<th>Base Spring Stiffness</th>
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</thead>
<tbody>
<tr>
<td>1st</td>
<td>29128.5</td>
<td>17598487.0</td>
<td>8799243.4</td>
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<tr>
<td>2nd</td>
<td>21443.8</td>
<td>12955649.0</td>
<td></td>
</tr>
<tr>
<td>3rd</td>
<td>9950.8</td>
<td>6011956.1</td>
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### Table 5.9 Five-Story Model Characteristics

<table>
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<th>Floor</th>
<th>Column Inertia</th>
<th>Beam Spring Stiffness</th>
<th>Base Spring Stiffness</th>
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<tr>
<td>1&lt;sup&gt;st&lt;/sup&gt;</td>
<td>34835.8</td>
<td>21046656.0</td>
<td>10523328.0</td>
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<tr>
<td>2&lt;sup&gt;nd&lt;/sup&gt;</td>
<td>31420.9</td>
<td>18983457.0</td>
<td></td>
</tr>
<tr>
<td>3&lt;sup&gt;rd&lt;/sup&gt;</td>
<td>25307.1</td>
<td>15289718.0</td>
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<tr>
<td>4&lt;sup&gt;th&lt;/sup&gt;</td>
<td>17785.5</td>
<td>10745435.0</td>
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<td>5&lt;sup&gt;th&lt;/sup&gt;</td>
<td>7751.9</td>
<td>4683419.8</td>
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### Table 5.10 Ten-Story Model Characteristics

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<th>Floor</th>
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<th>Base Spring Stiffness</th>
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<tr>
<td>1&lt;sup&gt;st&lt;/sup&gt;</td>
<td>45574.7</td>
<td>27534704.0</td>
<td>13767352.0</td>
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<td>2&lt;sup&gt;nd&lt;/sup&gt;</td>
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<td>27024993.0</td>
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<td>3&lt;sup&gt;rd&lt;/sup&gt;</td>
<td>41709.7</td>
<td>25199626.0</td>
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<td>24135692.0</td>
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<td>35820.0</td>
<td>21641266.0</td>
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<td>15752116.0</td>
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<td>19774.9</td>
<td>11947322.0</td>
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<tr>
<td>9&lt;sup&gt;th&lt;/sup&gt;</td>
<td>13029.5</td>
<td>7871962.0</td>
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<td>10&lt;sup&gt;th&lt;/sup&gt;</td>
<td>5454.2</td>
<td>3295233.7</td>
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### Table 5.11  One-Story Model Springs Yield Characteristics

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<tbody>
<tr>
<td>1(^{st})</td>
<td>9330.0</td>
<td>6660.0</td>
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### Table 5.12  Two-Story Model Springs Yield Characteristics

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<td>1(^{st})</td>
<td>26800.0</td>
<td>14900.0</td>
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<tr>
<td>2(^{nd})</td>
<td>11100.0</td>
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### Table 5.13  Three-Story Model Springs Yield Characteristics

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<th>Floor</th>
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<th>Column Spring</th>
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<tr>
<td>1(^{st})</td>
<td>43900.0</td>
<td>23000.0</td>
</tr>
<tr>
<td>2(^{nd})</td>
<td>31300.0</td>
<td></td>
</tr>
<tr>
<td>3(^{rd})</td>
<td>12500.0</td>
<td></td>
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### Table 5.14  Five-Story Model Springs Yield Characteristics

<table>
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<tr>
<td>1(^{st})</td>
<td>61200.0</td>
<td>31200.0</td>
</tr>
<tr>
<td>2(^{nd})</td>
<td>54700.0</td>
<td></td>
</tr>
<tr>
<td>3(^{rd})</td>
<td>43800.0</td>
<td></td>
</tr>
<tr>
<td>4(^{th})</td>
<td>29400.0</td>
<td></td>
</tr>
<tr>
<td>5(^{th})</td>
<td>11100.0</td>
<td></td>
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</table>
### Table 5.15  Ten-Story Model Springs Yield Characteristics

<table>
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<th>Floor</th>
<th>Beam Spring</th>
<th>Column Spring</th>
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<tbody>
<tr>
<td>1st</td>
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<td>2nd</td>
<td>86400.0</td>
<td></td>
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<tr>
<td>3rd</td>
<td>81300.0</td>
<td></td>
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<td>4th</td>
<td>76700.0</td>
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<td>5th</td>
<td>68800.0</td>
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<td>6th</td>
<td>60000.0</td>
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<td>7th</td>
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<td>9th</td>
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<tr>
<td>10th</td>
<td>8530.0</td>
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#### 5.4.1 Properties of Building Models

As explained earlier in chapter 3, the first modal period is derived from an equation that depends on the structure height. Table 5.16 presents for each building model used in this study the total height in feet and the corresponding period. The floor height module was set to 12 feet.
Table 5.16   Building Models Total Height and Corresponding First Modal Period

<table>
<thead>
<tr>
<th>Number of Floors</th>
<th>Total Height (ft)</th>
<th>Period (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>One</td>
<td>12.0</td>
<td>0.129</td>
</tr>
<tr>
<td>Two</td>
<td>24.0</td>
<td>0.217</td>
</tr>
<tr>
<td>Three</td>
<td>36.0</td>
<td>0.294</td>
</tr>
<tr>
<td>Five</td>
<td>60.0</td>
<td>0.431</td>
</tr>
<tr>
<td>Ten</td>
<td>120.0</td>
<td>0.725</td>
</tr>
</tbody>
</table>

Table 5.17 displays the total base shear values for each of the buildings used in this study. The IBC (2003) equation was used to calculate the base shear. A distribution of the total base shear is then applied at each floor. The distribution follows NEHRP load pattern in order to get a linear slope for the first mode of the displaced floors under earthquake as demonstrated in figure 5.19.

![Deformed Shape Under NEHRP Load Pattern](image)

Figure 5.19   Deformed Shape Under NEHRP Load Pattern
Table 5.17  Base Shear Distribution at Each Floor Level (NEHRP Load Pattern, k=2)

<table>
<thead>
<tr>
<th>Floor #</th>
<th>One Floor</th>
<th>Two Floor</th>
<th>Three Floor</th>
<th>Five Floor</th>
<th>Ten Floor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>220</td>
<td>147</td>
<td>110</td>
<td>58</td>
<td>23</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>293</td>
<td>220</td>
<td>117</td>
<td>45</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
<td>330</td>
<td>175</td>
<td>68</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td>234</td>
<td>90</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td>292</td>
<td>113</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>135</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>158</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>180</td>
</tr>
<tr>
<td>9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>203</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>225</td>
</tr>
<tr>
<td>Total Base Shear</td>
<td>220</td>
<td>440</td>
<td>660</td>
<td>876</td>
<td>1240</td>
</tr>
</tbody>
</table>

5.4.2  DRAIN-2DX Runner / Parser

One of the major disadvantages of DRAIN-2DX is the limitation of records used per single analysis. The program allows only one record at a time to be analyzed. In order to use the 80 records contained in the four bins to perform fragility and displacement estimates curves, around 50 thousands simulations were required. The need to have an automated process mounted at this point and brought up the concept for a new software named Drain Runner.
Drain Runner is a software that deals with the end user of DRAIN-2DX in a friendly interface shown in figure 5.20. The program was created using “visual basic.net” which is a subset of the “visual studio.net” package. The software allows the user to pick the number of floors along with the earthquake records to be used in analysis. The output is then stored in an output file sorted by earthquake records.

![Drain Runner User Interface Window](image)

**Figure 5.20  Drain Runner User Interface Window**

A step counter and a progress bar features were added to the software to allow the user to monitor the current progress of the analysis. An application for the use of those two attributes is clear when studying a pushover analysis for one of the MDOF systems. The initial force is increased by increments defined by the user. Since DRAIN-2DX terminates the analysis at collapse, the user can check exactly the amount of force that caused failure by adding the increments up to the posted step number.
5.5 Displacement Estimates of MDOF Degraded Structures

The new factor \( \Omega_{\text{MDOF}} \) introduced to estimate displacements of MDOF structures combines higher mode effects and \( P-\Delta \) effect due to gravity along with material degradation. More importantly, the factor is a significant tool to predict collapse in its explicit form and not as a number on a damage index scale. The following equation defines in details the new factor \( \Omega_{\text{MDOF}} \):

\[
\Omega_{\text{MDOF}} = \frac{\delta_{\text{MDOF}}}{\delta_{\text{SDOF}}} = (5.4)
\]

Where

\( \delta_{\text{MDOF}} \) = The inelastic degraded roof displacement of the MDOF system, and

\( \delta_{\text{SDOF}} \) = The corresponding inelastic degraded roof displacement of the equivalent SDOF system.

The mean value of the coefficient \( \Omega_{\text{MDOF}} \) was derived through analytical simulations using the building models and the ensemble of earthquake records discussed earlier. The cases considered were bilinear and modified Clough models for one, three, five and ten floors. The bilinear model was subjected to a moderate degradation, equivalent to \( \gamma = 100 \), which is representative of steel buildings. In the modified Clough model, degradation was severe \( (\gamma = 50) \), which is typical of concrete buildings. Those two values resulted from the experimental and analytical calibration of the degradation factors for both materials. Strength reduction factors \( R \) used in the analysis were 4, 6 and 8. Collapse was represented in the plots by the (*) symbol. The dotted lines represent an estimate of the \( \Omega_{\text{MDOF}} \) value until the collapse point. \( \Omega_{\text{MDOF}} \) was plotted as a function of the fundamental period for the MDOF systems analyzed.
Figure 5.21 shows the MDOF effect for a non-degraded system with $R = 8$. The value of $\Omega_{MDOF}$ is higher for long period structures than for short period structures. Specifically, $\Omega_{MDOF} = 1.4$ for 10-story buildings, and 1.15 for two-story buildings. This is expected since it is well known that equivalent SDOF systems produce larger errors if used to simulate the behavior of MDOF systems with large number of degrees of freedom. The plot was also constructed for systems with cap and low cyclic degradation with cap. In this case, degradation had a severe effect on the inelastic MDOF displacements, particularly for short period structures. The trend of the plot changed and became decreasing. In other words the value of $\Omega_{MDOF}$ for short period structures was much higher than for long period structures (2.2 and 1.5 for 2 and 10-story structures).

The effect of degradation was even more evident for systems with moderate or severe degradation, as shown in figure 5.22 with presents the value of $\Omega_{MDOF}$ for a bilinear system with $R = 4$. Two cases are presented, a system with cap only, and a system with cap and moderate cyclic degradation. From the plots, it is clear that degradation dominated the behavior of short period structures, while the trend for long period structures was similar to non-degraded cases. In addition, systems with both cap and cyclic degradation had higher $\Omega_{MDOF}$ values than systems with cap only. For instance, for a five story structure, $\Omega_{MDOF}$ was higher by 9%. Furthermore, collapse was observed for short period structures; but for systems with both cap and cyclic degradation, it occurred at higher $\Omega_{MDOF}$ values than for systems with cap degradation only.
Figure 5.23 shows the mean $\Omega_{MDOF}$ plot for a Clough model with $R = 4$. Two cases are shown, a system with cap only, and a system with cap and severe cyclic degradation. The effect of degradation was more severe than for similar bilinear models. The trend of the curve was dominated by degradation, with larger values for short period structures than for large period structures. In addition, the $\Omega_{MDOF}$ values for the system with cap and cyclic degradation were much higher than those of the system with cap only. The increase in the value for a five-story structure was up to 32%. Moreover, collapse was observed in the severely degraded case for structures with period less than 0.4 sec.

Strength reduction factors ($R$) affected the ratio significantly. For a non-degraded ten-story bilinear structure with $R = 4$, $\Omega_{MDOF}$ was equal to 1.38 (Figure 5.22). This ratio increased by 7% in figure 5.24 to reach 1.47 when $R = 6$. The difference was even more noticeable when $R = 8$ where $\Omega_{MDOF}$ was equal to 2.46 (Figure 5.26). The percentage of increase was 78% compared to $R = 4$ and 67% compared to $R = 6$.

In some cases with higher strength reduction factor values as in figures 5.24 to 5.26 degradation conditions led to collapse for all buildings considered. This explains why the degraded cases are not plotted in those figures. Moreover, some degraded SDOF cases did not collapse, while their corresponding MDOF cases collapsed. Therefore it was not possible to estimate the displacement ratio $\Omega_{MDOF}$. In this case, analysis of equivalent SDOF systems is not considered accurate, and MDOF analysis is necessary to estimate the seismic behavior. These cases include systems with strength reduction factors $R > 6$, or systems with severe degradation parameters $\gamma < 50$. 
The previous conclusions have been drawn considering mean statistical values. To further examine the potential for collapse of MDOF systems, fragility analysis for a collapse criterion of MDOF systems need to be considered. The newly developed fragility curves will cover the entire spectrum of collapse probability, and are described in the next section.

Figure 5.21  $\Omega_{\text{MDOF}}$ – Bilinear Model – $R = 8$
Figure 5.22 \( \Omega_{MDOF} \) – Bilinear Model – \( R = 4 \)

Figure 5.23 \( \Omega_{MDOF} \) – Clough Model – \( R = 4 \)
Figure 5.24 $\Omega_{MDOF}$ – Bilinear Model – $R = 6$

Figure 5.25 $\Omega_{MDOF}$ – Clough Model – $R = 6$
5.6 Seismic Fragility of Collapse for MDOF Systems

A wide variety of structures was considered to conduct the fragility analysis in order to have a thorough evaluation of the collapse potential of MDOF systems. The variables were number of stories, degradation level, and material models. One, two and three stories were selected to represent short buildings whereas five and ten stories represented relatively long buildings. The levels of degradation considered in this study are: low, moderate, and severe degradation. An assumption was made, similar to the study for SDOF systems, which is that the four types of degradation: strength, stiffness, accelerated stiffness, and cap were present simultaneously when considering any level of degradation. Bilinear, modified Clough and pinching models were the three different material models considered. The pinching model was not used for cases of five and ten
stories since pinching models represent wood structures and from a practical point of view it is unlikely to find wood structures that high.

Each case from the above different combinations would result in a single point on the fragility curve for a single earthquake record and a single earthquake ground acceleration. In order to get results as accurate as possible, the four sets of scaled earthquake records, mentioned earlier, were used in the different cases. The plots are generated for a yield factor $\eta$ that corresponds to a strength reduction factor $R = 1$. The yield factor $\eta$ in this case is defined as the yield base shear divided by the total weight of the structure.

![Figure 5.27 One Floor, Bilinear Model, $\eta = 0.2231$](image-url)
Figure 5.28  Two Floors, Bilinear Model, $\eta = 0.1226$

Figure 5.29  Three Floors, Bilinear Model, $\eta = 0.1101$
Figure 5.30  Five Floors, Bilinear Model, $\eta = 0.0881$

Figure 5.31  Ten Floors, Bilinear Model, $\eta = 0.0622$
Figure 5.32  One Floor, Clough Model, $\eta = 0.2231$

Figure 5.33  Two Floors, Clough Model, $\eta = 0.1226$
Figure 5.34 Three Floors, Clough Model, $\eta = 0.1101$

Figure 5.35 Five Floors, Clough Model, $\eta = 0.0881$
Figure 5.36  Ten Floors, Clough Model, $\eta = 0.0622$

Figure 5.37  One Floor, Pinching Model, $\eta = 0.2231$
Figure 5.38  Two Floors, Pinching Model, $\eta = 0.1226$

Figure 5.39  Three Floors, Pinching Model, $\eta = 0.1101$
The fragility curves for bilinear one, two, three, five, and ten story structures are shown in figures 5.27 to 5.31. The curves are plotted for a yield base shear corresponding to a strength reduction factor $R = 1$. The plots reveal that tall structures (five and ten story structures) have a lower probability of collapse. The 50% and 90% probability of collapse for a moderately degraded one-story structure occur at 1.5g and 2.3g respectively. The same values for a ten-story structure are 3.3g and 6g. From the plots it is clear that short story ductile systems have at least a 50% probability of collapse. Tall structures, however, have a lower probability of collapse, but could still exceed a 50% probability for very ductile systems.

The fragility curves for the Clough model are shown in figure 5.32 to 5.36. As for the bilinear model, the probability of collapse for taller structures is less than that of short structures. The 50% and 90% probability of collapse for a moderately degraded one-story structure occur at 1.15g and 2.2g respectively. The same values for a ten-story structure are 2.55g and 5.05g. Ductile short-story systems have a probability of collapse larger than 50%, while tall structures have a probability of collapse less than 50%, similar to bilinear systems. From the plots, Clough models have a slower rate of collapse than bilinear models. This conclusion was also true for SDOF systems analyzed in chapter 4. The effect of the level of degradation, however, is smaller than for bilinear models.

Figures 5.37 to 5.39 show the fragility curves for the pinching model. Pinching models in general have a lower probability of collapse than similar bilinear and Clough models. Ductile pinching models with $R > 6$ still have a probability of collapse larger
than 50%. The level of degradation for pinching models did not have a major effect on the behavior though.

The final conclusion is that short period structures are more susceptible to damage than long period structures. Therefore, damage and collapse are expected in low rise buildings more than in high rise buildings for the same ground acceleration and degradation levels. Bilinear models have a higher collapse probability followed by Clough and pinching models. The effect of the level of degradation is more apparent for bilinear models than for Clough or pinching models. Furthermore a mean (50%) collapse probability is expected for ductile \( R > 6 \) severely or moderately degraded short structures for all material models. This conclusion doesn’t hold true however for structures with number of stories greater than 5.

### 5.7 Practical Use of Proposed Design Curves

To illustrate the use of the previously developed design curves, the following example is considered. Consider a five-story building with period \( T = 0.431 \) sec and subject to earthquake record defined using USGS LA 10/50 spectrum. The following steps are used to evaluate the parameter \( \eta \) needed to use the design curves:

- From the USGS spectrum, calculate the spectral acceleration of the building. For the case of a period of 0.431 sec, this value equals 1.1 g.
- As a designer, select the desired strength reduction factor \( R \). In this example, \( R \) is selected to equal 4.
- Assuming elastic behavior, determine the maximum spring forces if the building is subject to the record scaled to a value of 1.1 g determined in the first step. In this case
these values are: first floor beam spring 61200 kip-ft, second floor 54700 kip-ft, third floor 43800 kip-ft, fourth floor 29400 kip-ft, fifth floor 11100 kip-ft and column spring 31200 kip-ft.

- Divide the forces obtained in the previous step by the selected R value to obtain the yield forces of the springs. For the case of the selected \( R = 4 \) these values are: first floor beam spring 15300 kip-ft, second floor 13675 kip-ft, third floor 10950 kip-ft, fourth floor 7350 kip-ft, fifth floor 2775 kip-ft and column spring 7800 kip-ft.

- Conduct a pushover analysis of the building using a triangular load pattern to develop the base shear – roof displacement curve. From the curve identify the value of the yield base shear and in this case the value is equal to 22.02 kips.

- The parameter \( \eta \) is defined as the ratio of the yield base shear to the total weight of the building which is 1000 kips. In this example \( \eta \) is equal to 0.022.
CHAPTER 6
SUMMARY AND CONCLUSION

6.1 Summary

The research study presents a discussion on the behavior and collapse potential of degrading structures under seismic excitations. The study is essential for the design evaluation phase of a performance-based earthquake design process, particularly for collapse prevention limit states. New constitutive models for degrading structures are developed and added to the material library of the nonlinear structural analysis program DRAIN-2DX. These material models represent bilinear models for steel structures, Clough models for concrete structures, and pinching models for timber structures. All models include a strength softening branch, referred to as a cap, to model strength degradation under monotonic loads. An 8 parameter energy-based model was developed to model four different types of cyclic degradation: Yield (Strength) degradation, Unloading stiffness degradation, Accelerated stiffness degradation, and Cap degradation. Collapse is explicitly defined if the material completely loses its strength either due to severe cyclic deterioration or to strength softening. The degradation parameters were calibrated against available experimental data. A set of earthquake records is selected to conduct studies on degrading systems. An initial study proved that efficient scaling of the records can reduce considerably the variability in results without introducing any bias,
and thus a much smaller number of non-linear analyses are needed. The set of scaled earthquake records was used to conduct statistical analysis of a large ensemble of generic structural systems. The systems represented both SDOF and MDOF structures with different fundamental periods of vibration. For MDOF structures gravity loads and $P-\Delta$ effects were accounted for in the model. In addition, several other parameters were investigated. These included material type, yield force, and levels of degradation. For each study conducted, the degrading behavior was evaluated and compared to the non-degraded behavior through several numerical relationships. The relationships were expressed with plots that included: mean and 84% percentile inelastic displacement ratios, mean MDOF displacement ratios, mean incremental dynamic analysis plots, and seismic fragility curves for a collapse criterion. The potential for collapse was explicitly studied in the fragility analysis, and was investigated in the other analytical studies. The study proved to be essential for evaluating current analysis techniques and new seismic design codes for buildings. Several conclusions were drawn from the study and are explained in the next section.

6.2 Conclusion

The following conclusions were drawn from the study:

- Scaling of earthquake records proved to be an efficient way to reduce the variability in results, and therefore a smaller number of nonlinear analyses is needed to conduct statistical studies. A difference less than 10% between the scaled and unscaled responses is typically observed ensuring that no bias is introduced by the scaling procedure. In addition, the dispersion in results is considerably lower for the scaled
set of records. This conclusion is valid for both non-degraded and degraded, SDOF and MDOF systems.

- For SDOF systems, degradation had a great effect on the inelastic displacement ratios, especially for short period structures where the inelastic displacements were quite larger than the corresponding displacements of non-degraded systems. For very short period SDOF structures, collapse is typically expected even for systems with low strength reduction factors. For long period structures, the well-known equal displacement rule is preserved even for degrading systems. In this case, collapse is not expected even for systems with large strength reduction factors.

- For short period SDOF structures, bilinear models collapse due to cyclic degradation effects, due to the large energy dissipation. In this case, degradation strongly reduces the ductility capacity. Clough systems collapse mainly due to softening effects. Cyclic degradation accelerates failure, but to a much lesser extent than for bilinear models. Pinching models are strongly affected by accelerated degradation. In this case, the inclusion of degradation actually increases the ductility capacity.

- For medium period SDOF structures, bilinear models fail due to cyclic degradation effects, while both Clough and pinching models fail mainly due to softening effects, with the degradation accelerating the failure rate. For bilinear models, degradation strongly affects the ductility capacity. For Clough and pinching models, cyclic degradation has a smaller effect on the ductility capacity, while cap degradation strongly affected the ductility capacity.
For long period SDOF structures, the load-deformation dynamic response for degrading and non-degrading systems follows the same trend, confirming the equal displacement rule. Degradation in this case affects only the failure point.

For short period SDOF systems, bilinear models have the fastest collapse rate, followed by Clough and pinching models. The failure mode of each system is different though.

Medium period SDOF systems have a slower collapse rate than short period systems, with the bilinear model having the fastest collapse rate.

The collapse rate of long period SDOF structures is very slow, with most systems needing very intense earthquake records to collapse.

Degradation had a great effect on the displacements of MDOF structures. The effect of higher modes is typically larger for long period non-degraded structures than for short period ones. Degradation, however, strongly affected the displacements of higher modes of short period structures, while its effect on the displacements of higher modes of long period structures was less pronounced. The final outcome was that the effect of higher modes was eventually smaller for long period degraded structures than for short period ones. This conclusion is particularly true for severely degraded structures, but is also valid for systems with low degradation.

The analysis of MDOF structures showed that in some cases the MDOF structure collapsed, while its equivalent SDOF system did not collapse. In this case, analysis of equivalent SDOF systems is not considered accurate, and MDOF analysis is necessary to estimate the seismic behavior. These cases include systems with short
periods \( T < 0.4 \) sec.) and strength reduction factors \( R > 6 \), or systems with severe degradation parameters \( \gamma < 50 \).

- Seismic fragility analysis of MDOF structures showed that tall structures have a much lower probability of collapse than short story structures (3-story and less). Short period structures are therefore more susceptible to damage and collapse than long period structures.
- Bilinear MDOF structures have also a faster collapse rate followed by Clough and pinching structures. The effect of the level of degradation is more apparent for bilinear models than for Clough or pinching models.

6.3 Recommendations

While the current study was based on extensive statistical evaluation of the inelastic seismic behavior of both SDOF and MDOF degrading structures and their potential for collapse, further work still needs to be performed in order to better understand the complex degrading behavior of structures before fully implementing it in codes of practice. The following is a list of recommendations and ideas for possible future research work:

1. The current study focused on MDOF buildings designed according to the Beam-Hinge (BH) concept, where collapse mechanisms formed due to plastic hinges occurring at the beam ends and column base. The study needs to be extended to include also buildings designed according to the Column-Hinge (CH) and Weak First Story (WS) concepts.
2. The current study did not account for shear degradation of members, but rather focused on their flexural degradation. It is important to include shear effects in the analysis of MDOF structures particularly for shear critical members and columns. Earlier studies showed that the loss of shear capacity for columns might cause a subsequent loss of axial capacity which might lead to partial or full collapse of the entire building.

3. The current study was conducted for two-dimensional structures only. While the 2D assumption might be valid for regular symmetric buildings, it might not hold true for buildings with plan irregularities, where torsional deformations become an issue. It is therefore important to extend the current study to three-dimensional structures.

4. The current study assumed no coupling effect between the different force actions acting on a structural element. The combined effect of bending, shear, axial and torsional forces is a complex, yet important effect that needs to be addressed for collapse analysis of building structures.

5. The current study accounted for P-Delta effect due to gravity loads along with material degradation. The effect of P-Delta needs to be further explored. In other words, the effect of excluding P-Delta and accounting only for material degradation needs to be fully investigated.

6. The current study focused only on generic regular building structures. The study needs to be extended to evaluate the behavior of buildings with stiffness or mass discontinuities. Furthermore, the behavior of shear wall types of buildings needs to be also investigated.
7. The study assumed the building structures analyzed are subject to ordinary earthquake records recorded on stiff soil or soft rock. The study needs to be extended to consider other types of earthquake records, such as near fault and long duration records.

8. Current seismic specifications do not account for soil-structure interaction effects even though previous research studies confirmed the importance of this effect. The effect of the soil interaction on the collapse potential of MDOF buildings needs to be evaluated.

9. The current study focused on evaluating inelastic target roof displacements of degrading MDOF structures, in addition to the potential of the structure for collapse. Other seismic demand parameters such as inter-story drifts and maximum plastic rotations need to be also investigated for degrading structures. Strength parameters such as maximum base and story shears, and base and story overturning moments need to be also studied.

10. The study needs to investigate also the behavior of non-structural components of degrading buildings. This could be accomplished by investigating the effect of degradation on both, floor accelerations and velocities.

11. Finally, a thorough evaluation and modifications of existing seismic design codes needs to be performed based on the outcomes of this study in order to reflect the effect of degradation and potential for collapse. Such effect is not accounted for in existing methods for seismic demand evaluation.
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ABOUT THE AUTHOR

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